JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL ENGINEERING STREAM

EFFECT OF SPECIMENS' HEIGHT TO DIAMETER RATIO ON UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL: A CASE OF JIMMA CITY

By:

HAILE TSEGAY

A Research thesis submitted to the School of Graduate Studies of Jimma University in partial fulfillment of the requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

June, 2021

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Main-Advisor: Damtew Tsige (PhD)

Co-Adviser: Yada Tesfaye (MSc)

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Jimma, Ethiopia

APPROVAL SHEET

I, the undersigned certify that the thesis entitled: **"Effect of Height to Diameter Ratio on Unconfined Compressive Strength of Cohesive Soil, A Case of Jimma Town"** is the work of Haile Tsegay and has been accepted and submitted for examination with my approval as university advisor in partial fulfillment of the requirements for Degree of Master of Science in Geotechnical Engineering.

Name	**Signature**	**Date**
1. **Main-Advisor: Damtew Tsige (PhD)**	____________________	__________________
2. **Co-Adviser: Yada Tesfaye (MSc)**	____________________	__________________

As member of Board of Examiners of the MSc Thesis Open Defense Examination, we certify that we have read, evaluated the thesis prepared by Haile Tsegay and examined the candidate. We recommended that the thesis could be accepted as fulfilling the thesis requirement for the Degree of Master of Science in Geotechnical Engineering.

1. **Endalu Tadele (PhD)**	____________________	____________________
[External Examiner]	[Signature]	[Date]
2. **Adamu Beyene (MSc)**	____________________	____________________
[Internal Examiner]	[Signature]	[Date]
3. **Worku Firomsa (MSc)**	____________________	____________________
[Chair Person]	[Signature]	[Date]

DECLARATION

I declare that this research entitled **"Effect of Height to Diameter Ratio on Unconfined Compressive Strength of Cohesive Soil, A Case of Jimma Town"** is my own original work, and has not been submitted as a requirement for the award of any degree in Jimma University or elsewhere.

Haile Tsegay ____________________ ______ /_____ /_________

[Researcher] [Signature] [Date]

As master's research advisors, I hereby certify that I have read and evaluated this MSc thesis prepared under my guidance by Haile Tsegay entitled **"Effect of Height to Diameter Ratio on Unconfined Compressive Strength of Cohesive Soil: A Case of Jimma Town".**

Damtew Tsige (PhD) ____________________ ______ /_____ /_________

[Main Adviser] [Signature] [Date]

Yada Tesfaye (MSc) ____________________ ______ /_____ /_________

[Co-Adviser] [Signature] [Date]

ABSTRACT

Unconfined compression test is commonly used to detemine the undrained shear strength (Su) and cohesions (Cu) of cohesive soils. The test results are significantly affected by specimen sizes. This leads to overestimate or underestimate for the shear strength of cohesive soils and causes a problem for geotechnical analysis and designs. Therefore, this study assessed the effect of height to diameter ratio on unconfined compressive strength of cohesive soil in Jimma Town by testing a compacted cylindrical specimen at maximum dry density and optimum moisture content with a range of 1 to 3 height to diameter ratio for 38mm, 50mm and 100mm specimen diameters. Disturbed sample specimens were collected from five test pits of different locations in Jimma town. Accordingly, the classification of the collected soil specimens determined from grain size analysis and Atterberge limits tests. The maximum dry unit weight and optimum moisture content of the collected sample specimen also determined from the standard proctor compaction test. Hence, the unconfined compression test for the compacted soil specimens at maximum dry density and optimum moisture content conducted for nine height-to-diameter ratios of the three diameters of cohesive soil specimen. The laboratory test results showed that; the unconfined compressive strength value of cohesive soil drops rapidly with increasing of both height-to-diameter ratios and specimens’ diameter. But the value was stable from 1.75 to 2.25 H/D ratios and the UCS of 2.00 H/D ratio was the closest to mean of all height to diameter ratio for all diameter of specimen. As the specimens’ diameter and H/D ratio increase, the axial strain of peak UCS value was decreased. Similarly, the gap between the axial strains of peak UCS value for the smallest and the largest H/D ratio was decreased with the increasing of specimens’ diameter. The number of failure patterns failed in clear and distinct shear failure planes within the series H/D ratios increased as the diameter of the specimens increased. Therefore, both height to diameter ratio (specimens’ height) and diameter has a significant effect on the unconfined compressive strength of cohesive soils. Then it is recommended to use height to diameter ratio of 2 and larger diameter specimen to reduce the effect of specimens’ size for the test. However, platen end frictions and rate of loading strain may be taken in to consideration for further studies.

Keywords: *Unconfined compressive strength, Height to Diameter ratio, Cohesive soil*

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LIST OF ABBREVIATIONS

AASHTO	American Association Society of Highway and Transport Organization
ASTM	American Society for Testing and Materials
BSI	British Standard Methods of Test for Soil
H/D	Height to Diameter Ratio
JIS	Japanese Standard Association
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
UCT	Unconfined Compression Test
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
TS	Turkish Standard
A_o	Initial cross-sectional area
H_o	Specimens initial height
A_c	Corrected cross-sectional area
Cu	Undrained cohesion
q_u	Unconfined compressive strength
ε	Axial strain

CHAPTER ONE

1. INTRODUCTION

1.1. GENERAL

The shear strength of soils is significant in Geotechnical Engineering problems such as; the bearing capacity of foundations, the slope stability of dams and embankments, and lateral earth pressure on retaining walls (Yilmaz et al., 2019). Hence, shear strength of soil materials is determined in the laboratory by direct shear test, triaxial test, and unconfined compression test. For fine-grained soil materials, unconfined compression test is important to determine unconfined compressive strength (q_u) and undrained shear strength (Cu) of soils in terms of total stress.

Unconfined compressive strength of cohesive soil is determined by the unconfined compression test which is widely used for designing and analyzing dam foundations, building foundations, and other geotechnical engineering fields. The unconfined compression test is simple and quick in an investigation of unconsolidated undrained shear strength of undisturbed, remolded, and compacted cohesive soil sample specimens. The test is more applicable for fast construction projects; where, pore water doesn’t have enough time to drain (Yilmaz et al., 2019). Accordingly, the test is performed to determine undrained shear strength and unconfined compressive strength for a structures rested on fine-grained or cohesive soils (ASTM, 2007b).

In terms of total stress, the unconfined compression test is a significant and widely used method of determining cohesive soil shear strength. However, the test result is affected by the size of the specimen. The size of a specimen is significant in determining the unconfined strength of cohesive soil, which increases or decreases the test result (Wang et al., 2020). Consequently, based on the results of unconfined compressive tests, the shear strength parameters of cohesive soil are overestimated or underestimated. Thus, the cost of construction projects increases as a result of overestimation of the cost of treating for underestimated soil shear strength. Similarly, if the soil's shear strength is overestimated, the cost of construction for maintaining collapsed structures will increase.

The specimen size of unconfined compressive strength is the height to diameter ratio of the sample specimen. The height to diameter ratio of unconfined compression soil test is different in different authorities. According to the ASTM D 2166 standard, the height to diameter ratio of a cylindrical sample specimen is recommended between 2.00 and 2.50 with a diameter of greater than 33mm (ASTM, 2007b). British standard methods of testing for soil (BSI 1377-7) use a 2.00 height to diameter ratio with a diameter between 35mm and 100mm (British Standards Institution, 2015). Japan's Geotechnical Society (JGS A 1216) suggests that 35mm specimen diameter and 80mm height (Japanese Geotechnical Society, 2016). The Turkish Standards Institution (TS 1900-2) recommends that the aspect ratio of an unconfined compression test specimen's diameter should be between 38mm and 50mm with a height to diameter ratio of 2.00 (Türk Standardlari Enstitüsü, 2006). Braja M. Das also suggested that the height to diameter ratio of the unconfined compressive strength test sample specimen is between 2.00 and 3.00 (Das, 2002). But a recent study shows that the minimum aspect ratio of unconfined compression test (i.e. 1.00) also gives a representative test result for zero end platen frictions (Wang et al., 2020). Therefore, the recommendation values of specimens' diameters and heights to diameter ratio by the different countries has no agreement among the standard codes (Güneyli & Rüşen, 2016).

Therefore, this study was assessed the effect of cohesive soil specimen size on unconfined compressive strength and its relations in Jimma town, Ethiopia.

1.2. Statement of the Problem

For designing and analyzing foundations, embankments, slope stability, and other infrastructure construction, the unconfined compression test is a very effective and cost-effective way to determine the unconsolidated undrained shear strength of cohesive soils (Kalinski, 2011). However, due to the end paten effect (end restraint) and the buckling effect, the test result is influenced by specimen size.

To avoid a restrained end loading plate and buckling effect, the sample specimen for an unconfined compression test should have a sufficient height to diameter ratio. If the height to diameter ratio of the specimens is small, the entire specimen is restrained by the friction of the end loading plate and a possible intervention of failure planes occurs. By preventing the development of the weakest failure plane, the unconfined compressive strength of the soil sample can be increased. If the specimen is long, it tends to buckle, develops local bulging failure. Consequently, this decreases the unconfined compressive strength of the soil specimens (Moores & Hoover, 1966; Verveckaite et al., 2007; Wang et al., 2020). To avoid this problem there is no agreement between different country's standard code and different literatures. Consequently, the test value varies and this causes a problem in designing of building foundations and other geotechnical applications (Ghosh, 2013).

Disturbance of sample specimens is also an additional difficulty to perform the test. During drilling, sampling, transportation, and preparation of the specimen for a test; the specimen often breaks, create fissures and loses its moisture. This is difficult to extract the required number of samples and recommended specimen sizes. Accordingly, it increases the cost of investigation and takes time to drill, and to take sample again.

Therefore, determining the effect of height to diameter ratio of unconfined compressive strength for cohesive soil is important for designing and analyzing of geotechnical application for a predominant soil type of fine grain or cohesive soils.

1.3. Research question

1. What is the engineering classification of the collected soil specimens from Jimma Town?
2. How could the variation of height to diameter ratio of specimen affects the unconfined compression strength of cohesive soil?
3. Which height to diameter ratio could optimize the unconfined compressive strength for cohesive soils?

1.4. Objectives

1.4.1. General Objectives

The general objective of this research is to determine the effect of specimen's height to diameter ratio on unconfined compression strength of cohesive soil in Jimma town.

1.4.2. Specific Objectives

- To determine the classification of the collected soils samples from the study area
- To determine the unconfined compressive strength of the cohesive soil at maximum dry density and optimum moisture content by varying specimen height to diameter ratio.
- To recommend optimum height to diameter ratio and to develop a corrective formula for unconfined compressive strength of the optimum height to diameter ratio of cohesive soil specimens in the study area.

1.5. Scope of the Research

The scope of this study was limited to determining the effect of the specimen's height and diameter on unconfined compressive strength of compacted (disturbed) cohesive soils. The laboratory tests conducted for this study were Grain size analysis test, Atterberge limits, Standard proctor compaction test, and Unconfined compression test according to ASTM standards. The unconfined compression test specimen was then compacted at OMC and MDD. The diameters of compacted specimens were 38mm, 50mm, and 100mm. Each specimen's diameter had a height to diameter ratio of 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, 2.50, 2.75, and 3.00.

1.6. Limitation of The Study

Due to the limitation of time and budget, the soil specimens were collected from five test pits from Jimma Town. The soil sample for all test types was used disturbed (compacted) soil specimen. During the conducting of the unconfined compression test, constant strain loading rate was taken and end platen friction was not considered.

1.7. Significant of The Study

This study contributes to a better understanding of the impact of specimen height and diameter on cohesive soils' unconfined compression strength. Based on the relationship between height to diameter ratio and peak UCS value, stress-strain curve behavior, and failure patterns for the soil type of the study area, this study recommends the appropriate height to diameter ratio for three specimens' diameter.

In this study, corrective formulas were also made for the specimens which are difficult to get the required size (the right of height to diameter ratio) to correlate the unconfined compressive strength of disturbed (small size) cohesive soil specimens with the optimum height to diameter ratio of the specimen.

1.8. Organization of The Study

There are five chapters in this thesis. The general introduction to the study background, statement of the problem, research questions, general and specific study objectives, scope and limitation of the study, significance of the study, and structure of the thesis are all covered in chapter one. In the second chapter, literature is briefly reviewed. The literature review of cohesive soils is presented in UCS and UCT; factors affecting the test, scientific papers, and technical standards on the effect of specimen size and research gap have been discussed. Chapter three deals the study area, study design, study population, study variables, data collection processes and analysis, ethical considerations and data assurances. The analysis and discussion on the objective of the thesis is made in chapter four. Chapter five closes the whole team of the thesis by making conclusion and recommendation.

Finally, appendices of different tabular calculations, tables of index properties, UCS values, stress-strain curves, photo of failure patterns and photo of test procedures for each specimen are attached.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. Overview

The unconfined compression test is the simplest, easiest, and least expensive test for investigating the shear strengths of cohesive and semi-cohesive soils in either the undisturbed or disturbed soil specimens. For cohesion-less or coarse-grained soils, however, it is difficult to determine undrained shear strength using an unconfined compression test due to the lack of cohesive behavior in those soil materials (Güneyli & Rüşen, 2016).

The test is used on soil construction sites where the rate of construction is high and pore water does not have enough time to drain (Yilmaz et al., 2019). Accordingly, the result is used to estimate the short-term bearing capacity for foundations and the slopes stability of fine-grained soils (Kalinski, 2011). Similarly, it is used to determine the stress–strain characteristics under fast (undrained) loading conditions.

2.2. Factors Affecting Unconfined Compressive Cohesive Soils

Unconfined compression test results are influenced by disturbance of sample specimen during drilling and transporting, in preparation, and testing procedures. The loss of natural moisture contents and densities of soil specimens are caused by disturbance of the sample specimens. Accordingly, losing of initial densities of the specimen lowered its unconfined compressive strength then gives underestimated unconfined compressive strength (Zhang & Chen, 2018). The moisture content of sample specimens also significantly affects the shear strength parameter. Generally, an increase in moisture content decreases shear parameters of cohesive soils (Bláhová et al., 2013; Ghosh, 2013).

According to (Ghosh, 2013) the undrained van shear strength for compacted soil is steeply decreased with the increase of water content as shown Figure 2-1 below having all other parameters are constant (Ghosh, 2013).

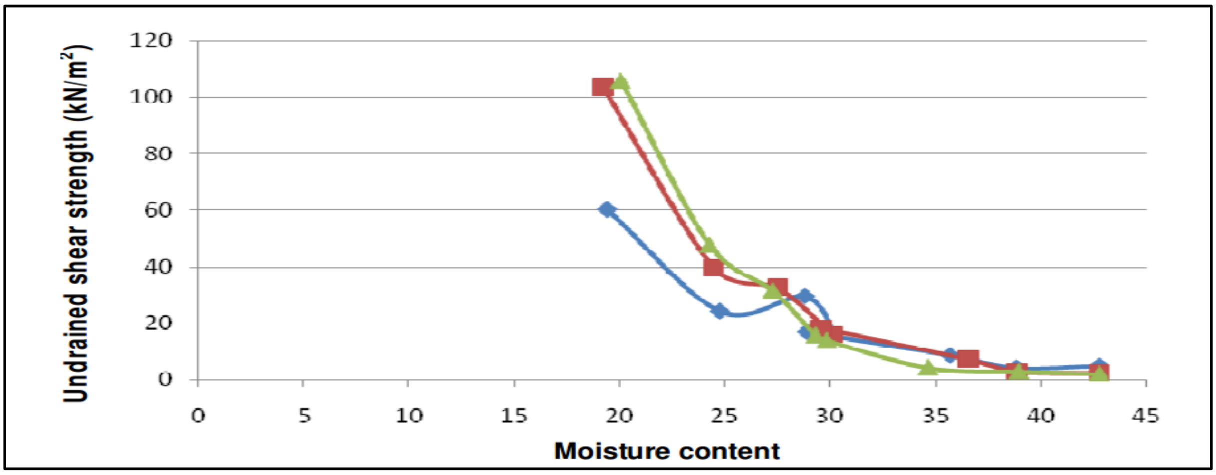

Figure 2-1 Effect Moisture Content on Undrained Van Shear Strength (Ghosh, 2013)

During testing producers, the shear strength is influenced by strain rate in a strain-controlled shear test. Due to the interaction of water and soil skeleton, shear strength increases as the strain rate increases, but decreases as the rate slows (Rabia, 2018; Sina & Magued, 2015). Hence, different standards recommend different strain rates for unconfined compression test. From these standards: ASTM D-2166 recommends 0.5%/minute to 2%/minute (ASTM, 2007b) whereas, Japanese Geotechnical Society Standards suggests that 1%/minute (Japanese Geotechnical Society, 2016).

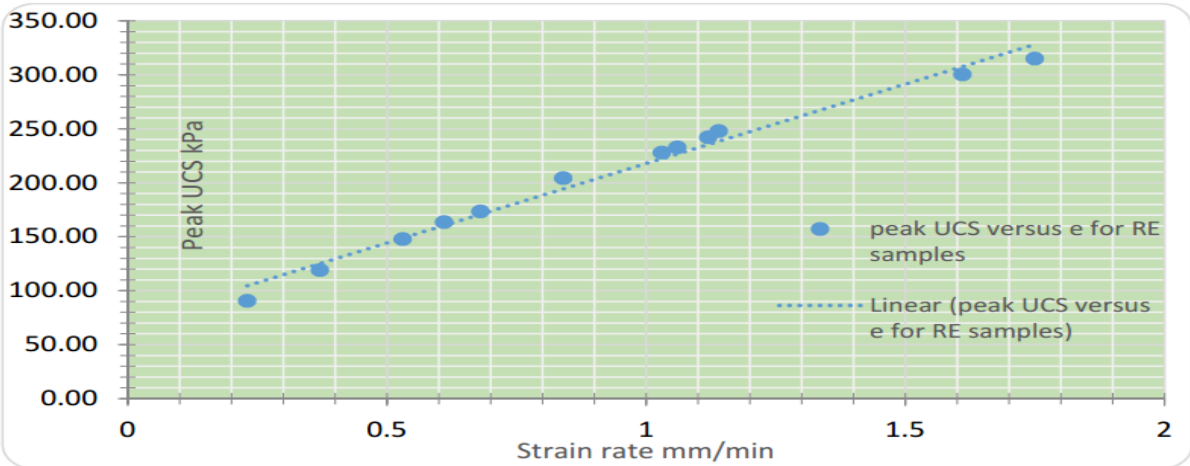

Figure 2-2 Effect of Strain Rate on Unconfined Compressive Strength (Rabia, 2018)

2.3. Effect of Specimen Size on Unconfined Compressive Strength of Cohesive Soil

For unconfined compressive strength of cohesive soil, sample specimens with a minimum diameter of 1.3 inches and a height to diameter ratio of 2 to 2.5 are recommended by (ASTM, 2007b). According to Turkish standards, the diameter of an unconfined compressive test of soil specimen is 50 mm and the height to diameter ratio is 2.00 (Türk Standardlari Enstitüsü, 2006). Japan's Geotechnical Society (JGS A 1216) suggests that 35mm specimen diameter and 80mm height (Japanese Geotechnical Society, 2016). Whereas Das B. recommended a 2 to 3 height to diameter ratio (Das, 2002).

The size of the specimen is important for determining cohesive soil shear parameters (Omar & Sadrekarimi, 2015). Following that, various studies in the last few decades assessed the effect of specimen size in unconfined compression tests, which are summarized below.

(Ang & Loehr, 2003) conducted an unconfined compression test for four different specimen sizes of reinforced silty-clay soil. The reinforced silty clay soil was compacted at maximum dry density and optimum moisture content. From the test results, they found that the specimen size of the test has a great effect on the unconfined compressive strength of the soils at maximum dry density and optimum moisture content.

(Shogaki, 2007) described a new procedure for performing unconfined compressive tests on small sample specimens (15 mm diameter and 35 mm height). The small size then varied by 10% of the mean UCS value from the ordinary diameter (35 mm diameter and 80 mm) height. Then (Shogaki, 2007) concluded that; since the variation of UCS value was in the range of 15-17% as indicated by (Matsuo & Shogaki, 1988), a small-sized specimen could be used instead of the large specimen size.

According to (Amiejus et al. 2009), as cited in (Yilmaz et al., 2019) a height to diameter ratio of 1.00 was a more representative value of an internal frictional angle and cohesive value due to the decreasing friction between specimen and endplates. The height to diameter ratio of 2.00 achieved a much greater shear strength parameters value. For zero endplate and specimen friction (Wang et al., 2020) also recommends a minimum ratio of 1.0 can effectively reduce defects in the middle of sampled specimens.

(Güneyli & Rüşen, 2016) carried out unconfined compression tests considering the effect of height to diameter (L/D) ratio. For that purpose, they used cylindrical soil samples with 11 different (L/D) ratios of 0.5 to 3 prepared from four different types of clay soil using 48mm diameter. As a result of the unconfined compression tests performed on compacted clay soils, the results decrease linearly as the (L/D) ratios increase, as shown in Figure 2-3 below. This reduction indicates that the aspect ratio of the cylindrical specimen is an important factor to consider when measuring the compressive strength of a soil specimen in a UCS test.

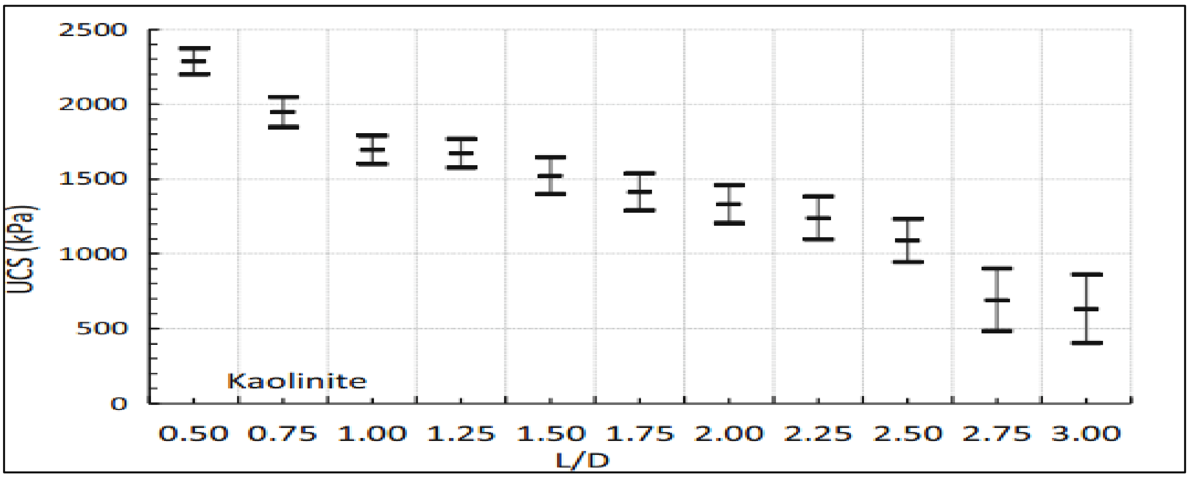

Figure 2-3 Effect of Length to Diameter for UCS Test of Kaolinite Clay (Güneyli & Rüşen, 2016)

They were also plotted Ratios of UCS to various standard UCS values (UCS/UCS_2, $UCS/UCS_{2.5}$ and UCS/ UCS_3) against H/D ratio where, UCS_2, $UCS_{2.5}$, and UCS_3 are the UCS values obtained with an L/D ratio of 2, 2.5, and 3, respectively. Linear regression lines were fitted to the resulting data for the four soils; thus, the general correction formula for L/ D ratios of 2, 2.5, and 3 were given by

$$\mathrm{UCS}_{(2;\,2:5\text{ and }3)} = \frac{\mathrm{UCS}}{a+b(\frac{L}{D})}$$

Then they found for each of the soils tested in their study, the resulting regression equations and values of a and b for H/D ratios of 2, 2.5, and 3 are listed in Table 2-1

Table 2-1 Regression of the Ratios UCS/UCS (2, 2.5, 3) Against L/D Ratio (Güneyli & Rüşen, 2016)

Regression equations $\mathrm{UCS}_{(\mathrm{Standared}\ \frac{L}{D}\ \mathrm{ratio})} = \dfrac{\mathrm{UCS\ (measured)}}{a - b(\frac{L}{D})}$	R^2	States
$\mathrm{UCS}_2 = \dfrac{\mathrm{UCS}}{1.83 - 0.39(\frac{L}{D})}$	0.91	Strong
$\mathrm{UCS}_{2.5} = \dfrac{\mathrm{UCS}}{2.01 - 0.43(\frac{L}{D})}$	0.92	Strong
$\mathrm{UCS}_3 = \dfrac{\mathrm{UCS}}{2.72 - 0.59(\frac{L}{D})}$	0.75	Moderately strong

(Merga, 2016) (unpublished thesis) also investigated on the effect of sample height to diameter ratio (H/D) on the stress-strain behavior of undrained shear strength of red clay soil in Addis Ababa. The unconfined compression tests were conducted on the undisturbed and compacted red clay soils at different moisture contents including (i.e., optimum moisture content and moisture content at the dry and wet side of OMC). (Merga, 2016) took four specimens H/D ratio with a constant 38mm specimen diameter. Then he found that the unconfined compressive strength of the Betel red clay soil was decreased with the increasing height-to-diameter (H/D) ratio of the specimens as shown in Figure 2-4.

Accordingly, (Merga, 2016) conclusions, based on the uniformity of the stress-strain curve, the failure patterns developed throughout the entire specimen's height and axial strain ranges at which the sample attains its peak strength for the investigated ranges of H/D ratio, it is possible to determine the undrained shear strength of red clay soil in Addis Ababa using samples with H/D = 1 to H/D = 2.

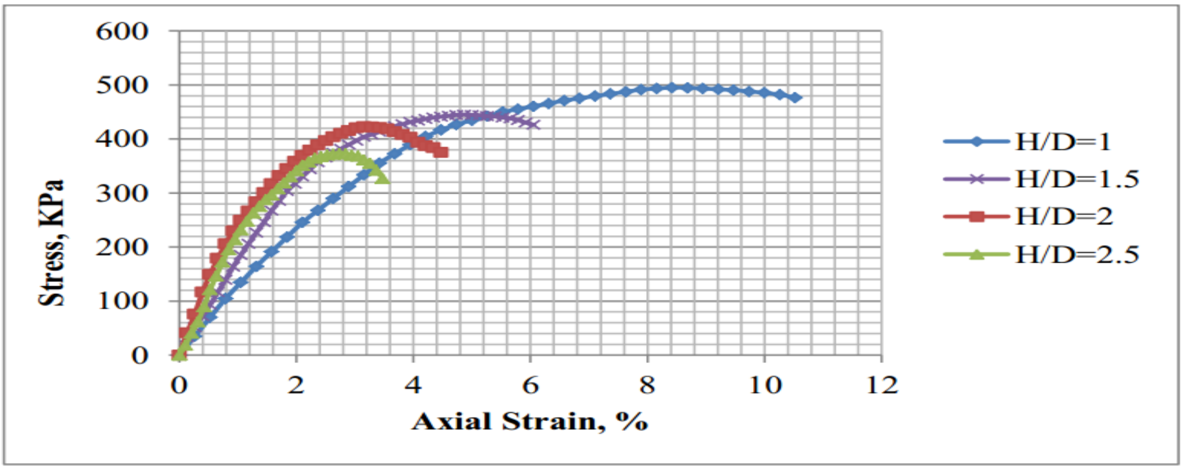

Figure 2-4 Stress Strain Behavior of Addis Ababa City Red Clay with Height to Diameter Specimen Ratio (Merga, 2016)

The failure pattern of an axially compressed cohesive soil specimen articulates the consistency of unconfined compressed strength of the specimen. A distinct vertical or inclined failure pattern indicates stiff soil. Whereas bulging failure patterns show for a soft soil specimen (Kalinski, 2011). The distinct vertical or inclined failure pattern may be splitting tensile, shear, and tensile shear failure patterns (Tang et al., 2016; Wang et al., 2020). Although stiff soil specimens fail in a distinct pattern under unconfined compression testing, due to specimen size, they also fail in a local bulging or throughout their entire length (Güneyli & Rüşen, 2016; Wang et al., 2020).

Kamei and Tokida, (1991) as cited in (Yilmaz et al., 2019) observed that, shear failure plans of unconfined compression test sample specimens before failures create a clear distinct failure line within sufficient distance from the lower and upper surface of specimen height for a sufficient heigh of specimens. This is because of the interaction of failure planes affects the test results (Moores & Hoover, 1966).

(Wang et al., 2020) conducted a recent study on specimen size and its effect on strength and failure mode in unconfined compressive strength tests for remolded earthen soil samples. They found that as the height to diameter ratio increases, the failure modes of sample specimens shift from tensile to shear, and defects in the middle of samples become more visible. The height-to-diameter ratio changed peak strain, elastic modulus, and residual stress. When the aspect ratio exceeds 1.0, the size effect in end frictionless samples reduces.

2.4. Summary of Literature Reviews

Various literature agrees, specimens’ size affects for the cohesive soil of unconfined compressive strength, but some literatures contradict for the issue (Shogaki, 2007; Yilmaz et al., 2019). (Yilmaz et al., 2019) agree that in compacted soil specimen size affects for unconfined compression test result. However, in high amount of moisture content greater than optimum moisture content, the size effect is negligible rather than it affects the amount of moisture content. To summarize, the size of specimens has a substantial effect on the shear strength of cohesive soil, according to the above literatures, and this is still a concern.

2.5. Research Gap

The height to diameter ratio of cohesive soil specimens of unconfined compression tests recommendation varies in various countries' standard codes and literatures. The effect of the specimen's height to diameter ratio on the unconfined compressive strength test of cohesive soil, however, has been the subject of several studies. However, those studies focused on the effect of specimen size on the constant sample specimen’s diameter of unconfined compression test of cohesive soils. Therefore, this study was conducted to determine the effect of the height to diameter ratio by varying both the diameter and height of cohesive soil specimens. Based on the soil conditions in the study area, this research also suggests an appropriate height to diameter ratio.

CHAPTER THREE

3. MATERIALS AND METHODOLOGY

3.1. Study Area

The study area is in Jimma town, which is the largest and oldest town in Southwestern Ethiopia, located at a latitude of 7040' N and longitude 36050' E in Oromia national state. It is 350km from Addis Ababa. The town has rolling terrain with an elevation ranging from 1670m to 1770m above mean sea level.

Jimma town is predominantly covered with red, black, and gray soil. The red-colored soils are found on rolling topography with high elevation and good drainage conditions. The black and gray soils, which cover the central and large part of the town, are found on the flat topography of the town with lower elevation and unfavorable drainage conditions (Jibril, 2017).

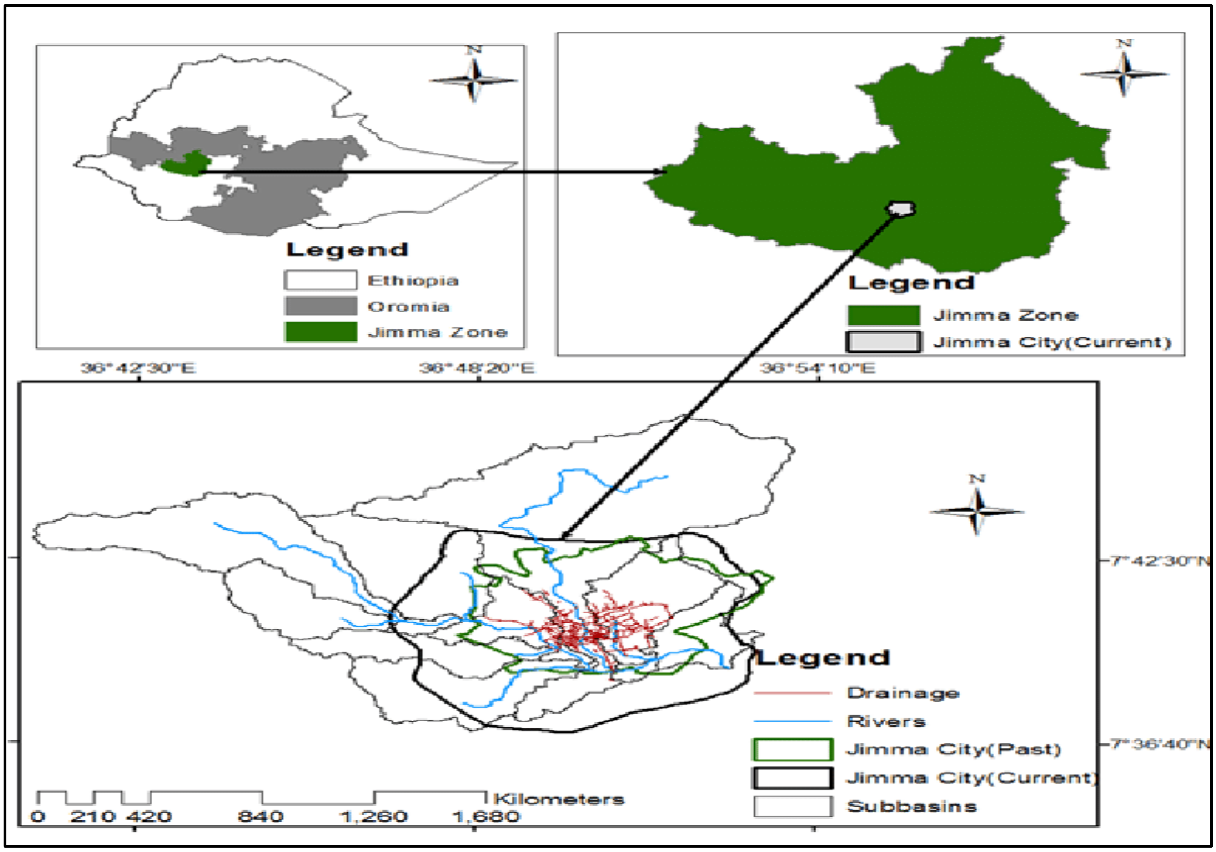

Figure 3-1 Map of study Area, Jimma City

3.2. Study Design

In this study, experimental approach study design was followed. To achieve the objectives of this research, the following methodologies was adopted.

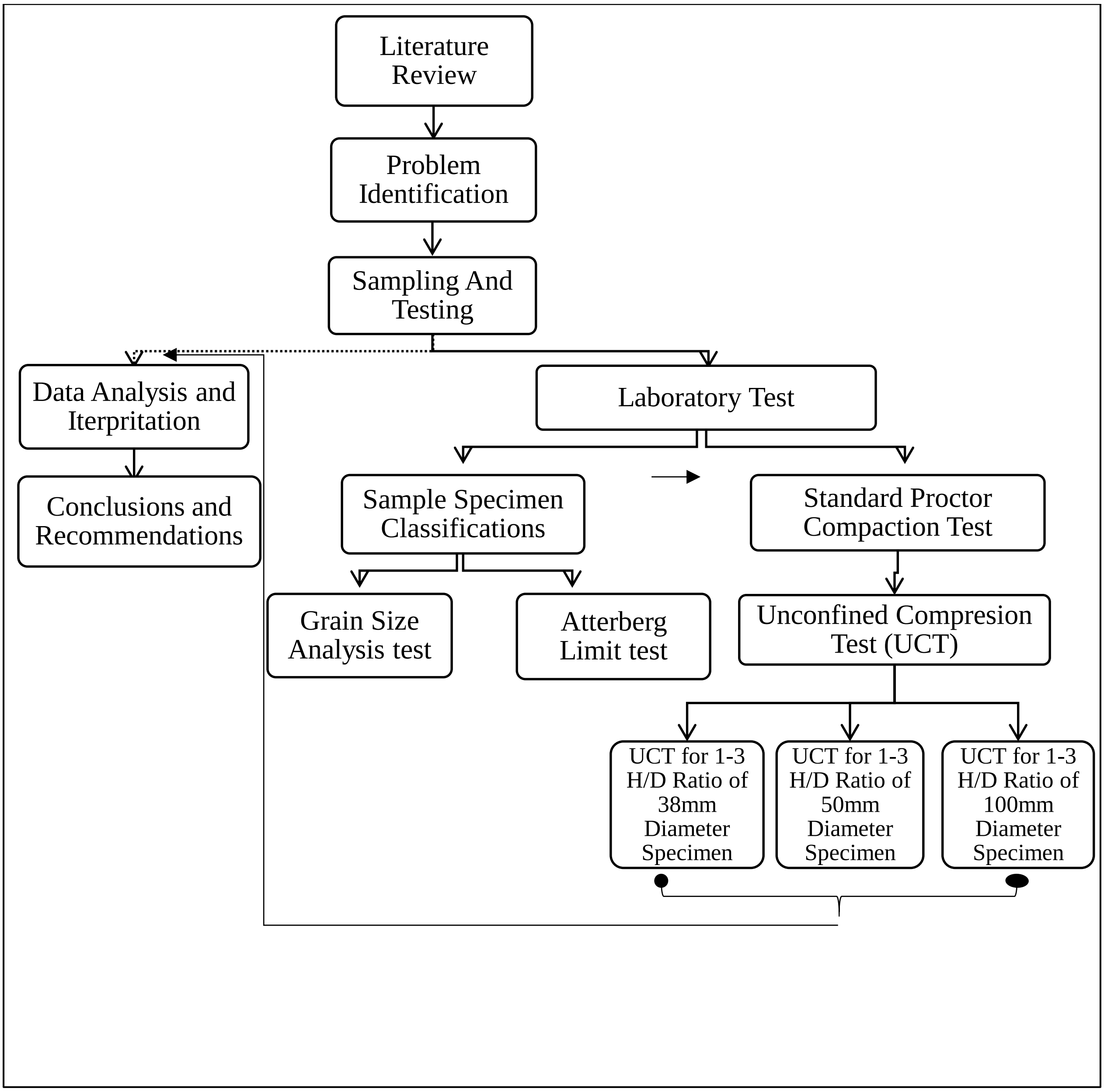

Figure 3-2 Study Design of Flow Chart

3.3. Study Population

The study population for this research were different types of cohesive soils in Jimma Town. The soil specimens were collected from five test pits to represent the different types of soils in the study area. i.e., from Aweytu, Ajip, Mariam Church, Saris and Jimma Institute of Technology campus (JIT).

3.3.1. Sample Size and Selection

For this study, the soil samples were collected using a disproportionate stratified probability sampling method which were two test pits from each red and black, one test pit from gray colored fine-grained soils. The collected samples for this study were disturbed and air-dried samples.

3.3.2. Sampling Techniques and Procedure

The soil samples were collected from five test pits at a depth of 3.0m below ground level. The test pits were excavated manually, and the collected samples were taken to the Jimma Institute of Technology-Geotechnical Engineering soil laboratory.

3.4. Study Variables

3.4.1. Independent Variables

The independent variables which were measured and manipulated to determine its relationship to observed the phenomena were:

- Particle grain size distribution and Atterberge limits,
- Optimum moisture content and maximum dry density,
- Diameter of sample specimens and their respective height to diameter ratios of the sample specimens.

3.4.2. Dependent Variables

The dependent variables which were observed and measured to determine the effect of the independent variables are;

- The unconfined compressive strength,
- stress strain curve, and
- failure patterns of the different diameter sample specimen with respective height to diameter ratios.

3.5. Data Collection Process

3.5.1. Field Survey

During the field survey, a preliminary visual survey was conducted on Jimma town's cohesive soils. To understand the general soil type of Jimma Town, the researcher used different soil investigation papers conducted on the study area.

3.5.2. Laboratory Tests

The laboratory tests conducted for this study were grain size analysis test, Atterberge limit, standard proctor compaction test, and unconfined compression test. For all laboratory tests, those samples were disturbed and air-dried soil specimens according to ASTM standards. The preparations of the materials and the test procedures of the conducted laboratory tests are discussed as follows:

I. Grain Size Analysis Test

The purpose of this test in this study is to determine the percentage of fine grain particles passing through 0.075micrometer sieve size. As a result, 1,000g of air-dried soil specimen was soaked for 24 hours before being transferred to a 0.075micrometer sieve. The clean slurry fine particles retained on that sieve size were then oven-dried for approximately 24 hours at 105+0.1oc. Finally, the percentage of fine-grain particles in oven-dried specimens retained by 0.075micrometer sieve size was determined using the ASTM D-421 (ASTM, 1998) standard.

II. Atterberge Limit Test

The moisture content of cohesive soil as it changes from a liquid to a plastic state and then from a plastic to a solid state is critical for determining soil classification. A 250g air-dried soil specimen was then soaked for 24 hours of the specimens that passed through a 0.425 mm sieve. Thus, from the 250gram soaked soil specimen; liquid limit, plastic limit, and plastic index were determined for each pit according to ASTM D 4318 – 00 (ASTM, 2000) standard.

III. Compaction Test

A compaction test is performed to determine the maximum dry density of a soil at its optimum moisture content by removing the air voids from soil particles. There are two types of compaction tests, modified and standard proctor tests. The modified proctor test performs for heavy construction projects while the standard proctor test applies for general constructions. For this research, a standard proctor test was selected for all test pits.

The specimens used in this test were air dried and passed through a 4.75 mm sieve. In order to fully wet, the specimens were soaked for about 4 hours and then compacted by 2.45kg rammers from 30.5 cm height of drops in three layers and 25 number of drops for each layer using a mechanical compactor machine according to ASTM D 698-07 (ASTM, 2007a) standard.

IV. Unconfined compression test

The main part of this research is an unconfined compression test. The test was programmed to assess the effect of the specimen's height to diameter ratio on the unconfined compressive strengths of cohesive soils. The unconfined compression tests were conducted for compacted sample specimens at their maximum dry density and optimum moisture content; under strain-control, at a constant strain loading rate according to ASTM D-2166 (ASTM, 2007b).

The compacted sample type was chosen for the unconfined compression test sample. Compacted soil specimens are more homogeneous, have less defects and voids. As a result, it provides uniform unconfined compressive strength and deformation results. The specimens were then soaked for 24 hours to obtain a uniform moisture content and compacted until they reached their maximum dry density at optimum moisture content.

A standard proctor's mold was used to compact the unconfined compression soil test specimens, which had a diameter of 38mm. However, specimen diameters of 50mm and 100mm were compacted in their respective specimen diameter molds. For each height of specimens as well as the diameter of specimens, trials were used to determine the number of layers and blows per layer.

Figure 3-3 Preparation of Compacting Cohesive Soil Specimen for Unconfined Compression Test

The compacted specimens were safely extruded from the lubricated Shelby tubes and molds using a hydraulic extruder, and their height was trimmed in accordance with their height to diameter ratio, as shown in Table 3-1. The specimens' density and moisture content were measured, and the differences between their optimum moisture content and maximum dry density ranged from -0.56 percent to 0.75 percent and -0.02 g/cm^3 to 0.02 g/cm^3, respectively. However, the difference was negligible (Wang et al., 2020).

Despite (Güneyli & Rüşen, 2016; Merga, 2016; Yilmaz et al., 2019), the strain rate used in this study was 1 percent strain per minute for all specimen heights, as stated in the Japanese Geotechnical standard (Japanese Geotechnical Society, 2016; Verveckaite et al., 2007), to reduce the effect of strain rate on the unconfined compression test result.

Table 3-1 Specimens' diameter, height and stain rate at 1%/minute with respect to their height to diameter ratio (H/D)

H/D Ratio	Specimen's Diameter					
	38mm		50mm		100mm	
	Height (mm)	Strain mm/minute	Height (mm)	Strain mm/minute	Height (mm)	Strain, mm/minute
3.00	114.0	1.14	150.0	1.50	300.0	3.00
2.75	104.5	1.05	137.5	1.38	275.0	2.75
2.50	95.0	0.95	125.0	1.25	250.0	2.50
2.25	85.5	0.86	112.5	1.13	225.0	2.25
2.00	76.0	0.76	100.0	1.00	200.0	2.00
1.75	66.5	0.67	87.5	0.88	175.0	1.75
1.50	57.0	0.57	75.0	0.75	150.0	1.50
1.25	47.5	0.48	62.5	0.63	125.0	1.25
1.00	38.0	0.38	50.0	0.50	100.0	1.00

As shown in Table 3-1, Figure 3-4 below; the number of H/D ratio of the specimen taken at 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, 2.50, 2.75 and 3.00 for each 38mm, 50mm and 100mm specimen's diameter. The height of the specimens for each specimens' diameter were from 38mm to 114mm, 50mm to 150mm and 100mm to 300mm respectively.

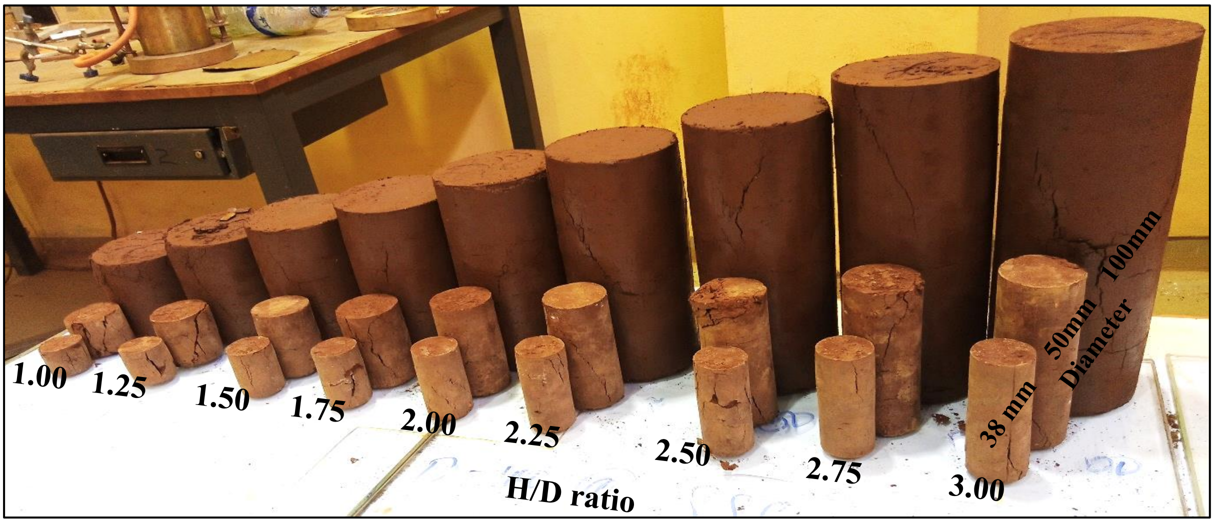

Figure 3-4 Compressed specimens of 38mm, 50mm and 100mm diameter with respect of their H/D ratio for Saris Sefer site

The unconfined compressive strength of the sample specimen is determined from the graph of stress strain curve in which the axial stress value of either the peak value or the value of at 15% axial strain (ASTM, 2007b; Kalinski, 2011). Accordingly, the UCS values of cohesive soil specimens were taken from the five test pits and nine height to diameter ratio with respect to their specimen diameters is summarized in Figure 4-5 in chapter four.

3.6. Data Processing and Analysis

3.6.1. Assuring of Collected of Laboratory Test Data

The dry density and moisture content of axially compressed soil specimen using an unconfined compression test machine which was prepared at different height to diameter ratio was checked with the optimum moisture content and maximum dry density. The difference before and after the test of the soil specimen’s dry density, water content, diameter of the specimen and height of the specimen were the same or a negligible variance. Then the determined UCS value with the corresponding of its H/D ratio was took for the data analysis. But for the tests which had a large gap among those control points, another trial was made till the difference had been minimized.

3.6.2. Data Analyzing Methods

The laboratory test results of grain size distributions, Atterberge limits, maximum dry densities, optimum moisture contents, peak unconfined compressive strengths, and axial strains were analyzed using the Excel software. The behavior of the stress-strain curve was assessed from a graph of axial stress verses percentage of axial strains which were plotted using OriginPro-8.5 software. The characteristics of failure patterns for the different specimens’ height and diameter soil specimens were determined from the photos that were taken during the unconfined compression tests.

3.6.3. Validation and Recommending of Appropriate Height to Diameter Ratio

The UCS value of compacted cohesive soil specimens, failure patterns, and stress strain curves of the various specimens' diameter with their respective H/D ratio were assessed, and an optimum aspect ratio was determined as follows.

The recommendation of an appropriate size of specimens was evaluated according to uniformity peak UCS values among the series H/D ratio of 38mm, 50mm, and 100mm diameters. When the aspect ratio of specimens increases, peak UCS value difference between the successive aspect ratio decreases and becomes more stable (Al-Rkaby & Alafandi, 2015; Wang et al., 2020). However, the difference in strength value between successive specimen sizes is large when the aspect ratio of the specimens is too small or too large. Therefore, the appropriate height to diameter ratio was selected according to the UCS uniformity.

The failure mechanism systems of the appropriate specimen size of the unconfined compression test specimen are clear, distinct, not intersecting inclined failure planes within the entire length of sample specimens (Yilmaz et al., 2019). But the failure patterns vary with the height to diameter ratios of the test sample specimens. For the small height to diameter ratio, the whole sample is restrained by the friction of the end-loading plates (Moores & Hoover, 1966). Consequently, the bulging failure mechanism occurred (Verveckaite et al., 2007). When increasing the aspect ratio, stress distribution inside the sample will become more uniform (Wang et al., 2020). Therefore, the failure mechanism will be changed. The buckling effect is also the other issue if the height to diameter ratio increases due to the slenderness effect (Moores & Hoover, 1966). As a result, complex failure mechanisms will happen (Güneyli & Rüşen, 2016). Accordingly, the failure mechanism of the specimens was assessed to determine the appropriate height to diameter ratio of UCS values.

Then according to failure mechanism, UCS value uniformity, the characteristics of stress-strain graph, and axial strains of the peaks UCS values of the H/D ratios for each diameter were evaluated and validated with previous literatures done in similar soil types.

3.6.4. Statistical Analysis for Predicting UCS Value

To quantitatively evaluate the effect of height to diameter ratio on the unconfined compressive strength of Jimma city’s cohesive soil, a single regression and correlation analysis was applied to obtain the relationships among the ratio of UCS value of optimum H/D ratio to the UCS measured values with the H/D ratios of the specimen’s diameter.

For this study, since the variables are single, single regression analysis was applied (Mohamed, 2015), using OriginPro-8.5 software. The analyses with high coefficients of determinations were selected. UCS equations are derived from the selected H/D ratio for specimens’ sample diameters, which is applicable to the study area.

3.7. Ethical Consideration

The data were collected after the ethical permission garneted from Civil and Environmental Engineering Department and all concerning bodies before proceeding the study acceptance should be given from local authorities. The purpose of the study was clearly described to the organizations and to the concerning local communities. The permission of Jimma University, Jimma Institute of Technology contact in order to precede the research.

3.8. Data Quality Assurance

The quality of the data was assured by repeating the samples by using standard operating procedures. To check the accuracy and validity of data instrument, first of all, calibration and verification were checked. Laboratory test manuals was prepared in order to avoid the error of data.

CHAPTER FOUR

4. RESULTS AND DISCUSSION

4.1. Classifications and Compaction of Soil Specimens

4.1.1. Grain Size Analyses Test Result

From the grain size analysis tests, Table 4-1 indicates the classification of particle grain size soil according to the Unified Soil Classification System (USCS). According to the test results, fine grained soil made up more than half of the particle size of the collected specimen. Even though the Aweytu test pit sample had relatively a large amount of sandy soil, all sample specimens were categorized as fine-grained soil in general.

Table 4-1 Grain size analyses test result

Site Location	Percentage of Course Soil (>4.75 mm)	Percentage of Sandy Soil (4.75mm-0.075mm)	Percentage of Fine Soil (<0.075mm)
Saris Sefer	0.00	1.59	98.41
Mariam Church	0.21	1.64	98.15
JiT campus	0.00	2.66	97.34
Ajip	0.02	16.7	83.28
Aweytu	10.66	33.61	55.74

4.1.2. Atterberge Limit Test Result

Figure 4-1 shows a flow of curves which plotted moisture content against the number of blows on a logarithmic scale. According to the flow curves, the Aweytu test pit had low plasticity, while the remaining test pits had high plasticity soils. As shown in Table 4-1, the test pits with high percentage of fine particles have a high liquid limit plasticity. While the test pit, which has low percentage of fine particles relative to others, has a low liquid limit value.

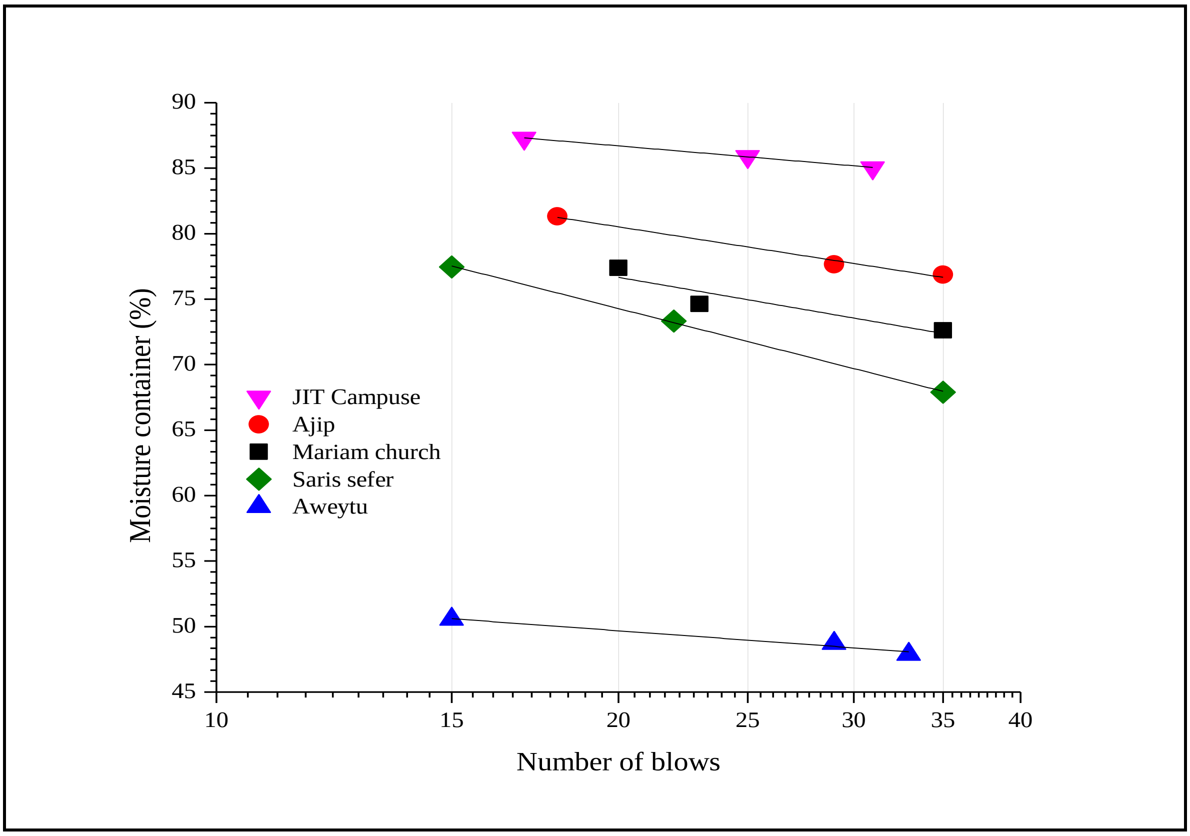

Figure 4-1 Flow charts cohesive soils in Jimma city

4.1.3. Compaction Test

Figure 4-2 shows the relationship between dry density and moisture content. Then the maximum dry density and optimum moisture content of the collected cohesive soil determined from the plotted curve. Accordingly, the specimens' maximum dry densities ranged from 1.27 g/cm^3 to 1.51 g/cm^3. The optimum moisture content was varied from 23% up to 39% as shown in Table 4-2.

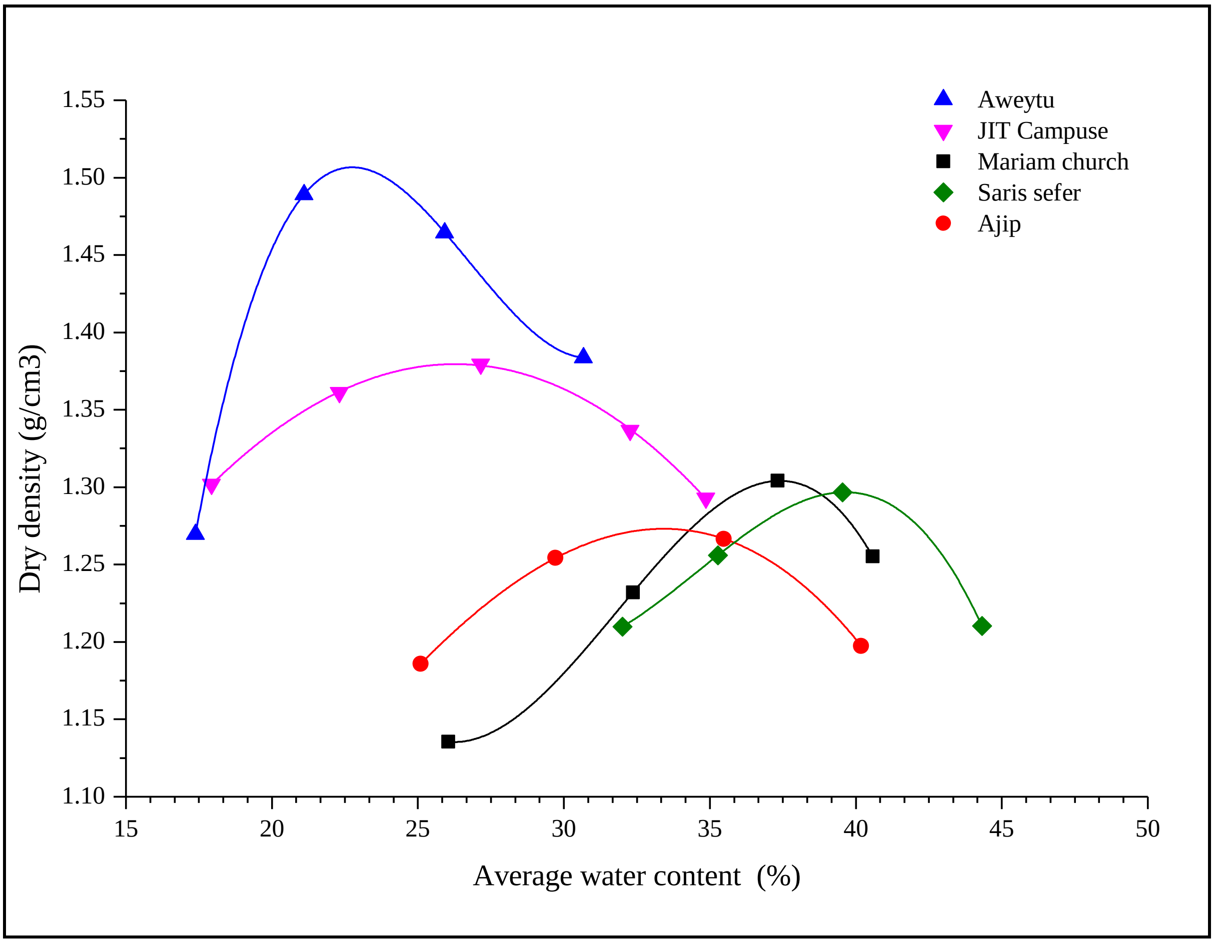

Figure 4-2 Graph of standard Proctor compaction curves

4.1.4. Classification Soil Specimens

Figure 4-3 shows the plasticity chart of the collected sample specimen according to the Unified Soil Classification System (USCS). According to the chart, the soil specimens from Aweytu test pit was low plasticity clay soil (CL) while, Ajip and JIT campus test pits soil specimen were high plasticity clay soil (CH). The Mariam Church and Saris Sefer test pits, on the other hand, were high plasticity silt soils (MH).

Figure 4-4 shows plastic index verses liquid limit chart of American Association Society of Highway and Transport Organization (AASHTO) soil classification system. As a result, the graph illustrated all test pits classified as A-7 Clayey soil group. So, while Aweytu and JiT campus test pits were classified as A-7-6; Ajib, Mariam Church, and Saris Sefer test pits were classified as A-7-5.

Accordingly, based on USCS and AASHTO classification system all test pits were fine grained soil either low or high plasticity consistency limit.

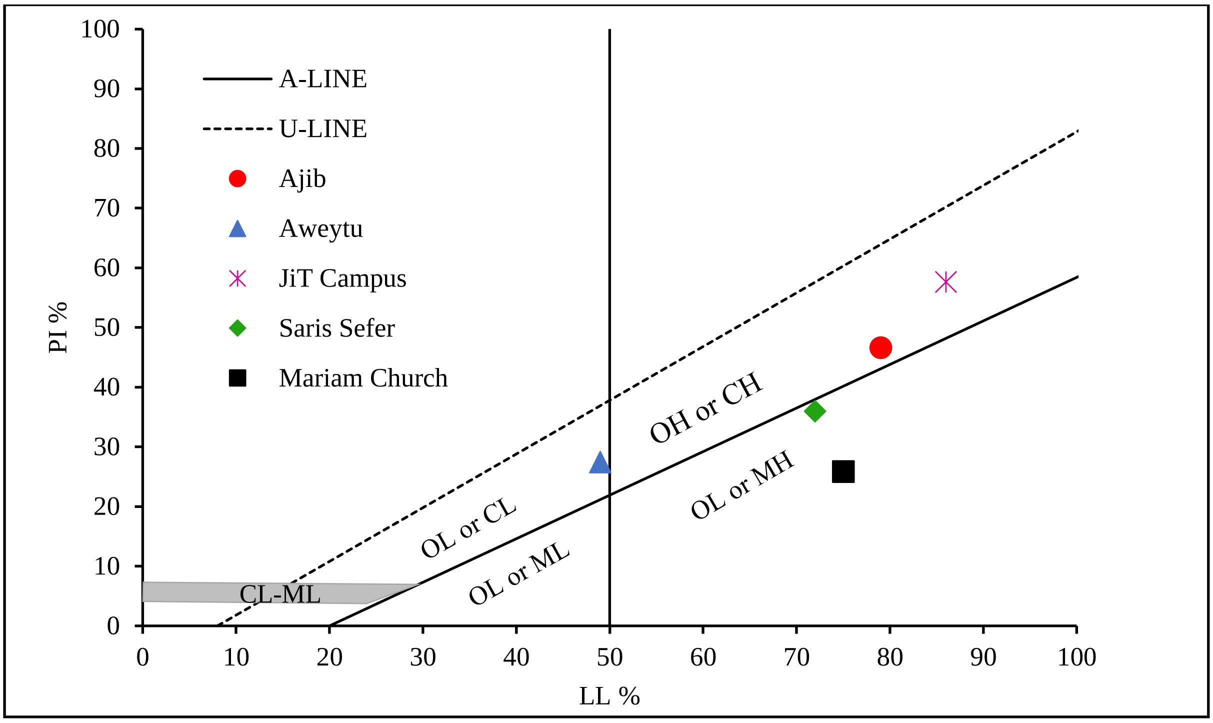

Figure 4-3 Plasticity Charts Cohesive Soil of Jimma City According to USCS

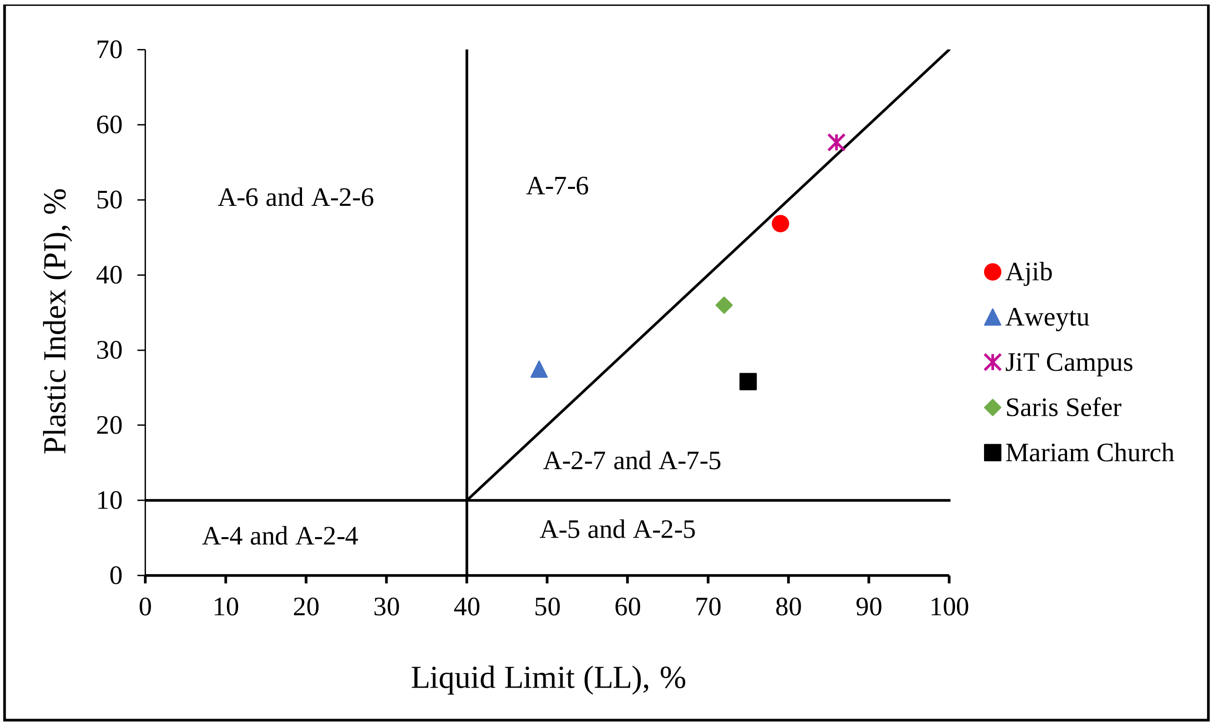

Figure 4-4 Plasticity Charts Cohesive Soil of Jimma City According to AASHTO

Generally, the index properties of the soils including the classification according to AASHTO soil classification and USCS soil classification standard summarized in Table 4-2 below.

Table 4-2 Summary of index properties, classifications, and compaction parameters of the soils tested

Parameters	Pit Sites				
	Ajib	Aweytu	Saris Sefer	JiT campus	Mariam church
Liquid limit, %	79	49	72	86	75
Plastic limit, %	32	22	36	28	49
Plastic Index, %	47	27	36	58	26
Percentage of Course Soil, %	0.0	10.7	0.0	0.0	0.2
Percentage of Sandy Soil, %	16.7	33.6	1.6	2.7	1.6
Percentage of Fine Soil, %	83.3	55.7	98.4	97.3	98.1
Soil Classification					
USCS	CH	CL	MH	CH	MH
Group Name	High Plasticity Clay Soil	Low Plasticity Clay Soil	High Plasticity Silty Soil	High Plasticity Clay Soil	High Plasticity Silty Soil
AASHTO	A-7-5	A-7-6	A-7-5	A-7-6	A-7-5
Group Name	Clayey soil	Clayey soil	Clayey soil	Clayey soil	Clayey soil
Standard Proctor Compaction Test					
Optimum Moisture Content, %	34	23	26.5	39	37.5
Maximum Dry Density, g/cm^3	1.27	1.51	1.38	1.3	1.31

4.2. Unconfined Compressive Strength Tests

4.2.1. Peak Unconfined Compression Tests and Specimens' Size

Figure 4-5 shows the effect of height to diameter ratio on unconfined compression strength test varying both height and diameter of soil specimen. As a result of the findings, the unconfined compression strength of soil decreases as the height and diameter of the soil specimen increase. The effects of the H/D Ratio (height) and specimen diameter on the peak unconfined compressive strength value of cohesive soils observed in this study are discussed separately as follows:

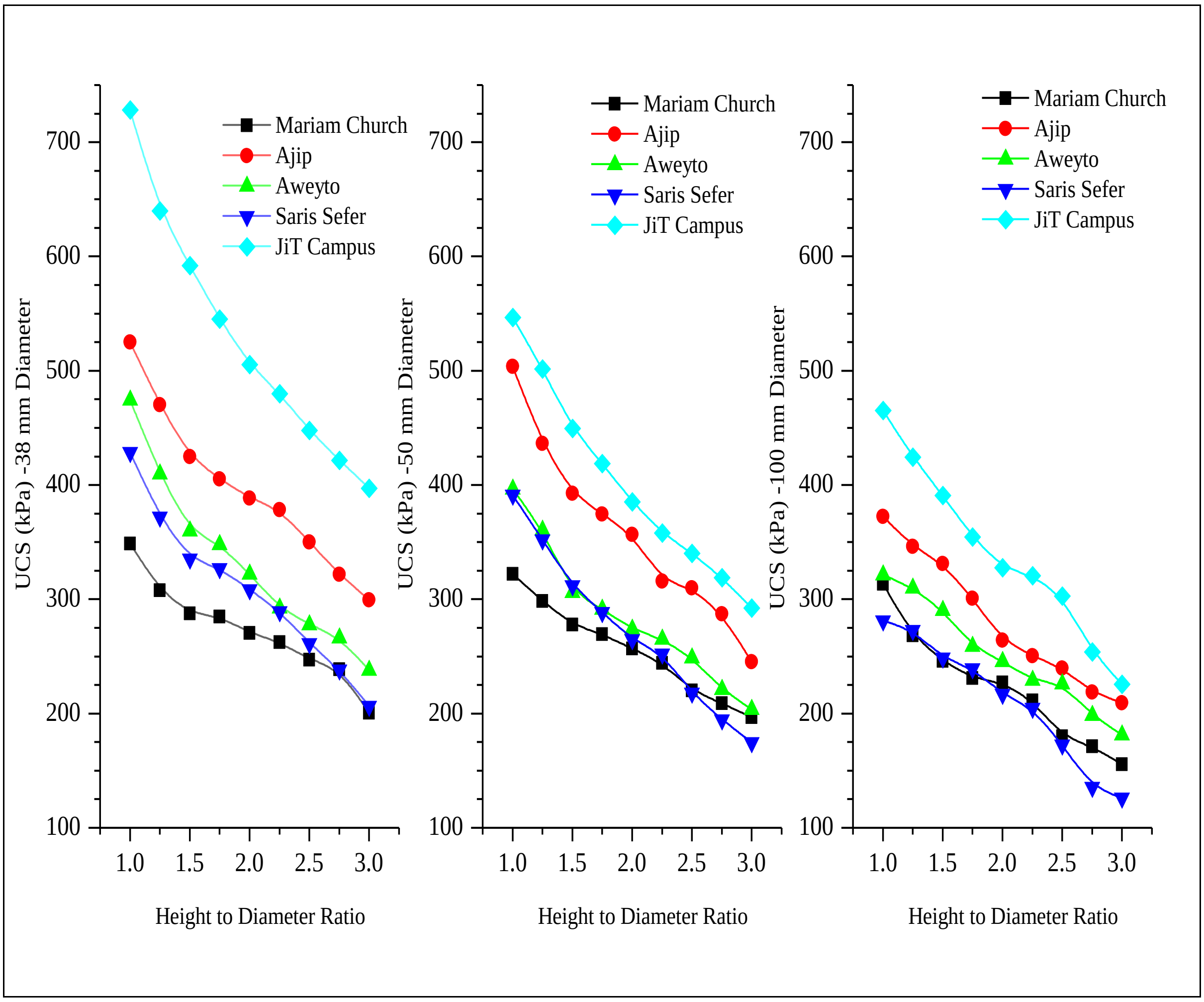

Figure 4-5 Peak value of unconfined compressive strength 38mm, 50mm, 100mm specimen diameter with their respective H/D ratio

I. Effect of H/D Ratio (specimen height) on Unconfined Compressive Strength

The variation in UCS value of the cohesive soil specimen between successive H/D ratios dropped considerably from 1.00 to 1.75 and 2.5 to 3 H/D ratios. The difference in UCS value of the specimens from an H/D ratio of 1.75-2.25, on the other hand, was very small. As seen in Figure 4-5, this effect does not change as the specimen diameter varies, which is similar to what (Güneyli & Rüşen, 2016) found.

Figure 4-6 shows the relation between the percentage difference of UCS from mean value and the H/D ratio for each specimens’ diameter. At 2.00 H/D ratio in all specimens' diameter, the difference between the peak UCS value and the mean value of consecutive H/D ratios was small. However, the difference in mean value of UCS was increased as the H/D ratio away from 2.00 H/D ratio to both 1 and 3 H/D ratios.

The polynomial fit curve trend lines in Figure 4-6 showed that, for all test pits except Aweytu's 38mm specimen diameter, the closest UCS of the specimen to the mean value was at 2 H/D ratio. The nearest UCS value for this test pit was 1.75 H/D ratio, but the difference in value was insignificant. (Wang et al., 2020) also discussed this property, and the result was the same.

According to laboratory test results, the UCS values of between 1.75 and 2.25 H/D ratios were stable. This is because the H/D ratio decreases both the end platen and the slenderness (buckling) effect. As a result, the test results of those H/D ratios came close to the UCS mean value for cohesive soils measured at various specimen sizes.

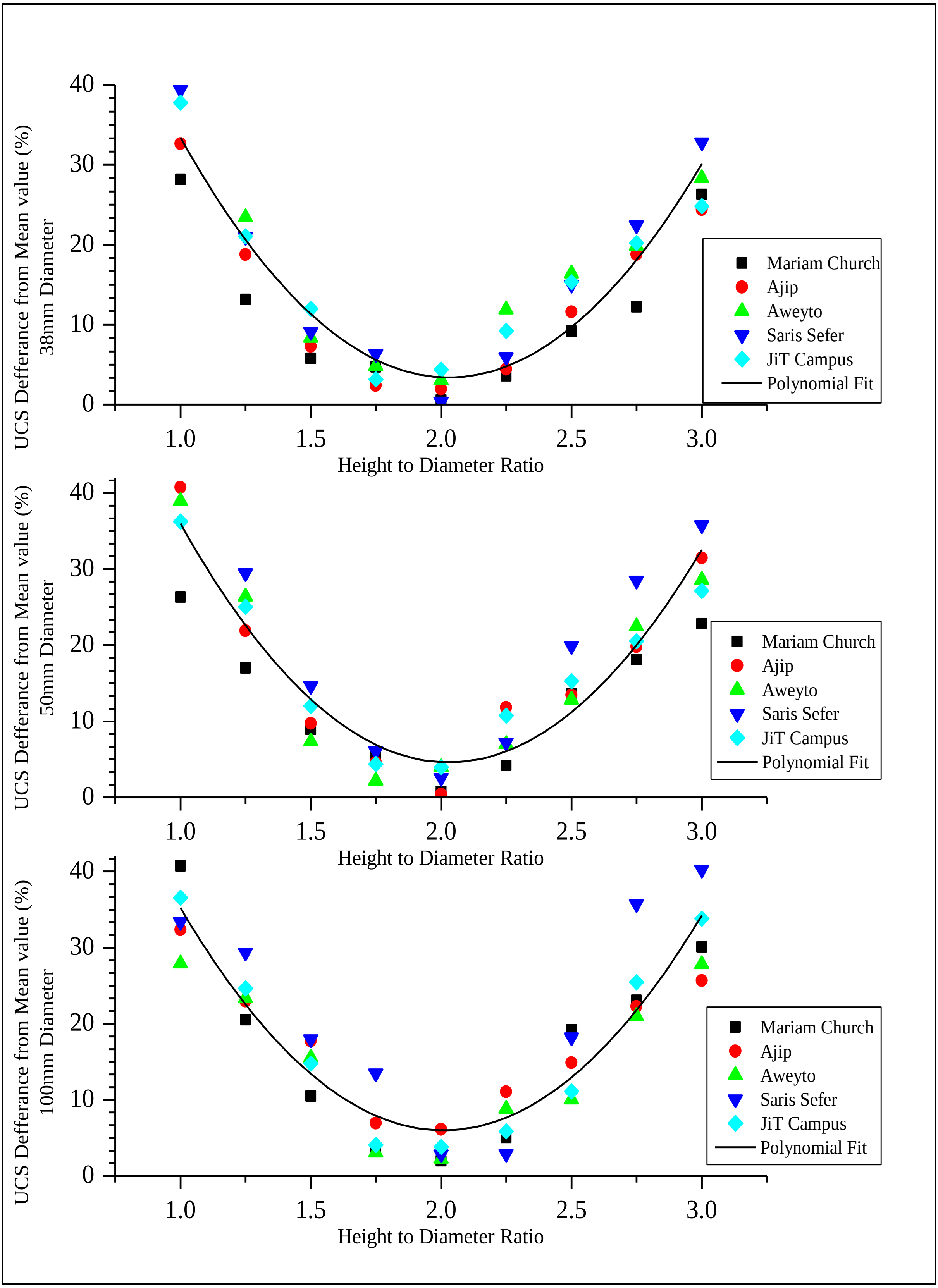

Figure 4-6 Percentage of UCS difference from mean value for 38mm, 50mm and 100mm specimen's diameter

II. Effect of Specimen's Diameter on Unconfined Compressive Strength

As the diameter of the specimen became wider, the peak value of UCS for each successive H/D ration was decreased. From a 38mm specimen diameter, the UCS value of specimens with diameters of 50mm and 100mm decreased by 11.4 percent to 17.2 percent and 24.6 percent to 34.2 percent, respectively, with each H/D ratio increment. On the other hand, the 100mm diameter specimen was reduced by 13.3 percent to 20.5 percent from the 50mm specimen diameter.

This difference of UCS of the cohesive soils from the smallest specimen diameter to the largest specimen diameter was relatively smaller than that mentioned by (Chew & Bharati, 2011) and (Dirg et al., 2019). The effect of specimen size was investigated in both studies using a triaxial test. Since the specimen's confining pressure in the triaxial test has an impact, the difference in UCS value investigated in this study was lower than in those researches.

Generally, as the H/D ratio increased, the reduction in UCS value was unaffected by increasing specimen diameter. The UCS value of the specimens with diameters of 38mm, 50mm, and 100mm decreased as the H/D ratio increased, as shown in Figure 4-5. Similarly, in all specimen diameters, the closest UCS value to the mean value was at 2 H/D ratio, followed by 1.75 and 2.25 H/D ratios.

In this study it is also observed that; the unconfined compressive strength value of compacted cohesive soil specimen in Jimma Town has low strength comparing to the previous studied such that (Güneyli & Rüşen, 2016; Wang et al., 2020). This could be due to the loading strain rate and the maximum dry density of all samples that were taken from the study area had lower than the earlier studies.

4.2.2. Stress Strain Curve

Stress-strain curve of unconfined compressive strength for soil specimen is a plot of graph axial stress verses axial strains in percent. The stress stain curve of specimens in all test pit sites was flat, with high axial strain exhibiting ductile behavior at low H/D ratios. However, with increasing the H/D ratio of the specimens, the property of the stress-strain curve changed to brittle behavior.

Figure 4-7 show the relation of stress strain of the unconfined compression test for cohesive soil JIT campus. These graphs demonstrates that the chrematistics of stress-strain curve in both with increasing of height to diameter ratio (height) and diameters of the cohesive soil specimens. Hence, the effect of height to diameter ratios and diameters of the specimens described as follows:

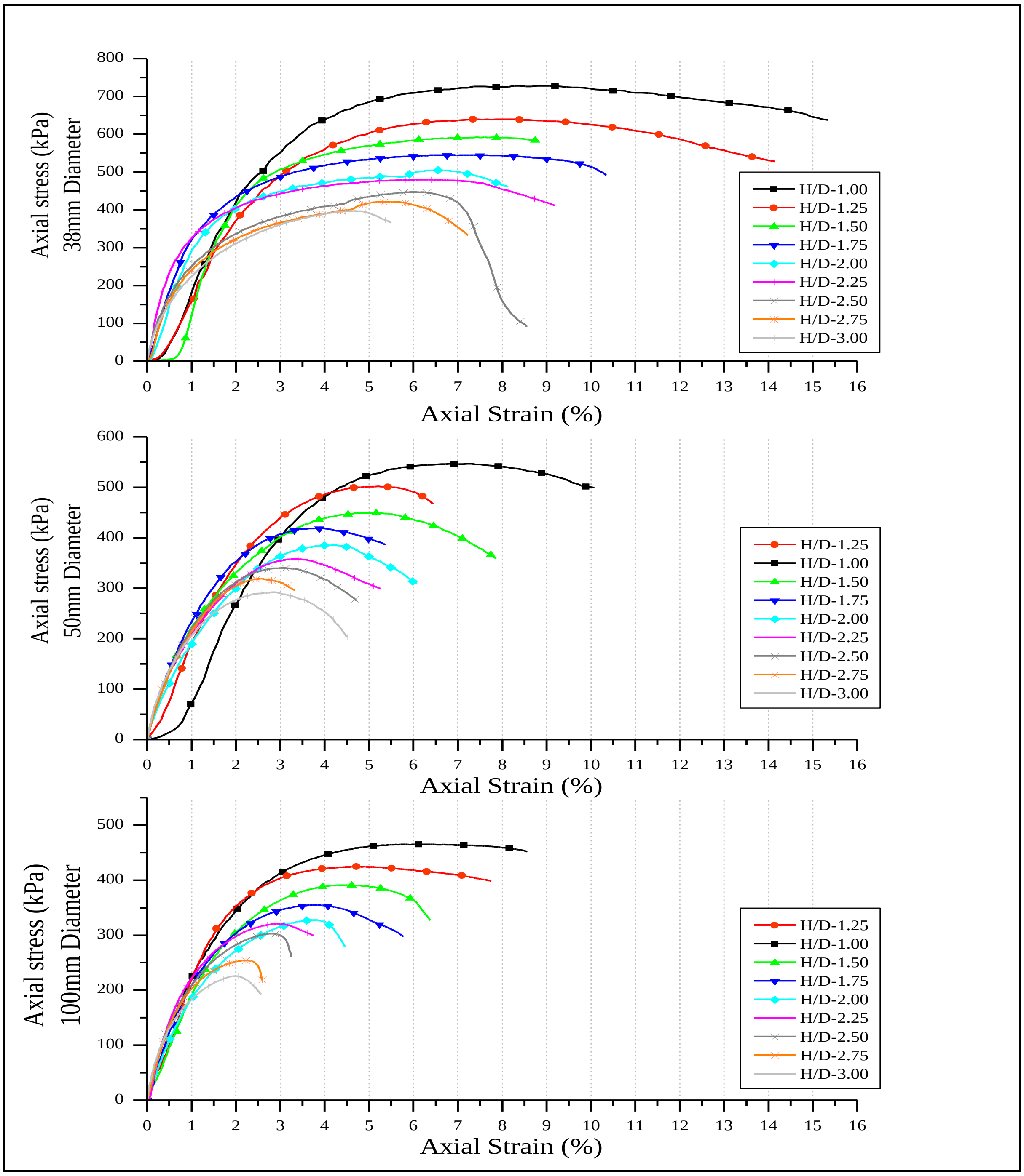

Figure 4-7 Stress-Strain Curve of JiT Campus's Cohesive Soil Specimen

I. Effect of Specimen's H/D Ratio (height) on Stress-Strain Curve

In this study, the stress strain curve of the specimens absorbed that the peak UCS value attained large axial strain in the smallest H/D ratio. But, the small H/D ratio specimen's stress strain curve experiences small axial stresses at the early axial strains. Consequently, the curves made gentle line. As the axial strain increases, then axial stresses speedily increase up to UCS peak value. As a result, the curve was changed from a gentle to a steep upward curve. After the axial stress has reached the peak value, the stress continued the same. The result was no more decreased and the curve was continued horizontally or slightly tiled downward as shown in Figure 4-7 for both 38mm and 50mm specimen diameter. This was since the height of the specimen is too short, the whole specimen is restrained by the friction of the end loading plates (Moores & Hoover, 1966).

For H/D ratios starting at 1.50 for 50mm and 1.75 for 38mm specimen diameters, the stress strain curve becomes slightly linear with relatively small slopes at the initial axial strains until reaching the peak UCS value. As the H/D ratio increased, the curve rapidly dropped after reached the peak UCS value. Specifically, from the 2.50 H/D specimen's ratio, the stress strain curve was slightly linear but lower slope till to reach the peak strength and the curve became sharp.

The percentage strains of the peak UCS value decreased as the specimen's H/D ratio increased. For 38mm diameter, the strain of peak UCS value was 9 percent to 7 percent, 6.5 percent to 6 percent, and 5.5 percent to 4.4 percent for 1.00-1.50, 1.75-2.25, and 2.5-3.00 H/D ratios, respectively. For 50mm diameter, axial strains of peak UCS were found to be 6 percent to 4 percent, 4 percent to 3 percent, and 3 percent to 2 percent for 1.00-1.25, 1.50-2.00, and 2.25-3.00 H/D specimen ratios, respectively. The strains of peak UCS value for the 100mm specimen's diameter was also 4.7 percent, 4 percent -3 percent, and 3 percent -1.8 percent for the H/D ratios of 1.00, 1.25-2.00, and 2.25-3.00, respectively.

Thus, starting from 1.5 H/D ratios and above, the effect of the end plate was reduced by increasing of the specimen height. But the axial stresses were rapidly dropped starting from 2.25 to 2.50 H/D ratios due to the increasing of local defects and buckling effects. This observation is the similar with the previous studies (Güneyli & Rüşen, 2016; Moores & Hoover, 1966; Wang et al., 2020).

II. Effect of Specimen's Diameter on the Stress Strain Curve

Figure 4-7 show, the effect of specimens' H/D ratio with respect to their specimens' diameter on the stress strain curve. Those graphs show, as the specimens' size increased, the axial strains of the peak UCS values decreased while the sharpness of the curves were increased. However, 50mm and 100mm specimen diameter remains relatively minor difference compering to 38mm diameter of specimen.

The consistency of the stress strain curves in the successive specimen's H/D ratio was increased as the specimen's diameter increased. For instance, the 38mm diameter specimen peak UCS strain varied from 9% to 3%, while 100mm specimen diameter varied from 4.7% to1.8% axial strain on average. The post peak UCS stress strain curve of 38mm specimen's diameter was a smooth curve from 1.00 to 1.75 H/D ratio. While it was a sharp curve from 2.5 and above H/D ratio. The stress strain curves started from 2 H/D ratio for diameters of 50mm and 100mm and were sharp curves. In this study, both ductile and brittle behavior governed the stress strain curve of 38mm diameter specimens, but brittle behavior governed the 50mm and 100mm specimen diameters. Accordingly, the effect of the end plate was decreased on both the increase of the specimen's diameter and H/D ratio.

4.2.3. Failure Pattern and Size of specimens

Figure 4-8, Figure 4-9, Figure 4-10 and Figure 4-11 shows the failure pattern of 38mm, 50mm and 100mm specimens' diameter during the unconfined compression test which were observed on nine series of H/D ratios respectively. The types of failure patterns are also mentioned for those specimens' diameter in Table 4-3, Table 4-4 and Table 4-5 respectively. Then the effect of specimens' size and failure patter of unconfined compression test results is discussed with respect to specimens' height (H/D ratio) and diameter below.

I. Effect of Specimen's H/D Ratio (Height) on Failure Pattern

A. Failure Pattern of 38mm Diameter Specimen

The failure pattern of the axially compressed cohesive soil specimen varied as the specimen's H/D ratio increased. For H/D ratios ranging from 1.00 to 1.50, the failure plane was vertical or nearly vertical, which is extended from end to end of the entire length of the specimen. This is categorized as splitting tensile failure. But the crack length and slope of inclination reduced as the H/D ratio increased from 1.75-2.25. The failure cruck of those H/D ratios for high plasticity clay soil specimen (CH), i.e., Ajip and JIT Campus test pits were extended from nearly the top of the specimen to the near bottom of the specimen classified as shear failure pattern. For test pits Aweytu low plasticity clay soil (CL), Mariam church and Saris Sefer high plasticity slit soil (MH), the failure cruck was started from the top the specimen and extended to the bottom of the specimen. But it did not intersect the lower edge of the specimen. Accordingly, the failure pattern was both tensile shear and shear failure.

The failure cruck of 2.50 to 3.00 H/D ratios were local failures. This primarily occurs at the top or middle of the specimens. The failure pattern of those H/D ratios were a mix of local splitting tensile, local tensile shear, local shear and buckling. As mentioned in previous studies (Güneyli & Rüşen, 2016; Wang et al., 2020), the failure pattern was quite complex in this range of H/D ratios.

According to the deformation characteristic in this study, from 1.00-2.00 H/D ratio the deformation was observed throughout the entire length. However, the deformation of the specimens under unconfined compression test of H/D ratio greater than 2.00 was locally deformed.

The specimens were bulged laterally to an H/D ratio of 1.75. The bulging length, on the other hand, decreased as the diameter of the specimens and their H/D ratios increased. Even though it was clearly observed for the specimens that were high plastic clay soil specimens, this property was not the same for all test pits.

Figure 4-8 Failure pattern of 38mm specimen diameter, a- Ajip, b- Aweytu, c- JiT Campus, d- Saris Sefer and e- Mariam Church

Table 4-3 Failure pattern of 38mm diameter with its respective H/D ratio

Specimens H/D Ratio	Test Pit Sites				
	Ajip	Aweytu	JiT Campus	Saris Sefer	Mariam Church
1.00	Splitting Tensile	Splitting Tensile	Splitting Tensile	Splitting Tensile	Splitting Tensile
1.25	Double Splitting	Splitting Tensile	Splitting Tensile	Splitting Tensile	Splitting Tensile
1.50	Tensile Shear	Splitting Tensile	Splitting Tensile	Splitting Tensile	Splitting Tensile
1.75	Tensile Shear	Splitting Tensile	Shear	Splitting Tensile and Shear	Splitting Tensile and Shear
2.00	Shear	Tensile Shear	Shear	Tensile Shear	Shear
2.25	Local Crossing Shear	Shear	Local Malty Shear And Tensile Shear	Tensile Shear	Shear
2.50	Local Tensile and Shear	Local Shear	Local Crossing Tensile Shear and Shear	Local Tensile and Shear	Local Crushing
2.75	Local Tensile and Shear	Local Tensile	Tensile, Tensile Shear And Shear	Local Shear	Local Splitting And Local Shear
3.00	Local Splitting Tensile	Splitting Tensile and Shear	Tensile, Shear and Buckling	Splitting Tensile, Shear and Buckling	Axially Splitting Tensile

B. Failure Pattern of 50mm Diameter Specimen

The tensile shear failure of the 50mm diameter specimens started from the 1.50 H/D ratio. The tensile shear failure changed to shear failure for test pit of Saris Sefer and Mariam Church soil specimens from 1.75 to 2.50; for Ajib and JIT Campus from 1.75 to 2.25 H/D ratio; and for Aweytu soil specimen from 1.50 to 2.00 H/D ratio. Subsequently, the failure patterns changed to local shear, local tensile, and or mixed failure as the H/D ratio of the specimen increased. However, the failure pattern changed to local failures, the complexity and combination of failures reduced with comparison to the 38mm diameter specimen. Generally, the failure pattern of each H/D ratio of 50mm diameter specimen is summarized in Table 4-4 below.

The bulging failure behavior was decreased for the 50mm specimen diameter. The specimens were deformed throughout their entire length from 1.00 to 2.50 H/D ratio for the low plasticity clay soil (CL) Aweytu test pit and high plasticity slit soil (MH) Saris Sefer and Mariam church test pits. Yet, the 38mm specimen diameter has deformed throughout their entire length up to 2.00 H/D ratio. However, for 2.75 and 3.00 H/D ratios, the specimens only deformed some portion of their height. But, the test pits from Ajip and JIT Campus were high plasticity clay soil (CH), the defamation was the same as with the specimens of 38mm diameter. While the bulging characteristic of the cohesive soil specimen with a 50mm diameter was decreased from 1.75 H/D ratios to 1.50 H/D ratio from the 38mm specimens' diameter as shown in Figure 4-9 below.

Figure 4-9 Failure pattern of 50mm specimen diameter, a- Ajip, b- Aweytu, c- JiT Campus, d- Saris Sefer and e- Mariam Church

Table 4-4 Failure pattern of 50mm diameter with its respective H/D ratio

Specimens H/D Ratio	Test Pit Sites				
	Ajip	Aweytu	JiT Campus	Saris Sefer	Mariam Church
1.00	Splitting Tensile	Splitting Tensile	Bulging	Splitting Tensile	Splitting Tensile
1.25	Splitting Tensile	Splitting Tensile	Bulging	Tensile Shear	Splitting Tensile
1.50	Tensile Shear	Tensile And Shear	Crushing	Tensile Shear	Splitting Tensile
1.75	Shear	Tensile And Shear	Shear	Shear	Shear
2.00	Shear	Tensile And Shear	Shear	Shear	Shear
2.25	Shear	Local Tensile	Shear	Shear	Shear
2.50	Tensile And Shear	Local Shear and Tensile	Local Shear	Shear	Shear
2.75	Local Shear	Local Crushing	Local Shear	Splitting Tensile and Surface Spalling	Local Splitting Tensile
3.00	Local Splitting Tensile	Local Shear	Local Shear	Local Shear and Tensile	Local Splitting Tensile and Shear

C. Failure Pattern of 100mm Diameter Specimen

Figure 4-10 and Figure 4-11 shows that, the failure patterns of 100mm diameter specimen. Except for the JIT Campus test pit, all failure patterns at 1.00 H/D ratio were splitting tensile. The failure pattern soil specimens of Ajip, Aweytu, and Saris Sefer test pits then changed splitting tensile to tensile shear failure at 1.25; shear failure from 1.50 to 2.25; and a mix of local failure patterns from 2.75 to 3.00 H/D ratio. While the failure pattern of the soil specimen from the Mariam Church test pit was a tensile failure at 1.25 H/D ratio; shear failure from 1.50 to 2.25 H/D ratio; both shear and splitting tensile failure from 2.50 to 3.00 H/D ratio. The soil specimen JIT Campus test pit was a bulging failure from 1.00 to 1.25; shear failure from1.50 to 2.25; and local shear failure from 2.50 to 3.00 H/D ratio. This difference may be due to the plasticity behavior and compaction uniformity of the soil specimen test pits. The detailed failure pattern of 100mm specimens’ diameter is summarized in Table 4-5 below.

The 100mm diameter soil specimens were deformed through the entire length up to 2.50 H/D ratio. But local failure was observed from 2.75 to 3.00 H/D ratio. Hence, the deformation of 2.75 and 3.00 H/D ratio was nonuniform except in all test pits except the soil specimen came from the Mariam Church test pit.

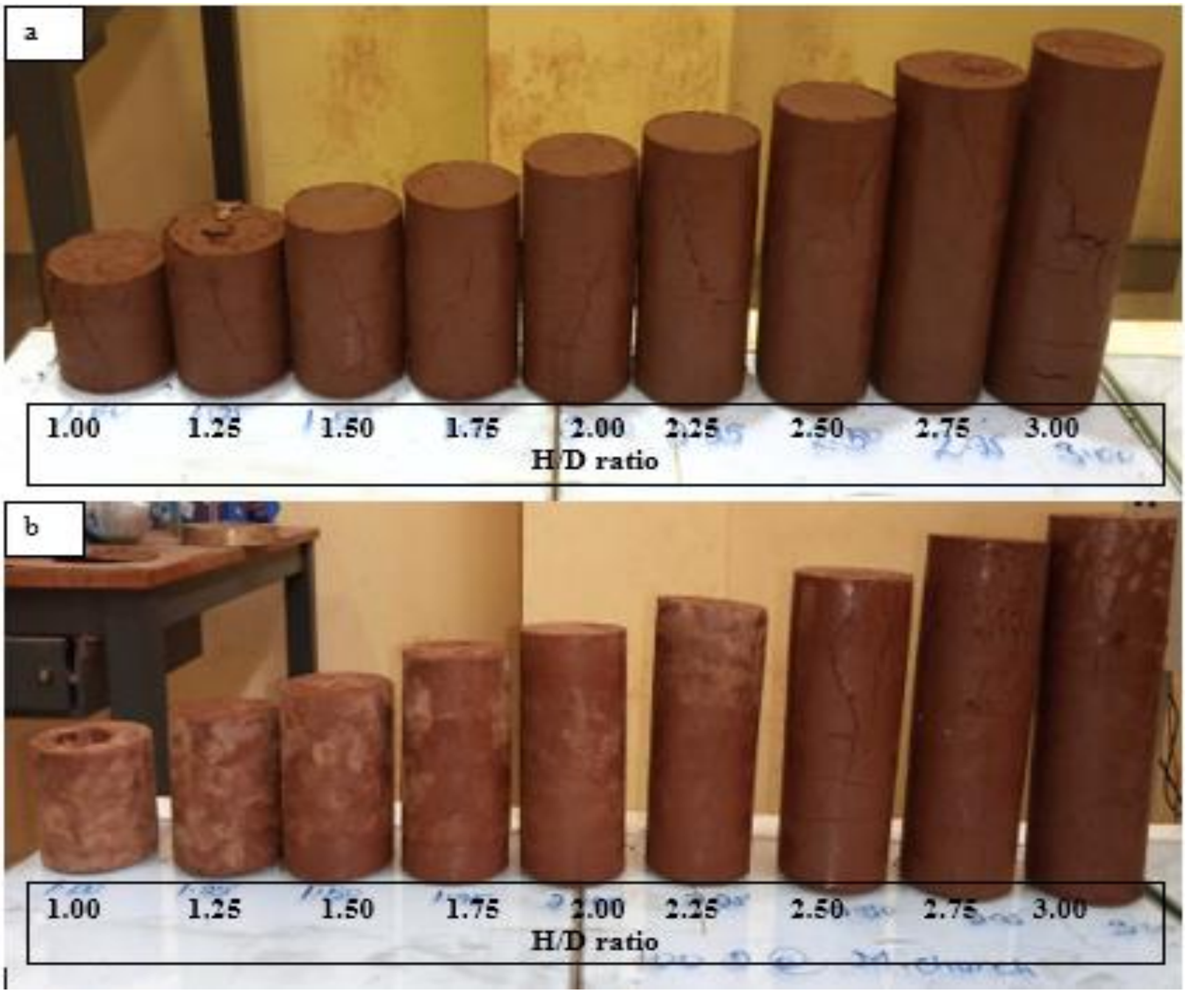

Figure 4-10 Failure pattern of 100mm specimen diameter, a- Saris Sefer and b- Mariam Church

Figure 4-11 Failure pattern of 100mm specimen diameter, a- Ajip, b- Aweytu, c- JiT Campus

Table 4-5 Failure pattern of 100mm diameter with its respective H/D ratio

Specimens H/D Ratio	Test Pit Sites				
	Ajip	Aweytu	JiT Campus	Saris Sefer	Mariam Church
1.00	Splitting Tensile	Splitting Tensile	Bulging	Splitting Tensile	Splitting Tensile
1.25	Tensile Shear	Splitting Tensile	Bulging	Tensile Shear	Tensile
1.50	Shear	Splitting Tensile	Shear	Shear	Shear
1.75	Shear	Shear	Shear	Shear	Shear
2.00	Shear	Shear	Shear	Shear	Shear
2.25	Shear	Shear	Shear	Shear	Shear
2.50	Shear	Shear	Local Shear	Shear	Shear And Splitting Tensile
2.75	Local Shear	Local Tensile	Local Shear	Shear And Splitting Tensile	Splitting Tensile and Shear
3.00	Local Shear and Surface Spalling	Local Crushed	Local Shear	Local Shear and Tensile	Splitting Tensile and Shear

II. Effect of Specimen's Diameter on Failure Pattern

The failure pattern of the 38mm diameter cohesive soil specimen in an unconfined compression test became more sensitive as the H/D ratio increased. The failure patterns changed with its aspect ratio. They deformed the whole length of the specimen up to a 2.00 H/D ratio. From the 2.25 H/D ratio, the failures were at some portion of the specimens and malty failure patterns as shown in Figure 4-8.

Figure 4-9, Figure 4-10, and Figure 4-11 shows that as the diameter of the specimens increases, so does the uniformity of the failure patterns. i.e., for 38mm diameter specimens, three test pits failed under shear failure at 2.00 H/D ratio and two test pits failed under tensile shear failure. However, four test pits of 50mm diameter specimens and all test pits of 100mm diameter specimens were under the shear failure category from 1.75 to 2.25 H/D ratio as shown in Table 4-3, Table 4-4 and Table 4-5.

Consequently, as the H/D ratio with respect to specimens' diameter increased, the end platen effect was significantly decreased. The bulging and splitting tensile failure changed to shear failure at 2.00 H/D ratio in all specimens' diameter. But the slenderness effect increased as the specimen's H/D ratio was raised to 2.25 H/D ratio for 38mm; 2.50 H/D ratio for 50mm and 100mm specimen's diameter. Thus, the failure patterns were local, complex, and non-uniform deformation. Accordingly, the UCS value was decreased and the stress-strain curve was a sharp curve as discussed in the previous sub-topic.

In general, the 50mm and 100mm diameters of the specimen gives more reliable and representative UCS test results than the 38mm specimen diameter, based on the uniformity of deformations, characteristics of failure patterns, and behavior of stress-strain curves observed in this study. Therefore, greater diameter of the specimen accurately representing the soil strengths in the fields as described in (Omar & Sadrekarimi, 2015).

The peak unconfined compression strength of cohesive soil tested at a H/D ratio of 2.00 was the most similar to the mean test result observed across all specimen diameters. This H/D ratio deformed in a consistent manner. In comparison to other specimens' H/D ratios, the stress strain curve from 1.75 to 2.25 H/D ratios was neither horizontal nor sharp. As a result, conducting an unconfined compression test at a height-to-diameter ratio of 2.00 yields a consistent and representative unconfined compressive strength of cohesive soil in specimens with diameters of 38mm, 50mm, and 100mm. This height to diameter ratio is also recommended by (Wang et al., 2020) for 50mm diameter sample specimens, and it is within the range of the (ASTM, 2007b) test standard.

4.3. Correction Factors for 2.00 H/D Ratio of Unconfined Compressive Strength Cohesive Soil

Correction formulas for unconfined compressive strength of clay soil were developed by (Güneyli & Rüşen, 2016) for length to diameter ratios of 2.00, 2.50, and 3.00 based on 48mm specimen diameter. In this study, the correction factor was set to the optimum height to diameter ratio (i.e., 2.00) for specimens with diameters of 38mm, 50mm, and 100mm. As a result, best fit line and least square regression were determined for cohesive soil specimens with low plasticity clay soil (Aweytu test pit), high plasticity clay soil (Ajip and JIT Campus test pits), and high plasticity silt soil (Saris Sefer and Mariam Church test pits) in all specimen diameters. Hence, all results show strong correlations between the peak UCS value and the cohesive soil specimens’ height to diameter ratio as shown in Figure 4-12.

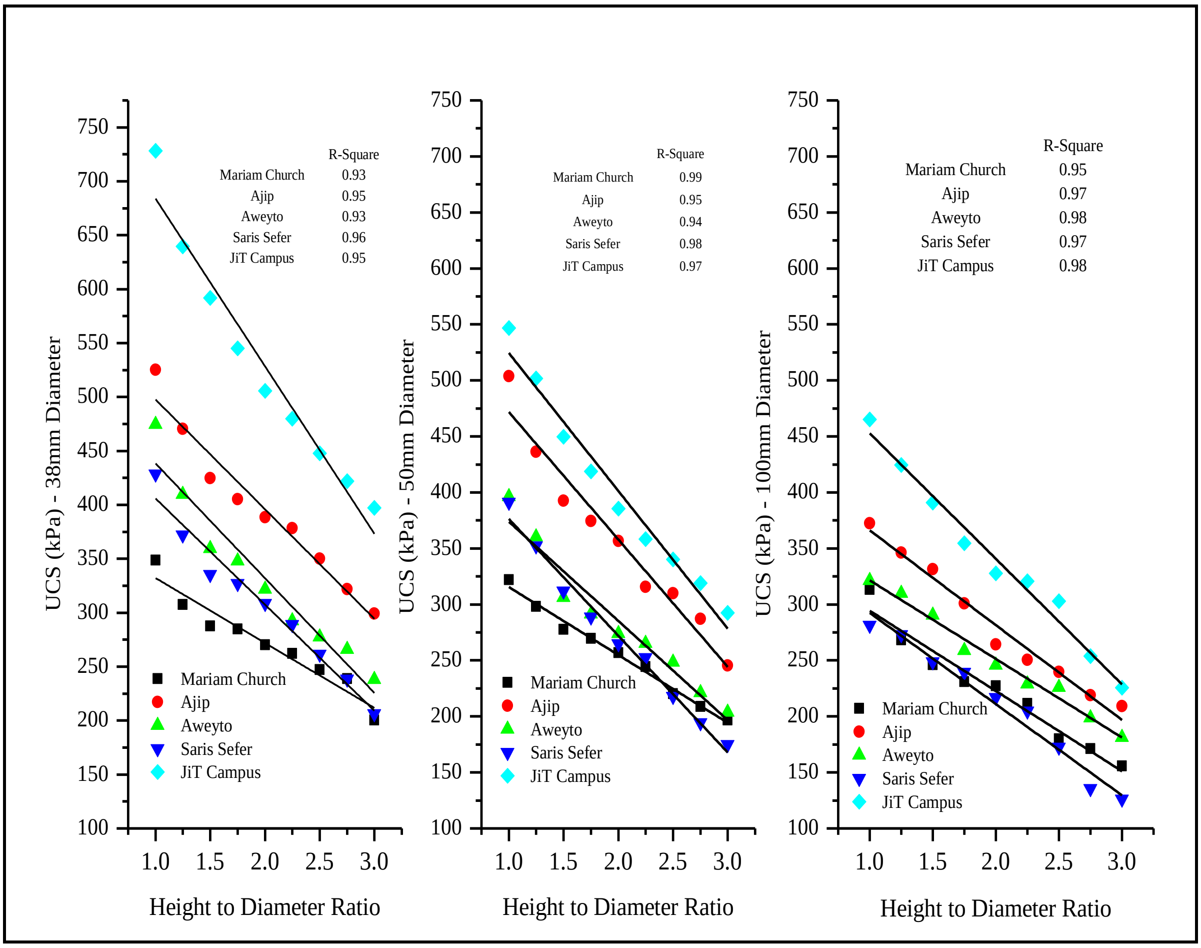

Figure 4-12 Least squares regression of peak UCS value verses height to diameter ratio at 38mm, 50mm and 100mm specimen diameter for all test pits.

As a result, for 38mm, 50mm, and 100mm specimen diameters, the ratio of peak unconfined compressive strength of different H/D ratios (i.e., from 1.00-3.00) to 2.00 H/D ratio ($\frac{UCS}{UCS_{2.00}}$) is plotted in Figure 4-13, Figure 4-14 and Figure 4-15 respectively. For those plotted graphs, linear regression lines were fitted to the resulting data for the cohesive soils which were collected from five test pits at different locations of Jimma Town.

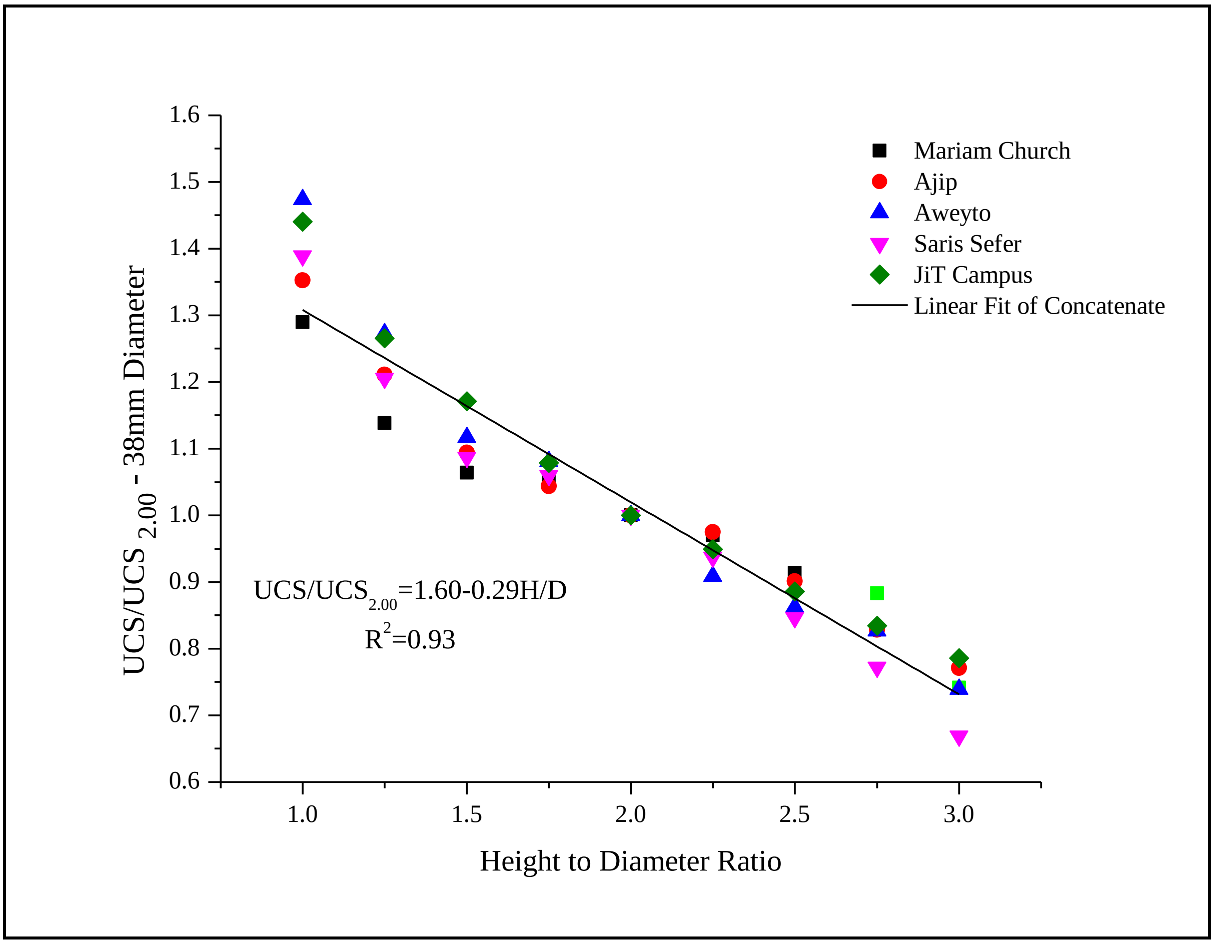

Figure 4-13 Plot of $\frac{UCS}{UCS_{2.00}}$ verses height to diameter ratio of 38mm specimen diameter

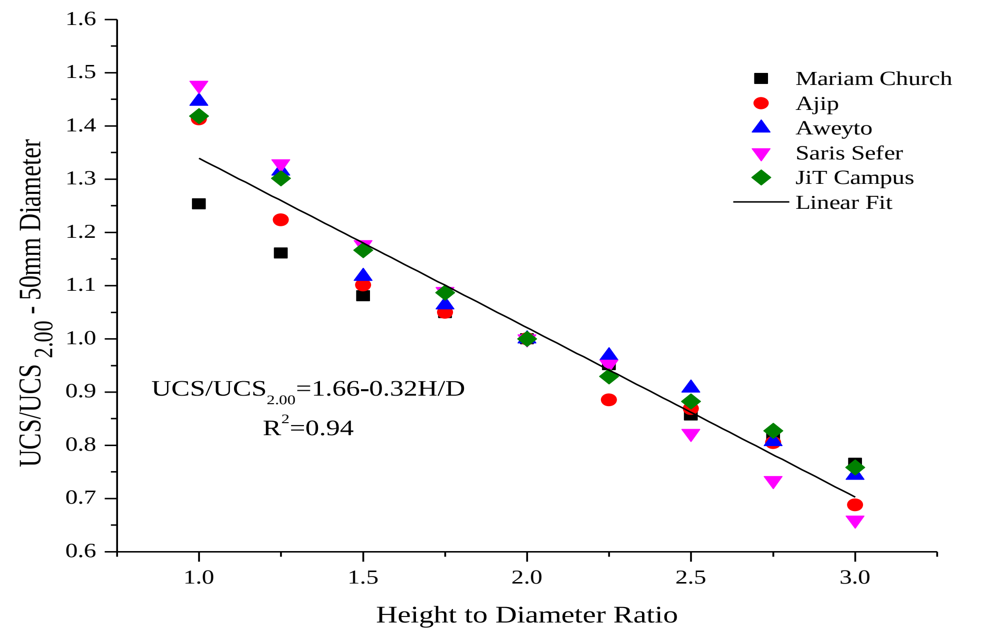

Figure 4-14 Plot of $\frac{UCS}{UCS_{2.00}}$ verses height to diameter ratio of 50mm specimen diameter

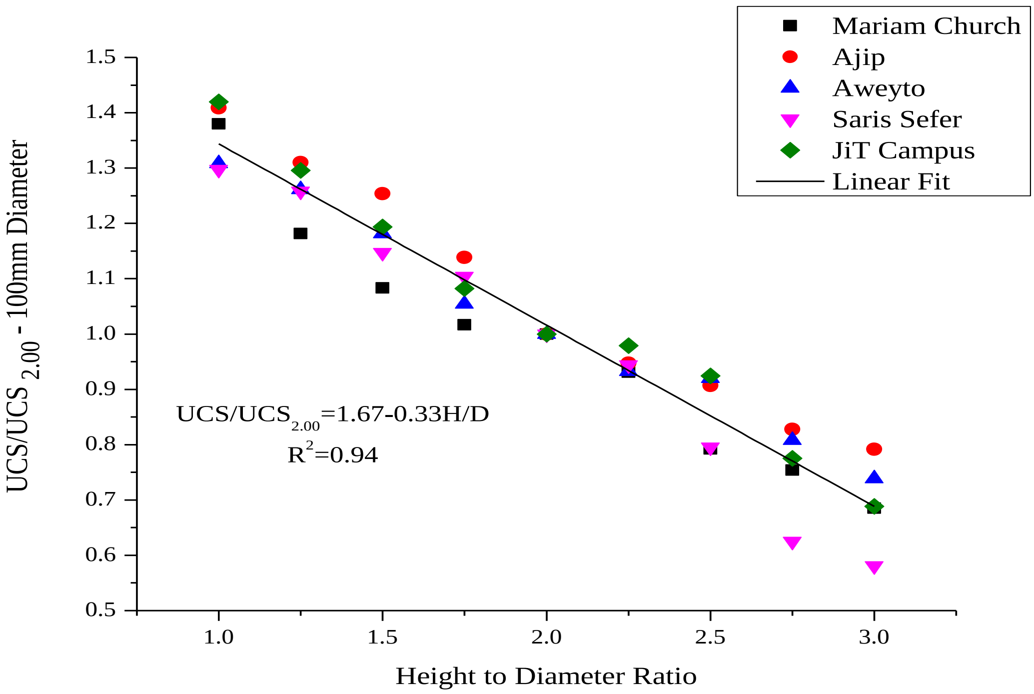

Figure 4-15 Plot of $\frac{UCS}{UCS_{2.00}}$ verses height to diameter ratio of 100mm specimen diameter

Accordingly, the general correction formulae for height to diameter ratio of 2.00 for the specimen's diameter of 38mm, 50mm and 100mm were given by:

$$\frac{UCS}{UCS_{2.00}} = a - b\left(\frac{H}{D}\right) \tag{1}$$

Where $a - b\frac{H}{D}$, the correcting factor for unconfined compressive strength of cohesive soil at 2 height to diameter ratio.

By rearranging equation (1) then

$$\mathrm{UCS}_{(\mathrm{corrected})} = \frac{\mathrm{UCS\ (measured)}}{a-b(\frac{H}{D})} \tag{2}$$

Where, $UCS_{2.00}$ is corrected unconfined compressive strength (kPa) at 2.00 height to diameter ratio of 38mm, 50mm and 100mm specimens' diameter, UCS is measured unconfined compressive strength (kPa) with in $1.00 \leq \frac{H}{D} \leq 3.00$, H is Specimen height, D is the specimen's diameter, 'a' is intercept and 'b' is the slope of linear regression fit lines.

Hence, the correlations coefficients, the intercept (a) and the slope (b) values were summarized in Table 4-6 below.

Table 4-6 Regression equations of $\frac{UCS}{UCS_{2.00}}$ ratio for different specimen diameter Vs height to diameter ratio

Specimens' Diameter	Regression equations	R^2	Correlation States	Equation Number
38mm	$\mathrm{UCS}_{2.00} = \frac{\mathrm{UCS}}{1.60 - 0.29(\frac{H}{D})}$	0.93	Strong	3
50mm	$\mathrm{UCS}_{2.00} = \frac{\mathrm{UCS}}{1.66 - 0.32(\frac{H}{D})}$	0.94	Strong	4
100mm	$\mathrm{UCS}_{2.00} = \frac{\mathrm{UCS}}{1.67 - 0.33(\frac{H}{D})}$	0.94	Strong	5

The linearly regression fitted trade lines in Figure 4-13 (38mm specimen diameter), Figure 4-14 (50mm specimen diameter) and Figure 4-15 (100mm specimen diameter) show that a strong negative correlation between $\frac{UCS}{UCS_{2.00}}$ ratio and H/D ratio. However, as shown in Table 2-1, they differ slightly from the previous study (Güneyli & Rüşen, 2016). This could be due to differences in the specimens' physical and engineering properties, as well as the loading strain rate used in the study.

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

By varying the height to diameter ratio of different specimens' diameter in a series of laboratory tests, the effect of specimen size on the unconfined compressive strength of Jimma Town's cohesive soil was assessed. The findings of this study lead to the following conclusions:

1. As the diameter of the specimens increases, the UCS value decreases linearly with increasing H/D ratio. Similarly, as the diameter of the specimens grows larger, the peak value of UCS for each successive H/D ratio decreases.
2. The stress-strain curve of small diameter sample specimens’ changes from ductile to brittle failure behavior as the H/D ratio increases. However, as the specimen's diameter increases, the curves of the series H/D ratios are governed by brittle behavior. The axial strains of the peak UCS values also decrease with both specimens’ diameters and their respective H/D ratios increase. Similarly, as the diameter of the specimens increases, the difference in axial strains between the smallest and largest H/D ratios decreases.
3. The failure patterns change as the diameters and H/D ratios of axially compressed cohesive soil specimens increase. Failure patterns with a clear, distinct with enough space length from the top and bottom of sample specimens (shear failure) were developed at 2 H/D ratios for 38mm diameter. This form of failure was observed in 50mm diameters with H/D ratios ranging from 1.75 to 2.25, and in 100mm diameters with H/D ratios ranging from 1.50 to 2.50.
4. Based on the uniformity of deformations, characteristics of failure patterns, and behavior of stress-strain curves found in this research, specimen diameters of 50mm and 100mm provide more precise and representative unconfined compression test results than specimen diameters of 38mm.
5. From 1.00 to 3.00 consecutive H/D ratios, the 2.00 H/D ratio in all diameters of specimens comes closest to the mean of unconfined compressive strength value. Accordingly, performing an unconfined compression test at a 2.00 H/D ratio yields a consistent and representative UCS of cohesive soil in 38mm, 50mm, and 100mm specimen diameters,

based on the uniformity of deformations, characteristics of failure patterns, and behavior of stress-strain curves.

6. Equations with strong correlation coefficients were then derived to convert the measured UCS values in between $1.00 \leq \frac{H}{D} \leq 3.00$ H/D ratio to UCS values of the optimum height to diameter ratio of 38mm, 50mm, and 100mm specimens' diameter.

5.2. Recommendations

According to these conclusions, the size of sample specimens affects the unconfined compressive strength of cohesive soil. As a result, to reduce the effect of specimen size, the test should be conducted at a height-to-diameter ratio of 2 and larger sizes of specimen's diameter. However, partitioners should consider that the loading strain rate used in this thesis to draw these conclusions was 1%/minute during the unconfined compression test.

For better understanding the implications of these findings, future studies could address

- The effect of endplate friction and specimens' diameter for unconfined compression strength of cohesive soils covered for wide coverage of study areas and sample specimens,
- The effect of loading strain rate and specimens' size on unconfined compressive strength of cohesive soils for the study area and around this study area.
- The conclusions were drawn based on the size of the specimens of compacted cohesive soils at their maximum dry density and moisture content. However, the study should be extended to undisturbed and remolded (at different moisture content) cohesive soil specimens.
- For future studies, the developed corrective formulas of unconfined compressive strength for the optimum height to diameter ratio in this research should accounts for water content, unit weight, and other physical properties of cohesive soil specimens.

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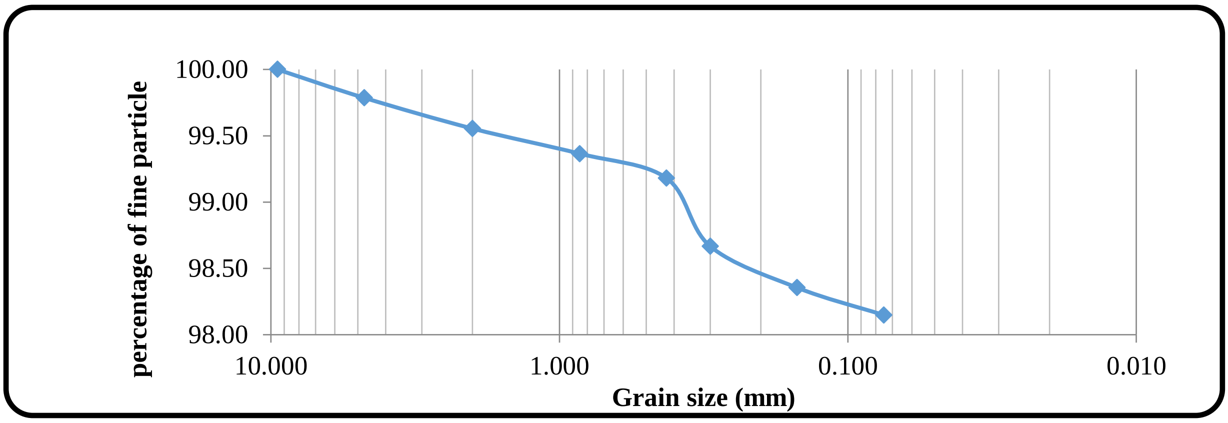
APPENDIXES

I. Grain Size Analysis Test Results

1.1.Grain Size Analysis Test of Mariam Church TEST PIT

Method of Testing:	Grain Size Analysis (ASTM D-421)
Method of Testing:	Wet Sieve Analysis
Wt. of Sample: (g)	1.0 Kg
Sample Location:	Mariam Church
Depth (m)	3m Blow Natural Ground Level

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.000	0.00	0.00	100.00
4.750	2.140	0.21	0.21	99.79
2.000	2.320	0.23	0.45	99.55
0.850	1.890	0.19	0.64	99.37
0.425	1.850	0.19	0.82	99.18
0.300	5.130	0.51	1.33	98.67
0.150	3.120	0.31	1.65	98.36
0.075	2.070	0.21	1.85	98.15
pan	981.480	98.15	100.00	0.00
sum	1000.000			

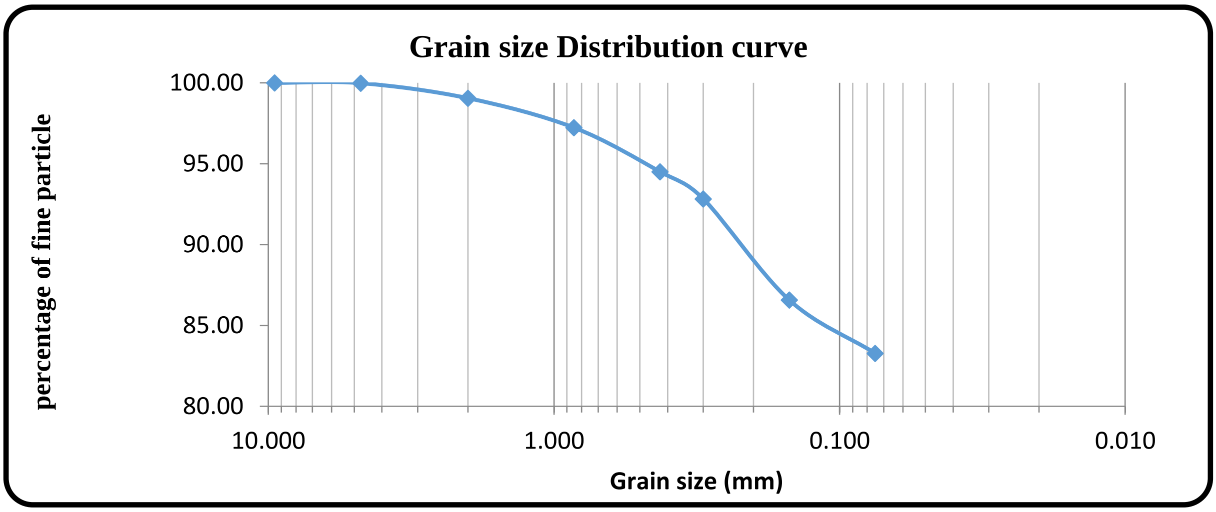


1.2.Grain Size Distribution Curve of Mariam Church Test Pit

2.1. Grain Size Analysis Test of Ajip Test Pit

Method of Testing:	Grain Size Analysis (ASTM D-421)
Method of Testing:	Wet Sieve Analysis
Wt. of Sample: (g)	1.0 Kg
Sample Location:	Ajib
Depth (m)	3m Blow NGL

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.000	0.00	0.00	100.00
4.750	0.187	0.02	0.02	99.98
2.000	9.300	0.93	0.95	99.05
0.850	18.230	1.82	2.77	97.23
0.425	27.230	2.72	5.49	94.51
0.300	16.750	1.68	7.17	92.83
0.150	62.560	6.26	13.43	86.57
0.075	32.930	3.29	16.72	83.28
pan	832.813	83.28	100.00	0.00
sum	1000.000			

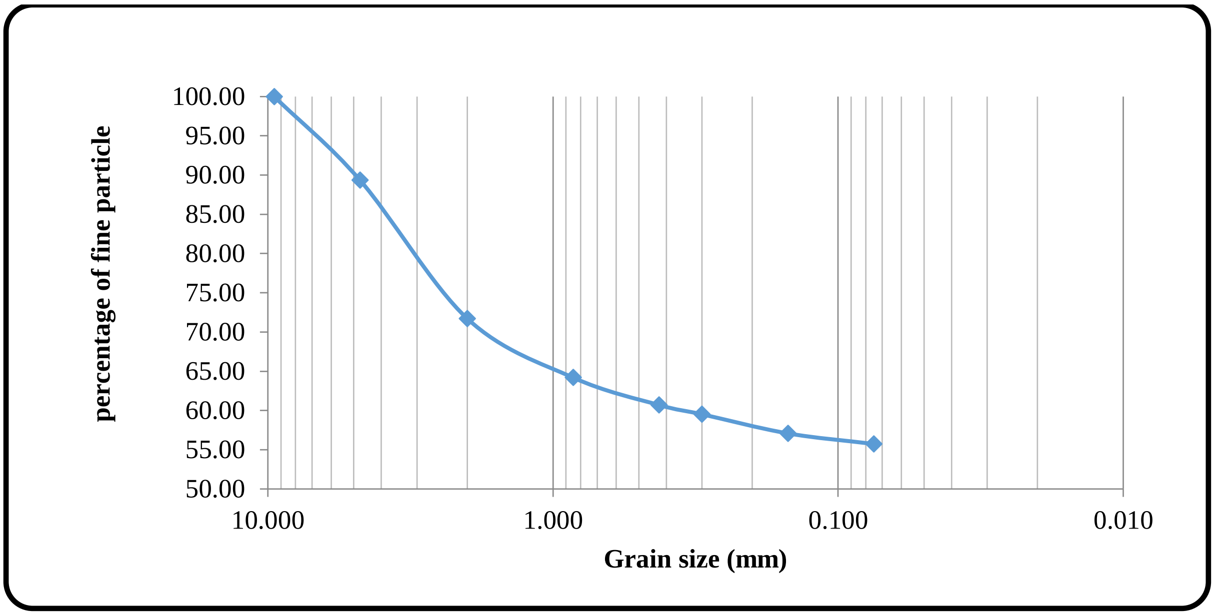


2.2. Grain Size Distribution Curve of Ajip Test Pit

3.1. Grain Size Analysis Test of Aweytu Test Pit

Method of Testing:	Grain Size Analysis (ASTM D-421)
Method of Testing:	Wet Sieve Analysis
Wt. of Sample: (g)	1.0 Kg
Sample Location:	Aweytu
Depth (m)	3m Blow NGL

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.000	0.00	0.00	100.00
4.750	106.550	10.66	10.66	89.35
2.000	176.210	17.62	28.28	71.72
0.850	75.340	7.53	35.81	64.19
0.425	34.860	3.49	39.30	60.70
0.300	11.880	1.19	40.48	59.52
0.150	24.350	2.44	42.92	57.08
0.075	13.460	1.35	44.27	55.74
pan	557.350	55.74	100.00	0.00
sum	1000.000			

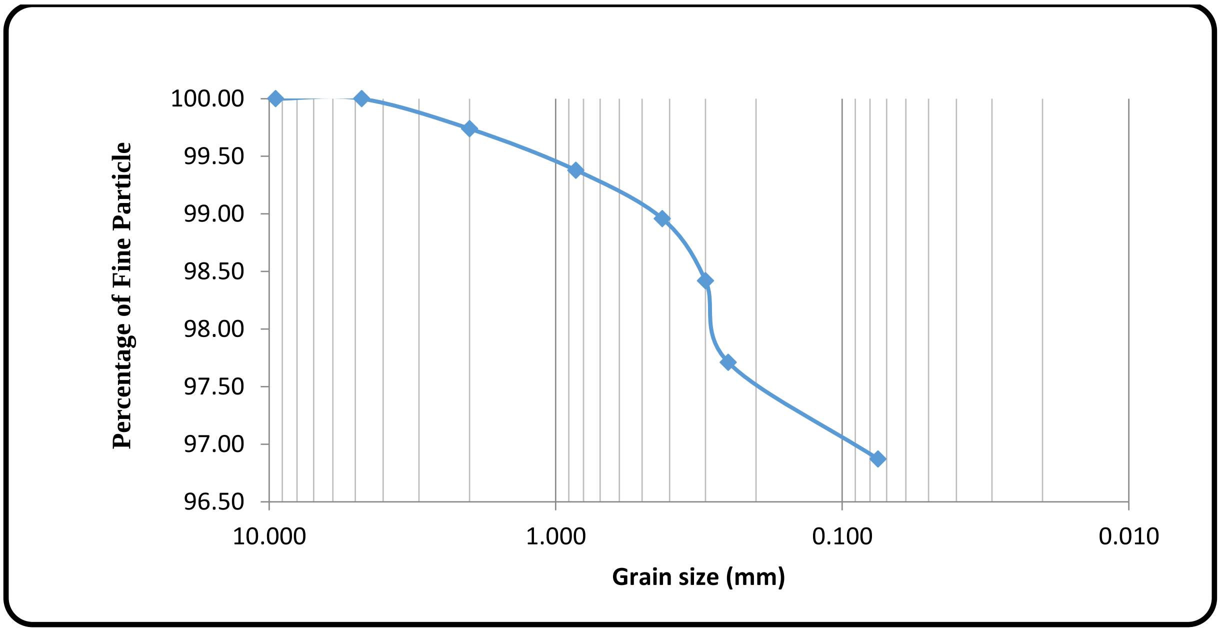


3.2. Grain Size Distribution Curve of Aweytu Test Pit

4.1. Grain Size Analysis Test of Saris Sefer Test Pit

Type of Testing:	Grain Size Analysis (ASTM D-421)
Method of Testing:	Wet Sieve Analysis
Wt. of Sample: (g)	0.50 Kg
Sample Location:	Saris Sefer
Depth (m)	2.5 m Below NGL

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.00	0.00	0.00	100.00
4.750	0.00	0.00	0.00	100.00
2.000	1.30	0.26	0.26	99.74
0.850	1.80	0.36	0.62	99.38
0.425	2.10	0.42	1.04	98.96
0.300	2.70	0.54	1.58	98.42
0.250	3.54	0.71	2.29	97.71
0.075	4.20	0.84	3.13	96.87
pan	484.36	96.87	100.00	0.00
sum	500.000			

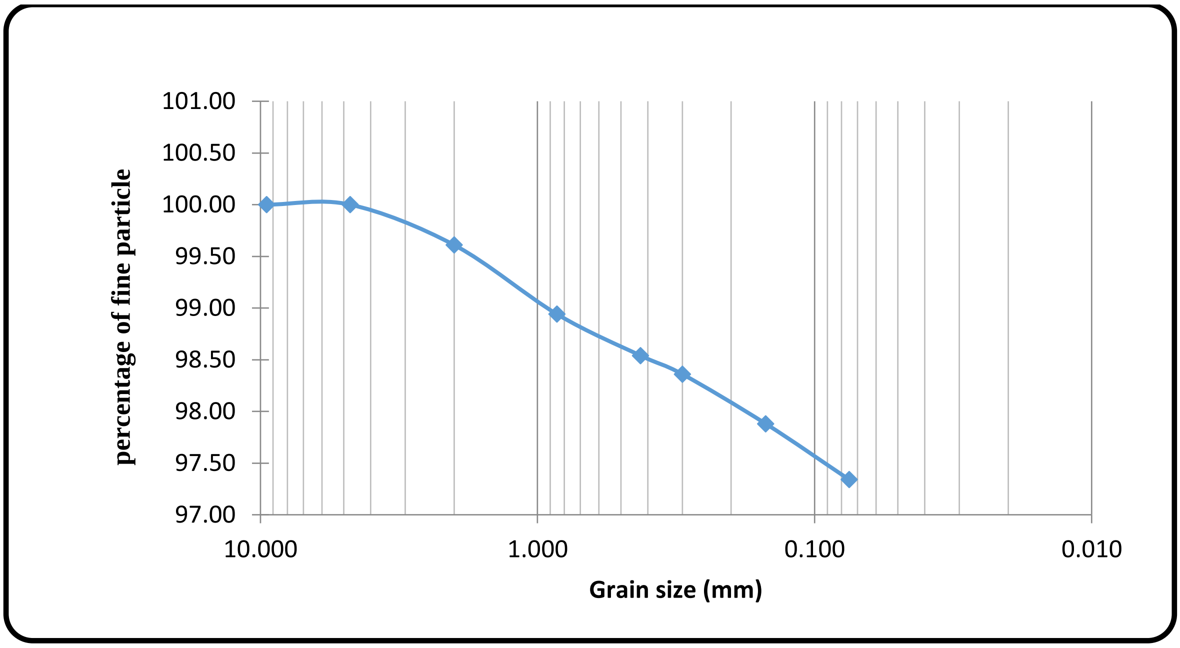


4.2. Grain Size Distribution Curve of Saris Sefer Test Pit

5.1. Grain Size Analysis Test of JiT Campus Test Pit

Method of Testing:	Grain Size Analysis (ASTM D-421)
Method of Testing:	Wet Sieve Analysis
Wt. of Sample: (g)	1.0 Kg
Sample Location:	JiT Campus
Depth (m)	3m Blow NGL

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.000	0.00	0.00	100.00
4.750	0.000	0.00	0.00	100.00
2.000	3.900	0.39	0.39	99.61
0.850	6.700	0.67	1.06	98.94
0.425	4.000	0.40	1.46	98.54
0.300	1.800	0.18	1.64	98.36
0.150	4.800	0.48	2.12	97.88
0.075	5.400	0.54	2.66	97.34
pan	973.400	97.34	100.00	0.00
sum	1000.000			

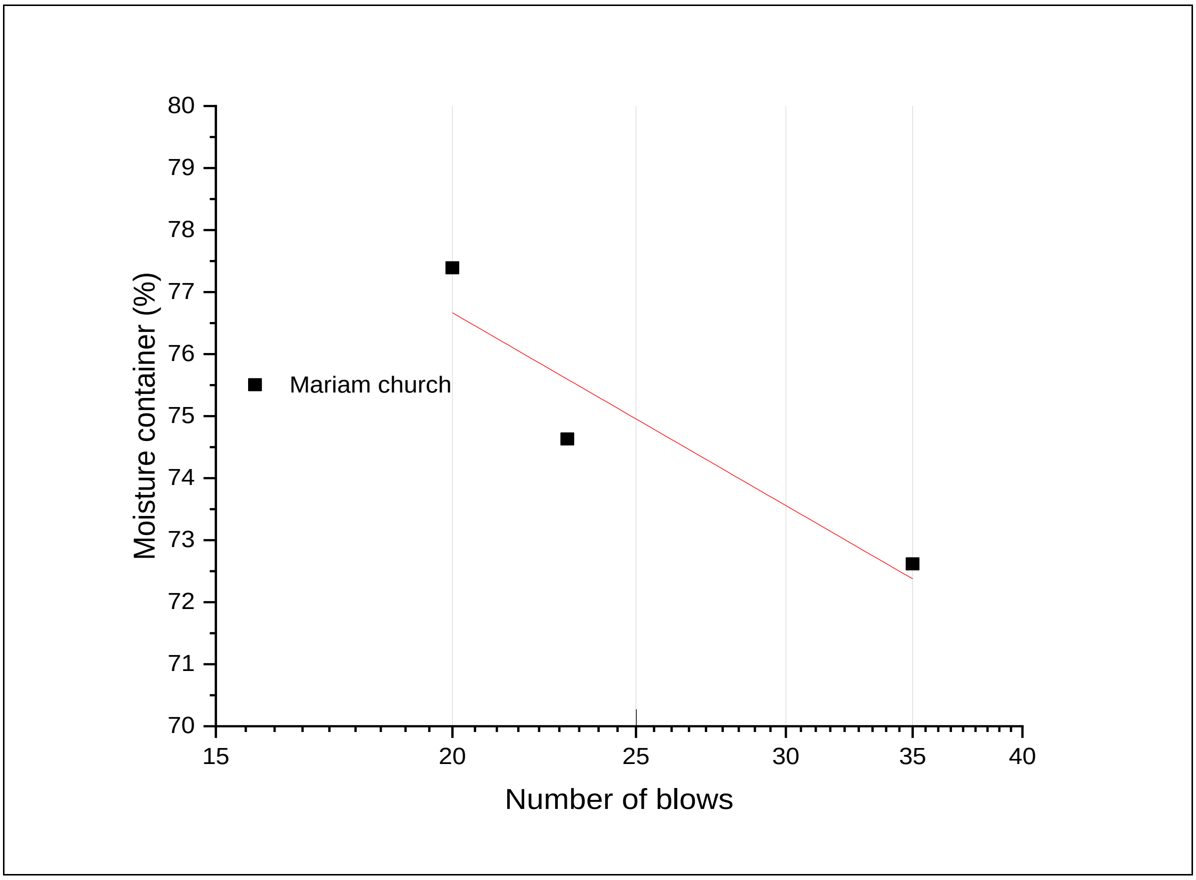


5.2. Grain Size Distribution Curve of JiT Campus Test Pit

II. Atterberg Limit Test Results

1.1.Liquid Limit and Plastic Limit Test Result Analysis of Mariam Church Test Pit

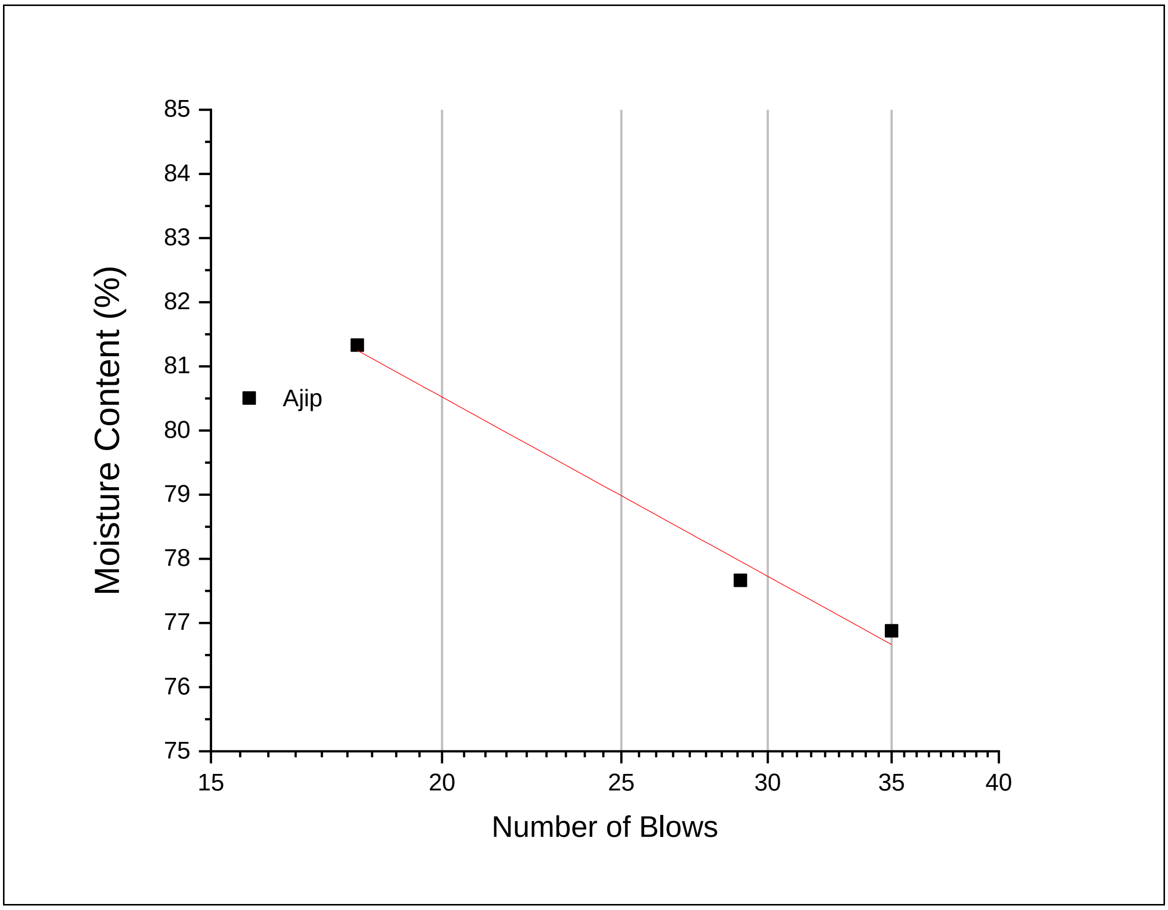
Test Pit Location:	Mariam Church				
Determination	Liquid Limit (ASTM D-4318)			Plastic Limit (ASTM D-4318)	
Number of blows	35	23	20		
Test No	01	02	03	01	02
Wt. of Container, (g)	18.00	17.66	17.18	17.43	18.53
Wt. of container + wet soil, (g)	29.41	31.84	28.87	24.63	30.41
Wt. of container + dry soil, (g)	24.61	25.78	23.77	22.26	26.44
Wt. of water, (g)	4.80	6.06	5.10	2.37	3.97
Wt. of dry soil, (g)	6.61	8.12	6.59	4.83	7.91
Moisture container, (%)	72.62	74.63	77.39	49.10	50.19
Average				49.10	



1.2. Flow Curve Analysis of Mariam Church Test Pit

2.1. Liquid Limit and Plastic Limit Test Result Analysis of Ajip Test Pit

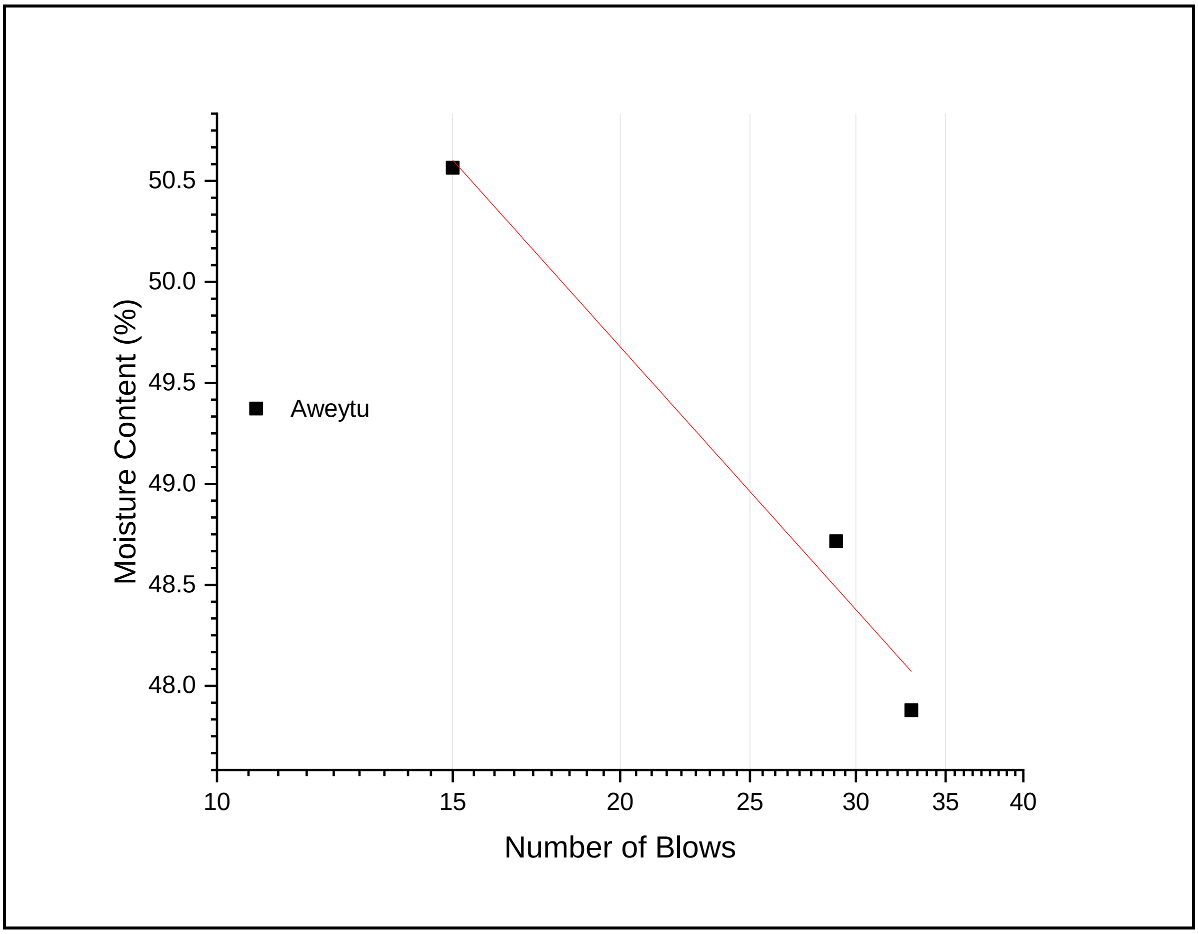
Test Pit Location:	Ajib				
Determination	Liquid Limit (ASTM D-4318)			Plastic Limit (ASTM D-4318)	
Number of blows	35	29	18		
Test No	01	02	03	01	02
Wt. of Container, (g)	6.48	5.85	6.21	18.04	17.32
Wt. of container + wet soil, (g)	18.49	19.85	20.10	23.15	25.46
Wt. of container + dry soil, (g)	13.27	13.73	13.87	21.91	23.45
Wt. of water, (g)	5.22	6.12	6.23	1.24	2.01
Wt. of dry soil, (g)	6.79	7.88	7.66	3.87	6.13
Moisture container, (%)	76.88	77.66	81.33	32.04	32.79
Average				32.42	



2.2. Flow Curve Analysis of Ajip Test Pit

3.1. Liquid Limit and Plastic Limit Test Result Analysis of Aweytu Test Pit

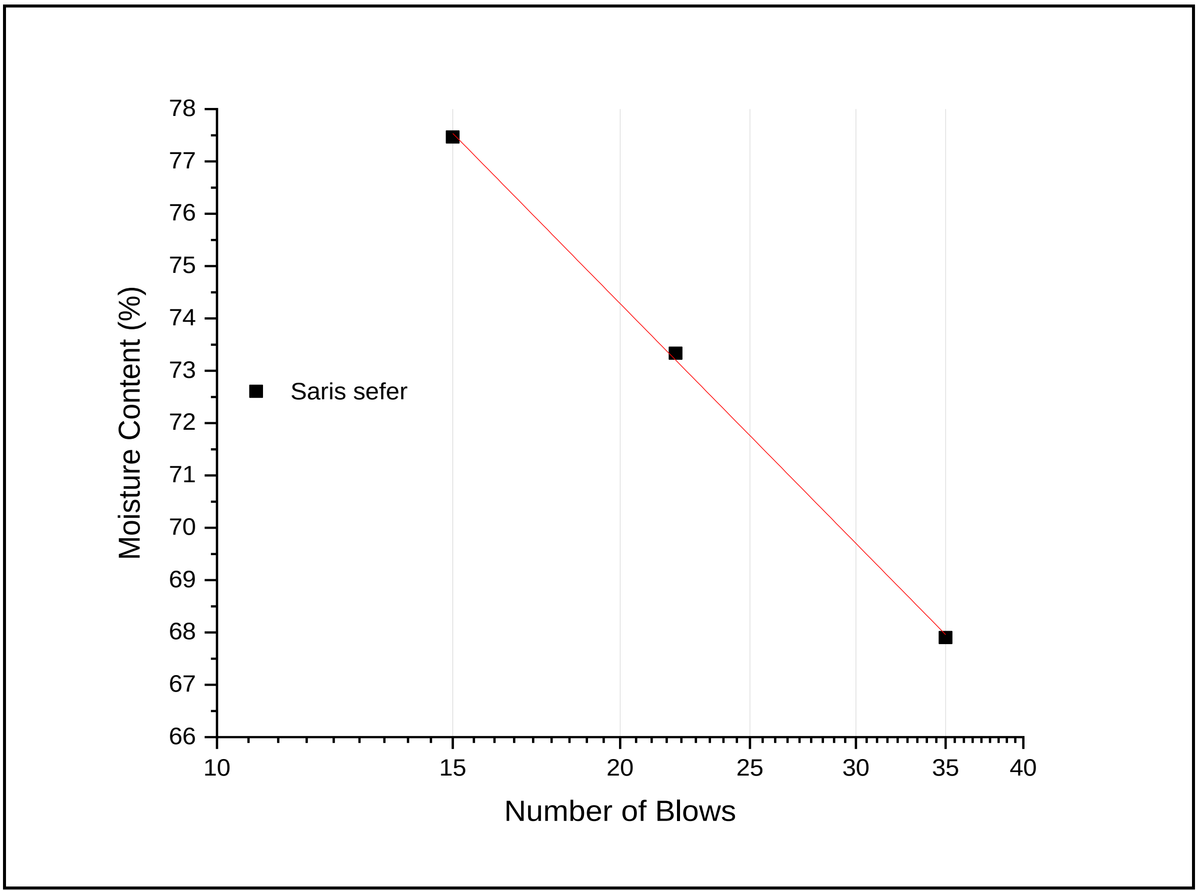
Test Pit Location:	Aweytu				
Determination	Liquid Limit (ASTM D-4318)			Plastic Limit (ASTM D-4318)	
Number of blows	33	29	15		
Test No	01	02	03	01	02
Wt. of Container, (g)	17.63	26.02	18.01	17.59	17.43
Wt. of container + wet soil, (g)	32.27	39.33	34.00	24.34	23.09
Wt. of container + dry soil, (g)	27.53	34.97	28.63	23.14	22.09
Wt. of water, (g)	4.74	4.36	5.37	1.20	1.00
Wt. of dry soil, (g)	9.90	8.95	10.62	5.55	4.66
Moisture container, (%)	47.88	48.72	50.56	21.62	21.46
Average				21.54	



3.2. Flow Curve Analysis of Aweytu Test Pit

4.1. Liquid Limit and Plastic Limit Test Result Analysis of Saris Sefer Test Pit

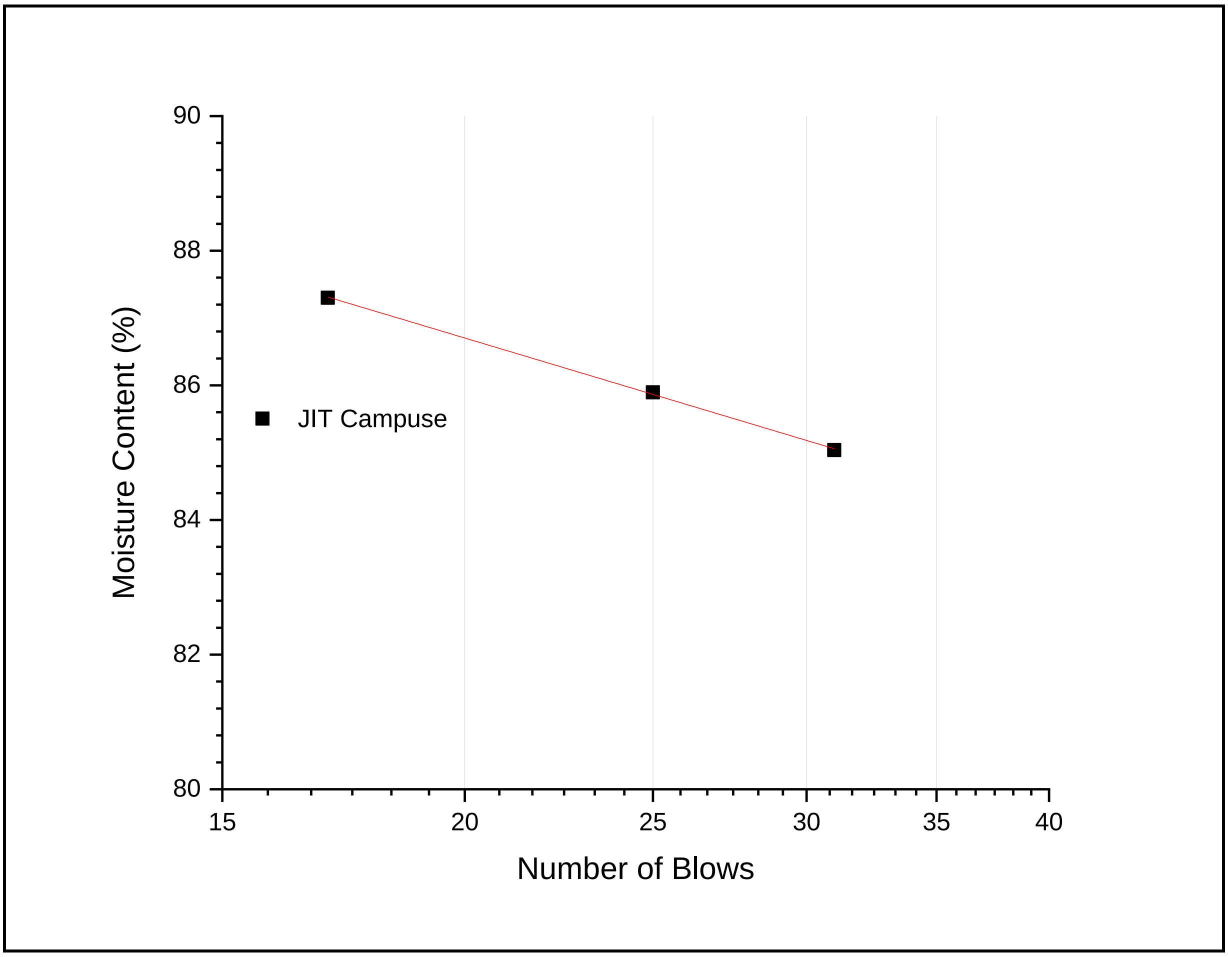
Test Pit Location:	Saris Sefer				
Determination	Liquid Limit (ASTM D-4318)			Plastic Limit (ASTM D-4318)	
Number of blows	35	22	15		
Test No	01	02	03	01	02
Wt. of Container, (g)	40.1	16.9	49.6	5.9	14.9
Wt. of container + wet soil, (g)	53.7	29.9	62.20	13.7	24.5
Wt. of container + dry soil, (g)	48.2	24.4	56.70	11.6	22
Wt. of water, (g)	5.50	5.50	5.50	2.10	2.50
Wt. of dry soil, (g)	8.10	7.50	7.10	5.70	7.10
Moisture container, (%)	68	73	77	37	35
Average				36	



4.2. Flow Curve Analysis of Saris Sefer Test Pit

5.1. Liquid Limit and Plastic Limit Test Result Analysis of JiT Campus Test Pit

Test Pit Location:	JiT Campus				
Determination	Liquid Limit (ASTM D-4318)			Plastic Limit (ASTM D-4318)	
Number of blows	31	25	17		
Test No	01	02	03	01	02
Wt. of Container, (g)	17.50	17.60	17.40	6.00	6.30
Wt. of container + wet soil, (g)	31.60	32.10	29.20	12.30	14.50
Wt. of container + dry soil, (g)	25.12	25.40	23.70	10.90	12.70
Wt. of water, (g)	6.48	6.70	5.50	1.40	1.80
Wt. of dry soil, (g)	7.62	7.80	6.30	4.90	6.40
Moisture container, (%)	85.04	85.90	87.30	28.57	28.13
Average				28.35	

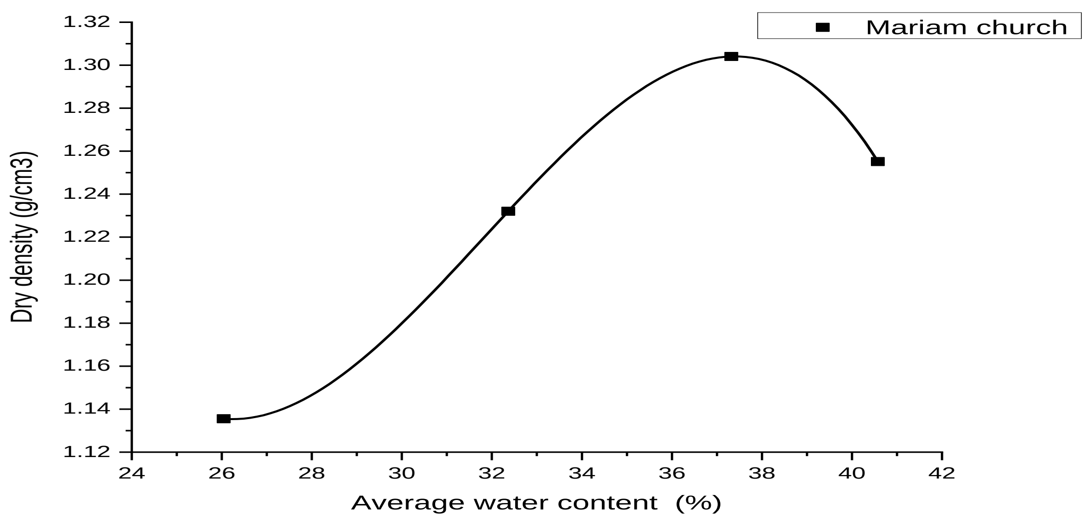


5.2. Flow Curve Analysis of JiT Campus Test Pit

III. Standared Proctor Compaction Test Results

1.1. Standard Proctor Compaction Test Result of Mariam Church Test Pit

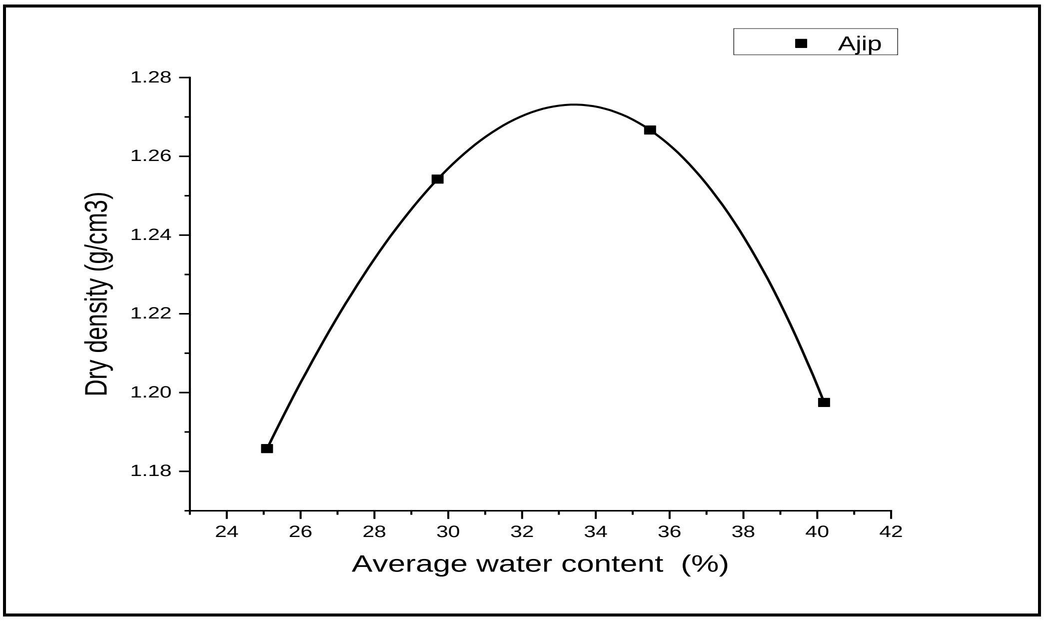
Determination of water content				
Trials	01	02	03	04
Mass of can, (gm)	25.9	40.7	41.2	17.9
	33.1	49.6	28.2	18.1
Mass of can +wet soil, (gm)	201.9	229.4	209.9	113.7
	215.3	238.8	191.4	116.3
Mass of can +dry soil, (gm)	163.7	183.4	164.2	86.2
	179.6	192.4	146.9	87.8
water content, (%)	27.72	32.24	37.15	40.26
	24.37	32.49	37.49	40.89
Average water content, (%)	26.04	32.36	37.32	40.58
Determination of Bulk density and Dry density				
Mass of mold + Compacted soil, (gm)	4475	4670.8	4827.8	4802
Bulk density, (g/cm^3)	1.43	1.63	1.79	1.76
Dry density, (g/cm^3)	1.14	1.23	1.30	1.26
Maximum Dry Density (g/cm^3)	1.31	Optimum Water Content (%)		37.5



1.2. Standard Proctor Compaction Curve of Mariam Church Test Pit

2.1. Standard Proctor Compaction Test Result of Ajip Test Pit

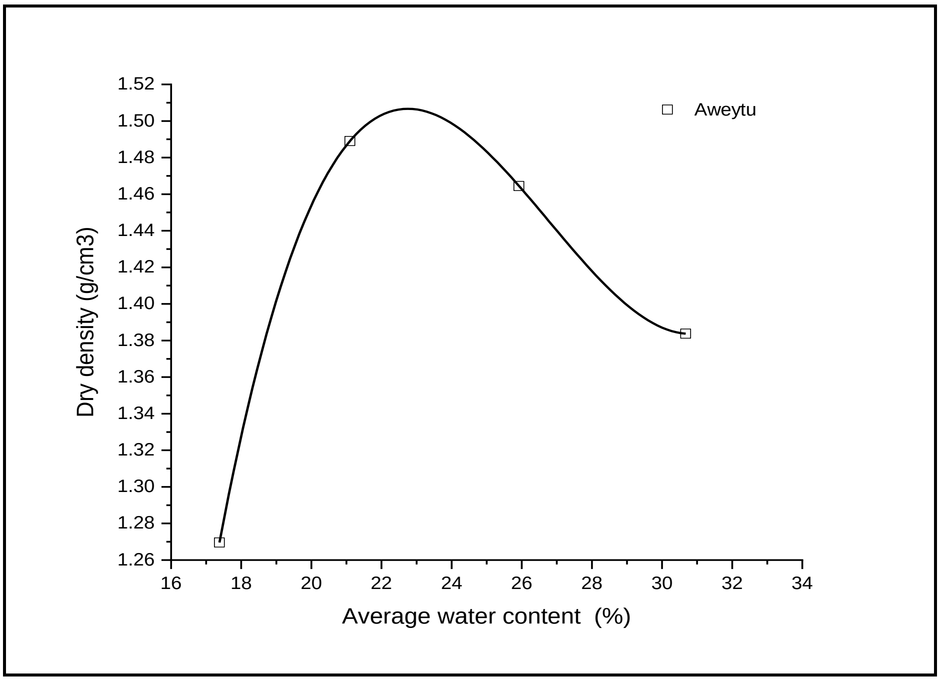
Determination of water content				
Trials	01	02	03	04
Mass of can, (gm)	18.1	17.6	18.2	17.4
	17.2	17.6	17.7	17.4
Mass of can +wet soil, (gm)	123.4	113.2	126.4	104.5
	118.9	131.6	129.6	123.8
Mass of can +dry soil, (gm)	102.4	91.3	99	79.2
	98.5	107	100.3	93.3
water content, (%)	24.91	29.72	33.91	40.94
	25.09	27.52	35.47	40.18
Average water content, (%)	25.09	29.72	35.47	40.18
Determination of Bulk density and Dry density				
Mass of mold + Compacted soil, (gm)	4526.1	4667	4754.4	4717.8
Bulk density, (g/cm^3)	1.48	1.63	1.72	1.68
Dry density, (g/cm^3)	1.19	1.25	1.27	1.20
Maximum Dry Density (g/cm^3)	1.27	Optimum Water Content (%)		34



2.2. Standard Proctor Compaction Curve of Ajip Test Pit

3.1. Standard Proctor Compaction Test Result of Aweytu Test Pit

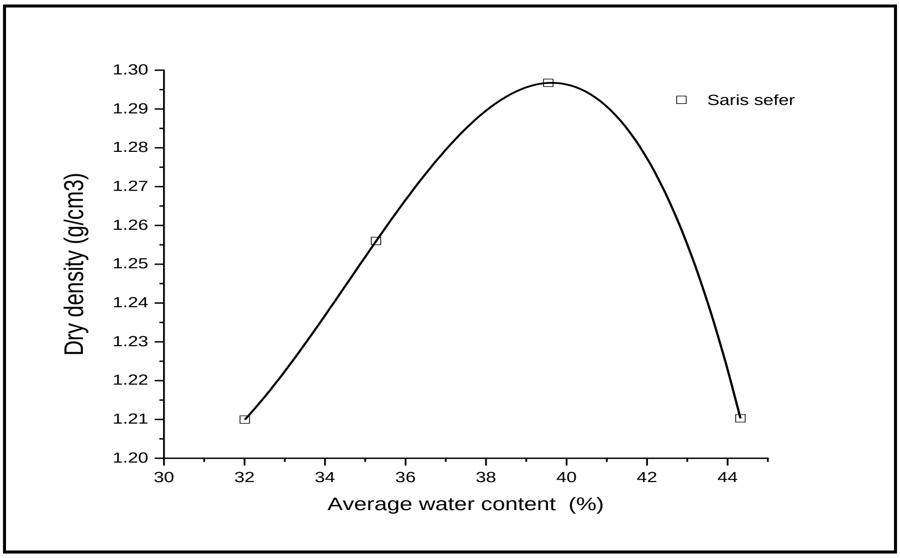
Determination of water content				
Trials	01	02	03	04
Mass of can	41.2	37.4	25.9	49.6
	28.7	29.5	31.1	40.7
Mass of can +wet soil	225.4	214.5	197.6	205.5
	205.8	204.8	200.9	214.1
Mass of can +dry soil	198	183.5	161.9	168.9
	179.7	174.4	166.3	173.4
water content	17.47	21.22	26.25	30.68
	17.28	20.98	25.59	30.67
Average water content	17.38	21.10	25.92	30.67
Determination of Bulk density and Dry density				
Mass of mold + Compacted soil, (gm)	4532.9	4840	4880.1	4844.9
Bulk density, (g/cm^3)	1.49	1.80	1.84	1.81
Dry density, (g/cm^3)	1.27	1.49	1.46	1.38
Maximum Dry Density (g/cm^3)	1.51	Optimum Water Content (%)		23



3.2. Standard Proctor Compaction Curve of Aweytu Test Pit

4.1. Standard Proctor Compaction Test Result of Saris Sefer Pit

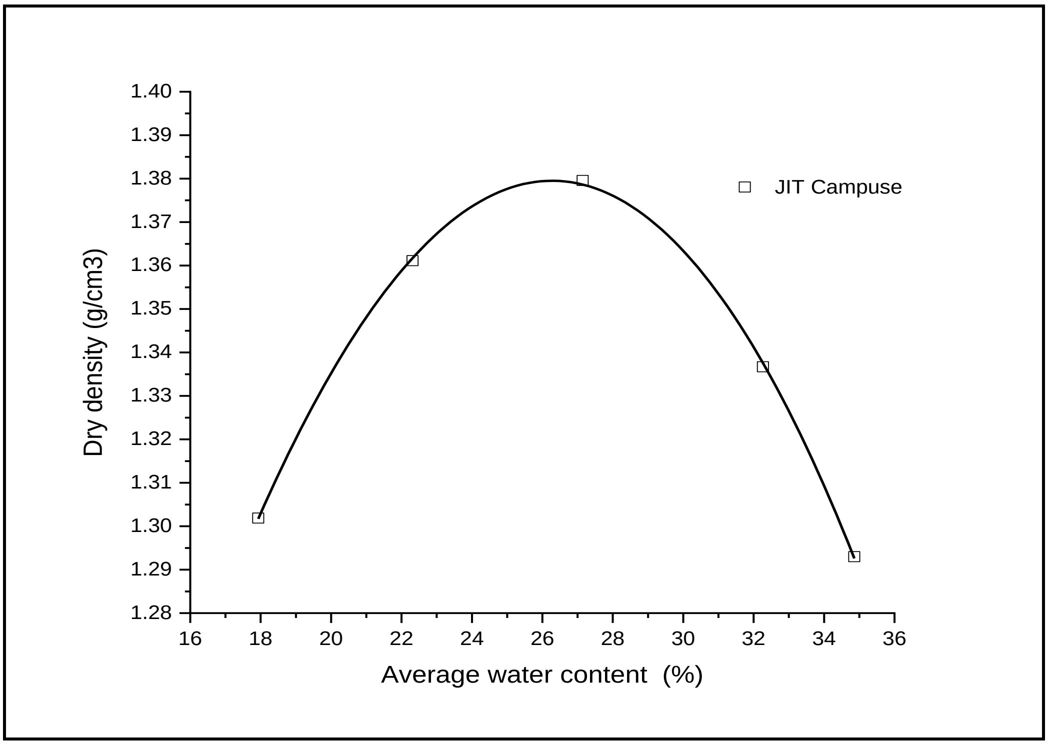
Determination of water content				
Trials	01	02	03	04
Mass of can	17.5	17.7	17.2	17.9
	17.2	17.7	17.5	17.4
Mass of can +wet soil	128.5	142.4	120.6	135.6
	119.8	114.1	129.7	124.4
Mass of can +dry soil	101.5	110.1	91.3	99.5
	95	88.8	97.9	91.5
water content	32.14	34.96	39.54	44.24
	31.88	35.58	39.55	44.40
Average water content	32.01	35.27	39.55	44.32
Determination of Bulk density and Dry density				
Mass of mold + Compacted soil, (gm)	4637.9	4737.7	4846.2	4784.5
Bulk density, (g/cm^3)	1.60	1.70	1.81	1.75
Dry density, (g/cm^3)	1.21	1.26	1.30	1.21
Maximum Dry Density (g/cm^3)	1.30	Optimum Water Content (%)		39



4.2. Standard Proctor Compaction Curve of Saris Sefer Test Pit

5.1. Standard Proctor Compaction Test Result of JiT Campus Test Pit

Determination of water content					
Trials	01	02	03	04	05
Mass of can	17.7	17.6	18	17.5	17.4
	18.6	17.6	17.1	17.7	17.6
Mass of can +wet soil	97.7	102.7	94.7	105.9	120.4
	103.7	104.6	124.7	114.2	108.9
Mass of can +dry soil	85.5	87.4	78.2	84.3	95.8
	90.8	88.5	101.9	90.7	85.3
water content	17.99	21.92	27.41	32.34	31.38
	17.87	22.71	26.89	32.19	34.86
Average water content	17.93	22.31	27.15	32.26	34.86
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	4577.1	4704.2	4791.8	4805.4	4781.6
Bulk density, (g/cm^3)	1.54	1.66	1.75	1.77	1.74
Dry density, (g/cm^3)	1.30	1.36	1.38	1.34	1.29
Maximum Dry Density (g/cm^3)	1.38	Optimum Water Content (%)			26.5



5.2. Standard Proctor Compaction Curve of JiT Campus Test Pit

IV. Unconfined Compresive Strength Test Results and Analysis

A. Analysis of unconfined compressive strength

i. The row data; sample's deformation (∆H) in millimeter and resistant load (L) in Newton were directly taken from the universal compression test machine which was calibrated by the outraised Institute and then since the calibrated value and the actual value was similar the researcher was taken directly from the universal compression test machine.

ii. Axial strain (ε), $\varepsilon = \frac{axial\ deformation\ (\Delta H)}{Specimen's\ Initial\ height\ (Ho)}$

iii. Axial strain in present (ε%), $\varepsilon\% = \frac{\Delta H}{Ho} * 100$

iv. Specimen's actual area (Ao), $Ao = \frac{\pi * (specimen'sdiameter\ (D))^2}{4}$

v. Specimen's corrected area (Ac), $Ac = \frac{Ao}{1-\varepsilon}$

vi. Axial stress (δ), $\sigma = \frac{Resistant\ reading\ load\ (L)}{\text{Specimen's corrected area (Ac)}}$

vii. Unconfined compressive strength (UCS), it is the peak or maximum axial stress from stress strain curve

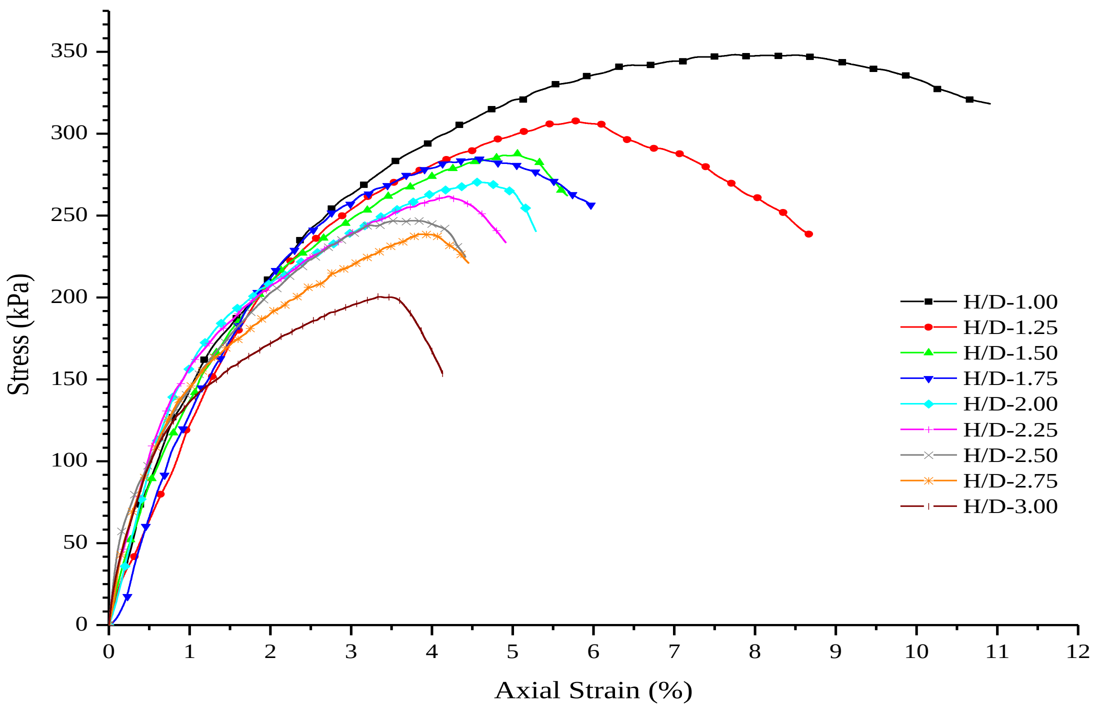
Those calculations were calculated using Microsoft excel 2016 and the graphs were plotted using origin lab 8.5 software. But since it was difficult to put for all 135 specimens' their UCS analyses of tables and graphs, the summery of peak UCS and graphs were arranged by their specimens' diameter for each test pits with respect of their height to diameter ratio.

B. Stress strain graphs and summery of unconfined compressive strength tables

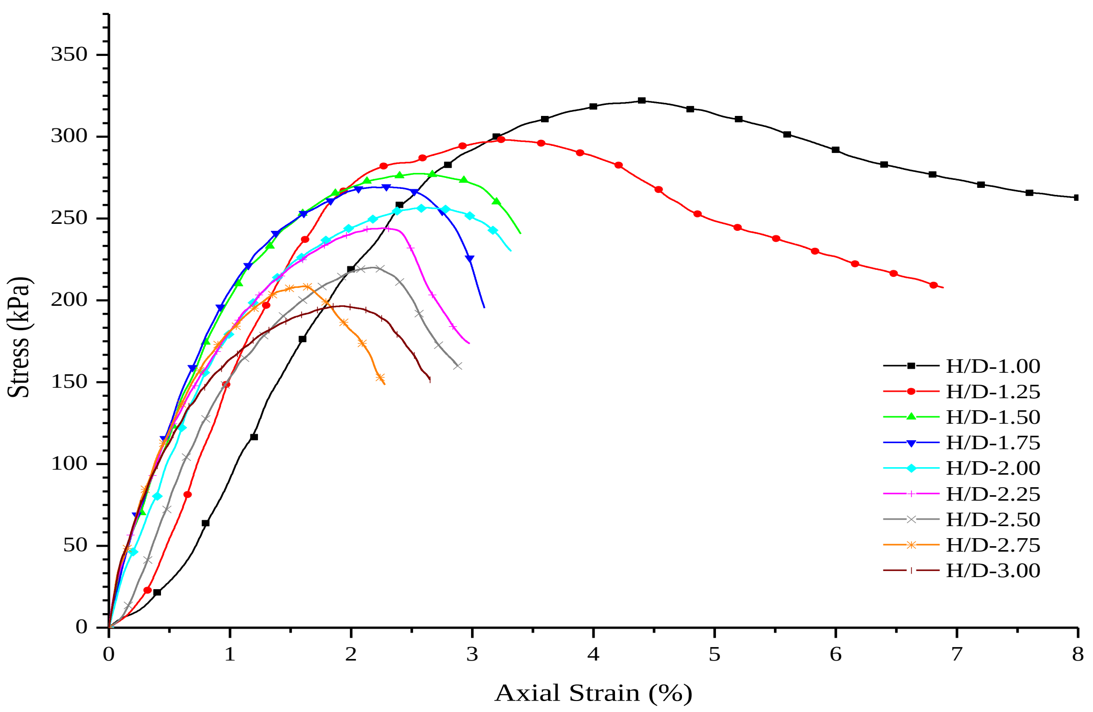
1. Table of detail calculation axil strain in present and axial stress for 38mm specimen diameter at 1 and 1.25 height to diameter ratio of Mariam Church test pit

H/D Ratio	Mariam Church -38mm Diameter, H/D-1.00				MH-38D-H/D-1.25			
Deformation (mm)	Diameter (mm)	38.1	Height (mm)	38.00	Diameter (mm)	38.1	Height (mm)	46.70
	Reading resistance load (N)	Corrected area (cm^2)	Strain (%)	Stress (kPa)	Reading resistance load (N)	Corrected area (cm^2)	Strain (%)	Stress (kPa)
0.00	0.0	11.40	0.0	0.0	0.0	11.40	0.0	0.0
0.05	28.1	11.42	0.1	24.6	24.4	11.41	0.1	21.4
0.10	47.5	11.43	0.3	41.6	38.7	11.43	0.2	33.8
0.15	84.2	11.45	0.4	73.6	47.7	11.44	0.3	41.7
0.20	103.7	11.46	0.5	90.5	64.8	11.45	0.4	56.6
0.25	123.1	11.48	0.7	107.3	77.9	11.46	0.5	67.9
0.30	145.8	11.49	0.8	126.9	91.5	11.47	0.6	79.8
0.35	154.4	11.51	0.9	134.2	102.3	11.49	0.7	89.1
0.40	171.7	11.52	1.1	149.0	117.7	11.50	0.9	102.3
0.45	186.8	11.54	1.2	161.9	137.0	11.51	1.0	119.0
0.50	199.8	11.55	1.3	172.9	147.8	11.52	1.1	128.2
0.55	207.4	11.57	1.4	179.2	163.7	11.54	1.2	141.9
0.60	217.1	11.58	1.6	187.4	175.1	11.55	1.3	151.6
0.65	224.6	11.60	1.7	193.7	185.3	11.56	1.4	160.3
0.70	234.4	11.61	1.8	201.8	199.5	11.57	1.5	172.4
0.75	245.2	11.63	2.0	210.8	208.6	11.59	1.6	180.0
0.80	256.0	11.65	2.1	219.8	218.3	11.60	1.7	188.2
0.85	262.4	11.66	2.2	225.0	229.1	11.61	1.8	197.3
0.90	274.3	11.68	2.4	234.9	238.7	11.62	1.9	205.4
0.95	284.0	11.69	2.5	242.9	245.6	11.64	2.0	211.0
1.00	288.4	11.71	2.6	246.3	251.2	11.65	2.1	215.7
1.05	298.1	11.72	2.8	254.2	259.2	11.66	2.2	222.2
1.10	305.6	11.74	2.9	260.3	264.9	11.68	2.4	226.9
1.15	310.0	11.76	3.0	263.6	271.1	11.69	2.5	232.0
1.20	316.4	11.77	3.2	268.8	276.3	11.70	2.6	236.1
1.25	322.9	11.79	3.3	273.9	284.2	11.71	2.7	242.6
1.30	328.3	11.80	3.4	278.1	288.2	11.73	2.8	245.7
1.35	334.8	11.82	3.6	283.2	293.3	11.74	2.9	249.8
1.40	340.2	11.84	3.7	287.4	298.4	11.75	3.0	253.9
1.45	344.5	11.85	3.8	290.7	302.4	11.77	3.1	257.0
1.50	348.8	11.87	3.9	293.9	308.1	11.78	3.2	261.6
1.55	355.3	11.89	4.1	298.9	310.9	11.79	3.3	263.7
1.60	357.5	11.90	4.2	300.4	315.5	11.81	3.4	267.2
1.65	364.0	11.92	4.3	305.4	319.5	11.82	3.5	270.3
1.70	367.2	11.93	4.5	307.7	322.3	11.83	3.6	272.4
1.75	372.6	11.95	4.6	311.8	326.3	11.84	3.7	275.5

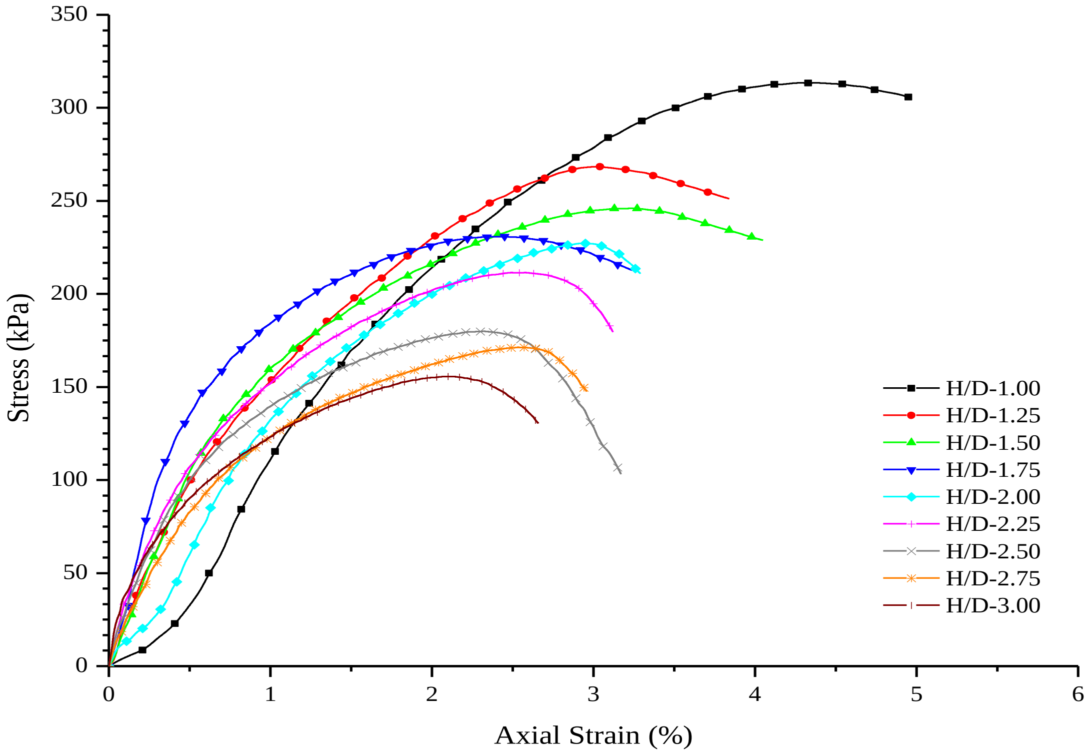
1.80	376.9	11.97	4.7	314.9	329.1	11.86	3.9	277.6
1.85	379.1	11.98	4.9	316.3	332.0	11.87	4.0	279.6
1.90	385.6	12.00	5.0	321.3	335.9	11.88	4.1	282.7
1.95	385.6	12.02	5.1	320.8	338.2	11.90	4.2	284.3
2.00	392.0	12.03	5.3	325.8	341.6	11.91	4.3	286.8
2.05	394.2	12.05	5.4	327.1	344.5	11.92	4.4	288.9
2.10	398.5	12.07	5.5	330.2	345.6	11.94	4.5	289.5
2.15	399.6	12.08	5.7	330.7	350.2	11.95	4.6	293.0
2.20	401.8	12.10	5.8	332.0	352.4	11.96	4.7	294.6
2.25	406.1	12.12	5.9	335.1	355.3	11.98	4.8	296.6
2.30	408.2	12.14	6.1	336.4	357.0	11.99	4.9	297.7
2.35	410.4	12.15	6.2	337.7	359.8	12.01	5.0	299.7
2.40	414.7	12.17	6.3	340.8	362.1	12.02	5.1	301.3
2.45	416.9	12.19	6.4	342.1	363.2	12.03	5.2	301.9
2.50	416.9	12.20	6.6	341.6	366.6	12.05	5.4	304.4
2.55	418.0	12.22	6.7	342.0	368.9	12.06	5.5	305.9
2.60	420.1	12.24	6.8	343.3	368.9	12.07	5.6	305.6
2.65	422.3	12.26	7.0	344.6	370.6	12.09	5.7	306.6
2.70	422.3	12.27	7.1	344.1	372.3	12.10	5.8	307.7
2.75	426.6	12.29	7.2	347.1	371.2	12.11	5.9	306.4
2.80	426.6	12.31	7.4	346.6	371.2	12.13	6.0	306.1
2.85	427.7	12.33	7.5	347.0	371.2	12.14	6.1	305.7
2.90	428.8	12.34	7.6	347.4	366.6	12.16	6.2	301.6
2.95	430.9	12.36	7.8	348.6	363.8	12.17	6.3	298.9
3.00	429.8	12.38	7.9	347.3	360.9	12.18	6.4	296.3
3.05	430.9	12.40	8.0	347.6	359.8	12.20	6.5	295.0
3.10	432.0	12.41	8.2	348.0	356.4	12.21	6.6	291.9
3.15	432.0	12.43	8.3	347.5	355.8	12.23	6.7	291.1
3.20	433.1	12.45	8.4	347.9	356.4	12.24	6.9	291.2
3.25	434.2	12.47	8.6	348.2	353.6	12.25	7.0	288.5
3.30	433.1	12.49	8.7	346.9	353.0	12.27	7.1	287.7
3.35	433.1	12.50	8.8	346.4	350.7	12.28	7.2	285.6
3.40	432.0	12.52	8.9	345.0	347.3	12.30	7.3	282.5
3.45	430.9	12.54	9.1	343.7	344.5	12.31	7.4	279.8
3.50	429.8	12.56	9.2	342.3	339.3	12.32	7.5	275.3
3.55	428.8	12.58	9.3	340.9	335.9	12.34	7.6	272.3
3.60	427.7	12.59	9.5	339.6	333.1	12.35	7.7	269.6
3.65	427.7	12.61	9.6	339.1	328.0	12.37	7.8	265.2
3.70	425.5	12.63	9.7	336.9	324.6	12.38	7.9	262.1
3.75	424.4	12.65	9.9	335.5	323.4	12.40	8.0	260.9
3.80	422.3	12.67	10.0	333.4	318.9	12.41	8.1	256.9
3.85	420.1	12.69	10.1	331.2	316.1	12.43	8.2	254.4
3.90	415.8	12.70	10.3	327.3	313.2	12.44	8.4	251.8
3.95	414.7	12.72	10.4	325.9	306.9	12.45	8.5	246.5
4.00	411.5	12.74	10.5	322.9	301.3	12.47	8.6	241.6
4.05	409.3	12.76	10.7	320.8	297.9	12.48	8.7	238.6
4.10	408.2	12.78	10.8	319.4				
4.15	407.2	12.80	10.9	318.1				



1.1. Stress strain curves for 38mm specimen diameter of Mariam Church test pit



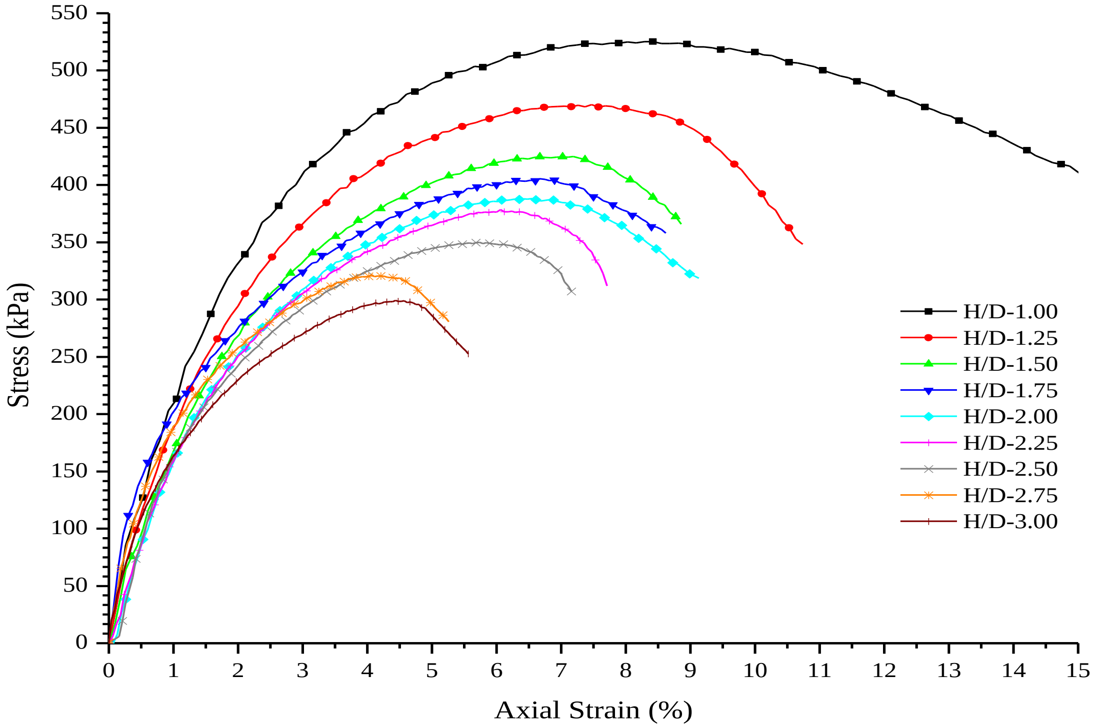
1.2.Stress strain curves for 50mm specimen diameter of Mariam Church test pit



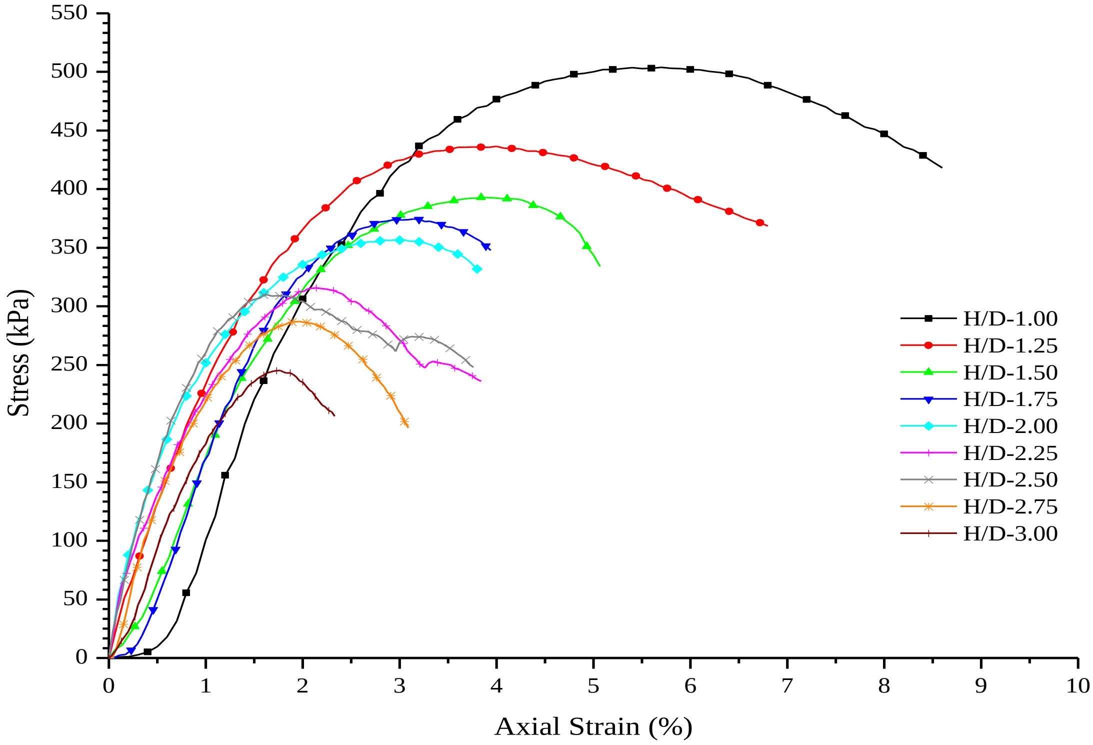
1.3.Stress strain curves for 100mm specimen diameter of Mariam Church test pit

1.4. Table of peak UCS value (kPa) for 38mm specimen diameter of Mariam Church test pit

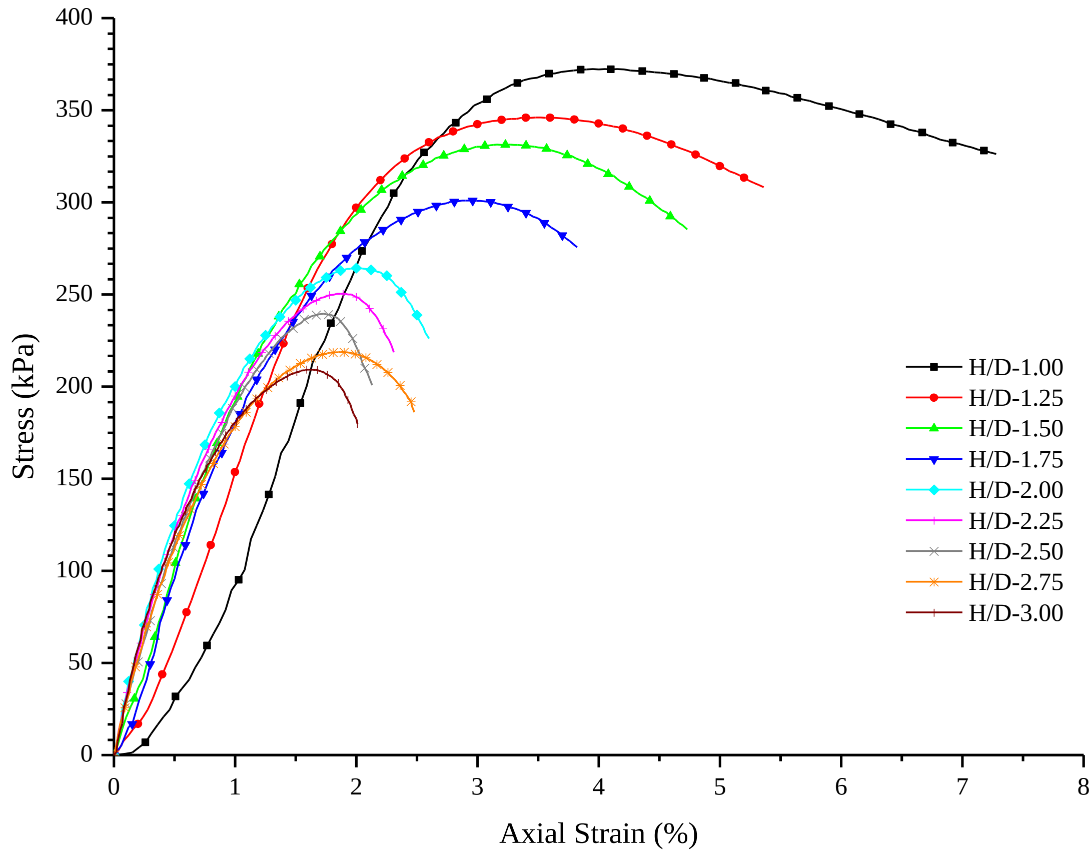
Height to diameter ratio	Specimen's diameter		
	38mm	50mm	100mm
1.00	348.63	321.97	313.46
1.25	307.69	298.23	268.37
1.50	287.61	277.64	246.06
1.75	284.75	269.48	230.90
2.00	270.38	256.87	227.17
2.25	262.25	244.32	211.54
2.50	247.04	220.10	179.93
2.75	238.73	208.81	171.32
3.00	200.54	196.73	155.66



2.1.Stress strain curves for 38mm specimen diameter of Ajip test pit



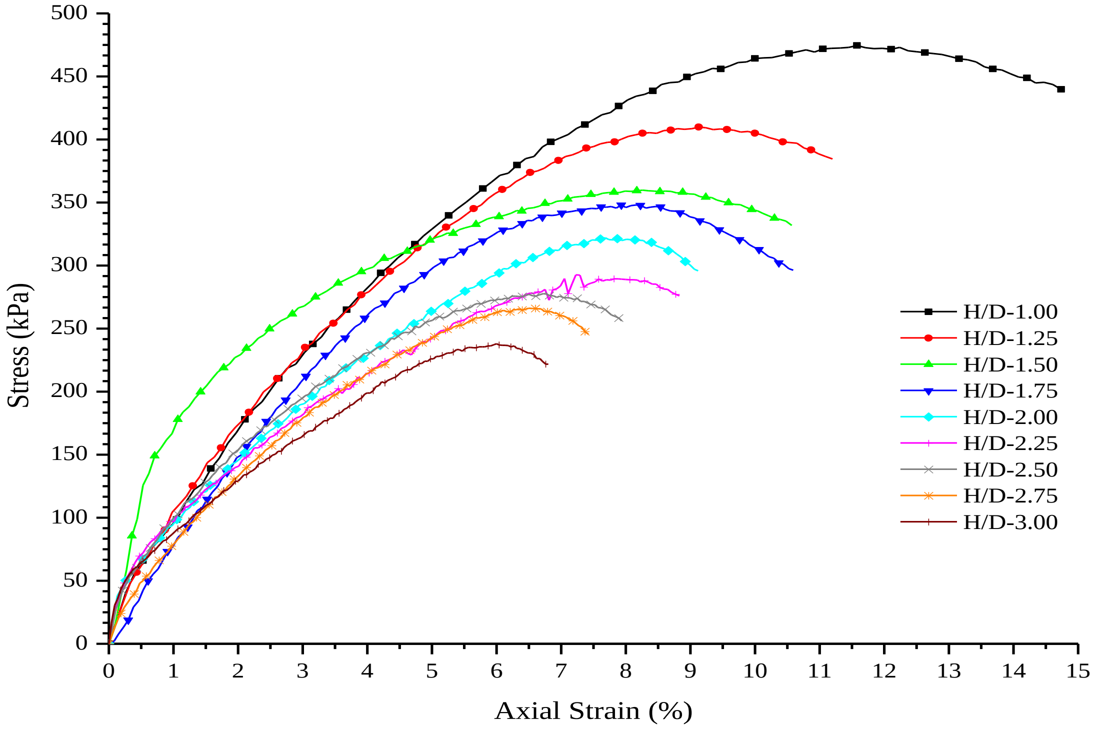
2.2.Stress strain curves for 50mm specimen diameter of Ajip test pit



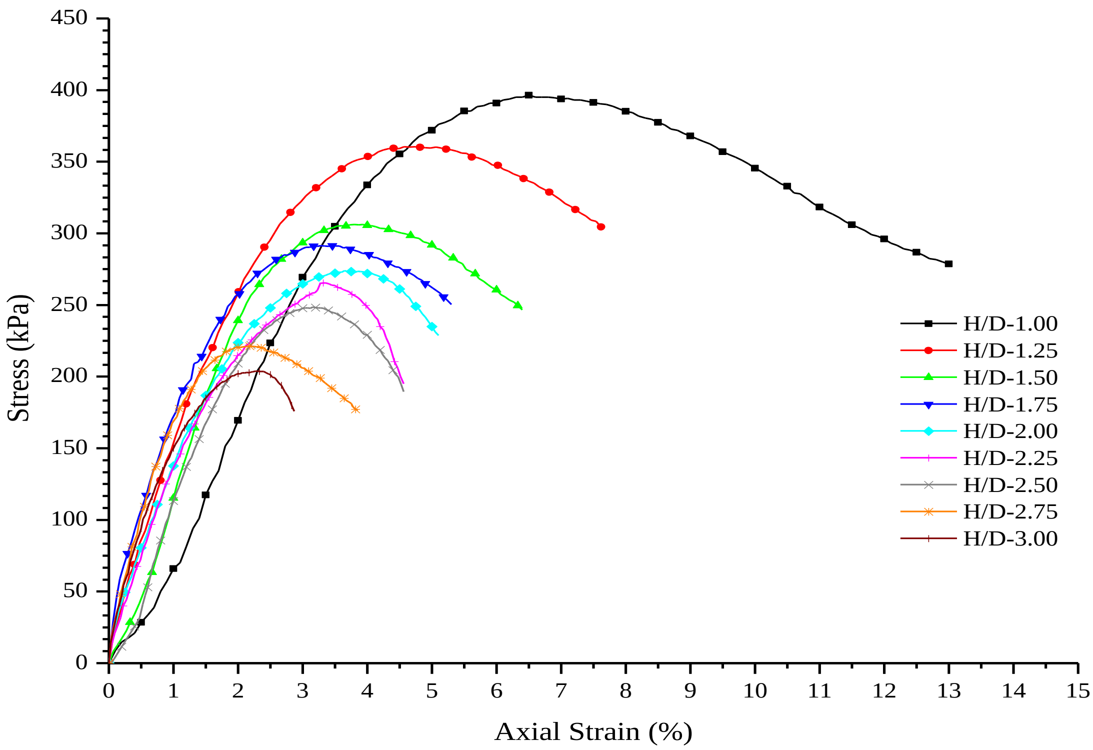
2.3.Stress strain curves for 100mm specimen diameter of Ajip test pit

2.4.Table of peak UCS value (kPa) for each specimens' diameter of Ajip test

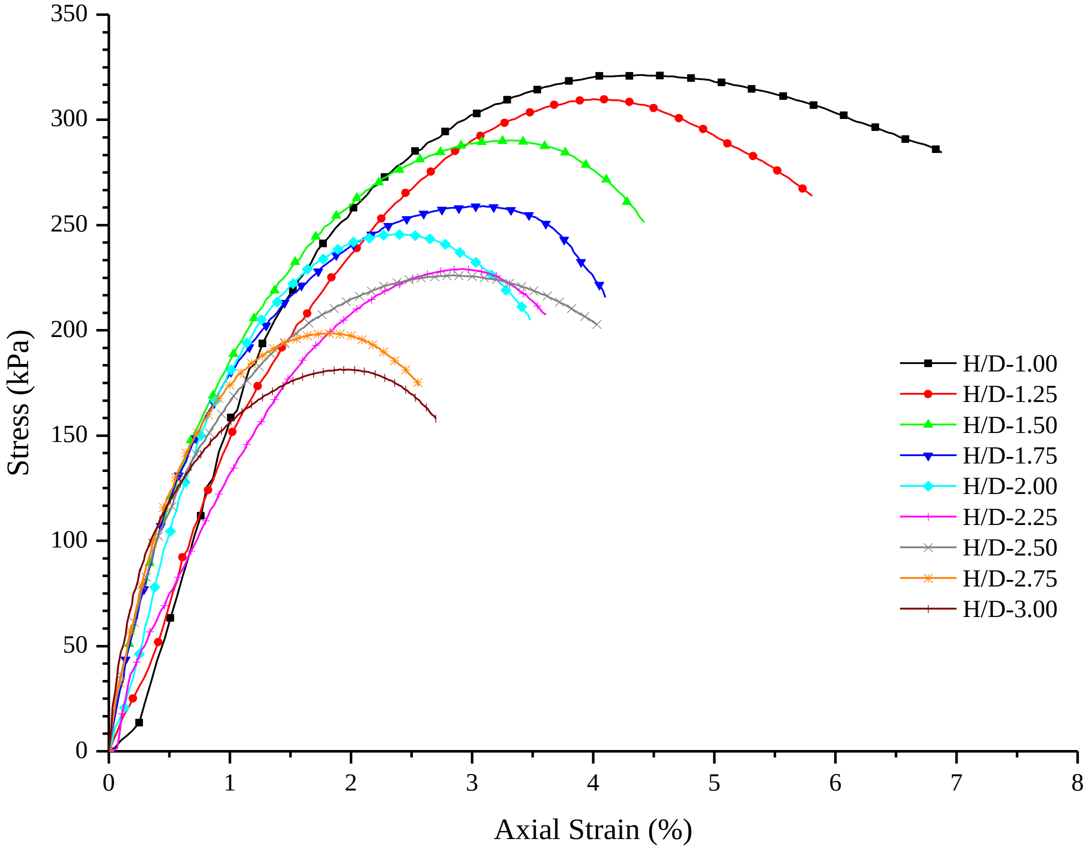
Height to diameter ratio	Specimen's diameter		
	38mm	50mm	100mm
1.00	525.21	503.90	372.44
1.25	470.32	436.37	346.17
1.50	424.81	392.74	331.32
1.75	405.40	374.42	300.99
2.00	*388.33*	*356.62*	264.32
2.25	*378.53*	*315.73*	250.39
2.50	*350.01*	*309.83*	239.65
2.75	*321.77*	*287.04*	218.84
3.00	*299.33*	*245.42*	209.21



3.1.Stress strain curves for 38mm specimen diameter of Aweytu test pit



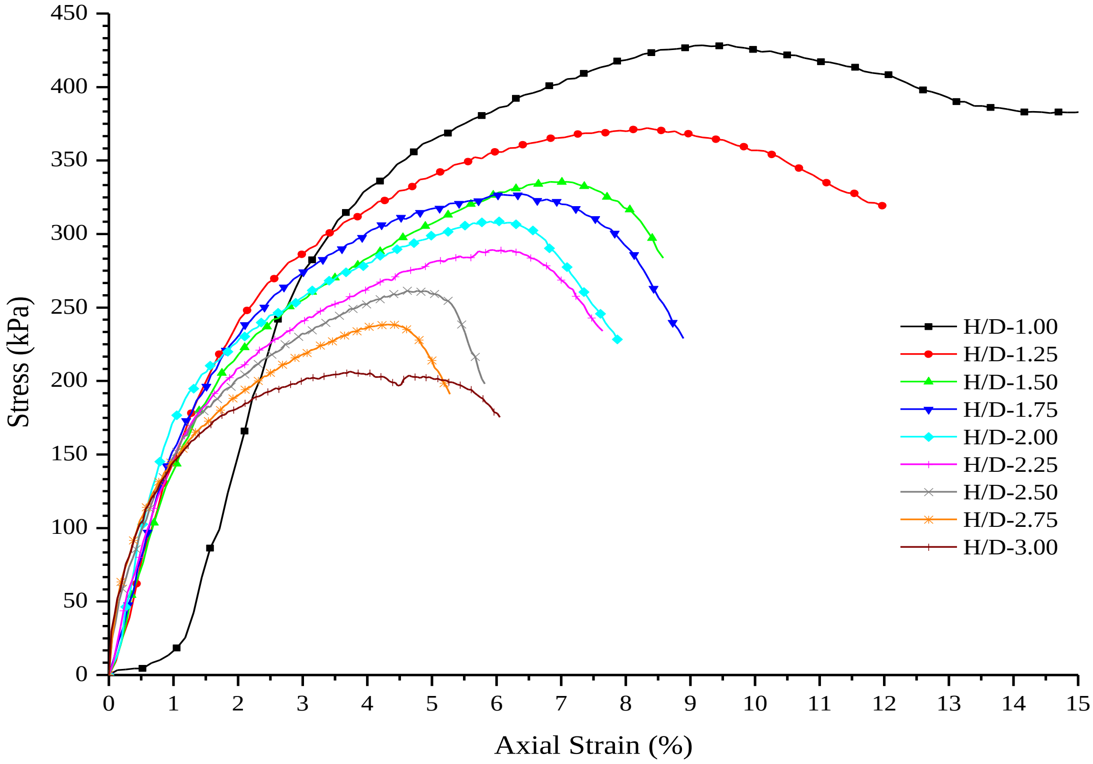
3.2.Stress strain curves for 50mm specimen diameter of Aweytu test pit



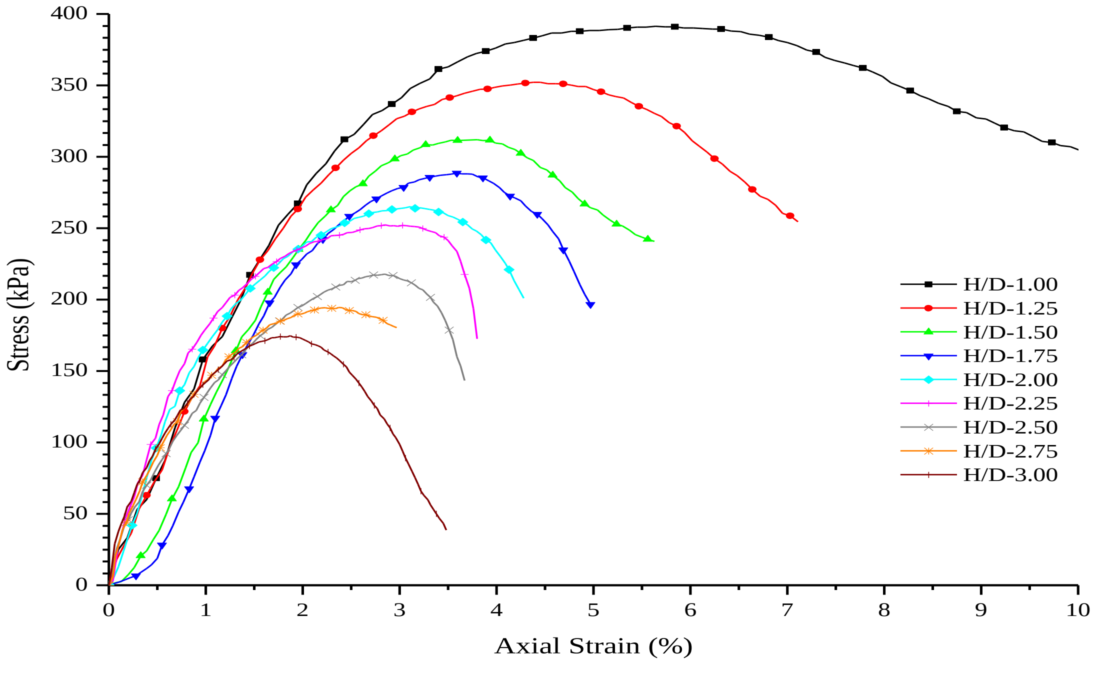
3.3.Stress strain curves for 100mm specimen diameter of Aweytu test pit

3.4.Table of peak UCS value (kPa) for each specimens' diameter of Aweytu test

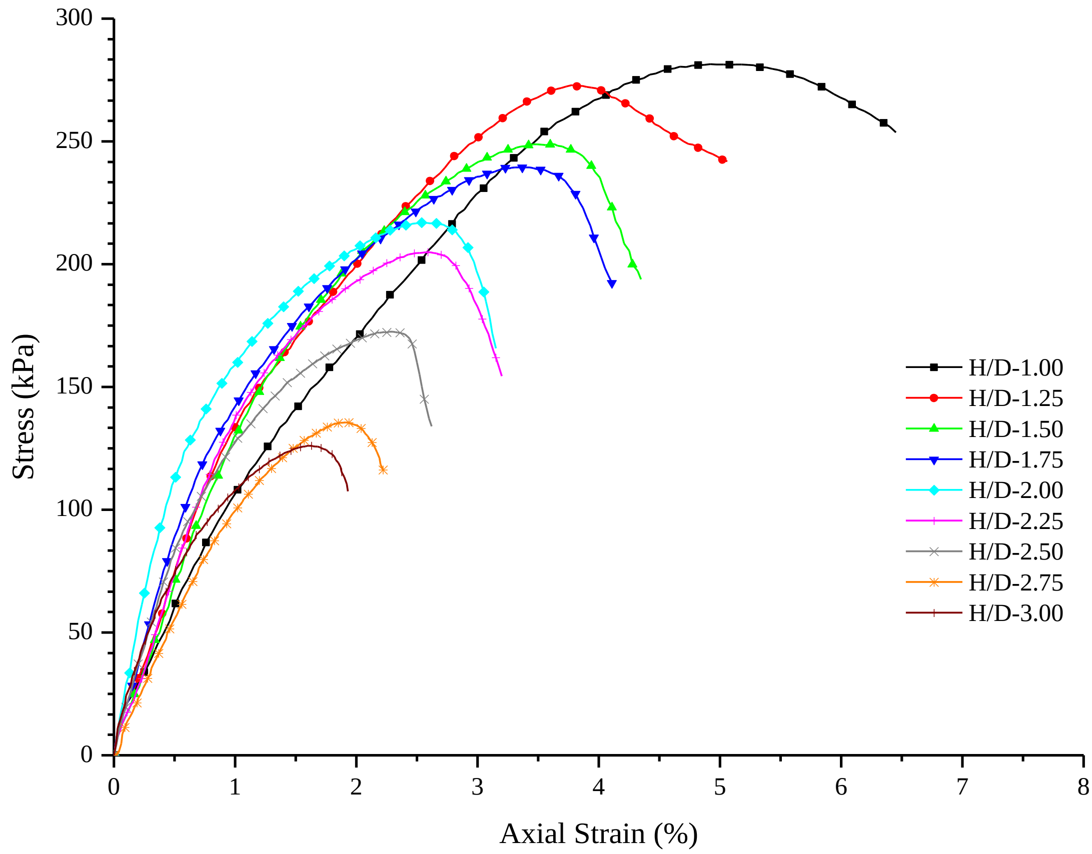
Height to diameter ratio	Specimen's diameter		
	38mm	50mm	100mm
1.00	474.49	396.40	321.42
1.25	409.74	360.47	309.91
1.50	359.69	306.15	290.31
1.75	348.01	291.45	259.04
2.00	321.97	274.00	245.71
2.25	292.62	265.36	229.15
2.50	277.62	248.64	226.11
2.75	266.13	221.22	198.60
3.00	238.07	203.80	181.44



4.1.Stress strain curves for 38mm specimen diameter of Saris Sefer test pit



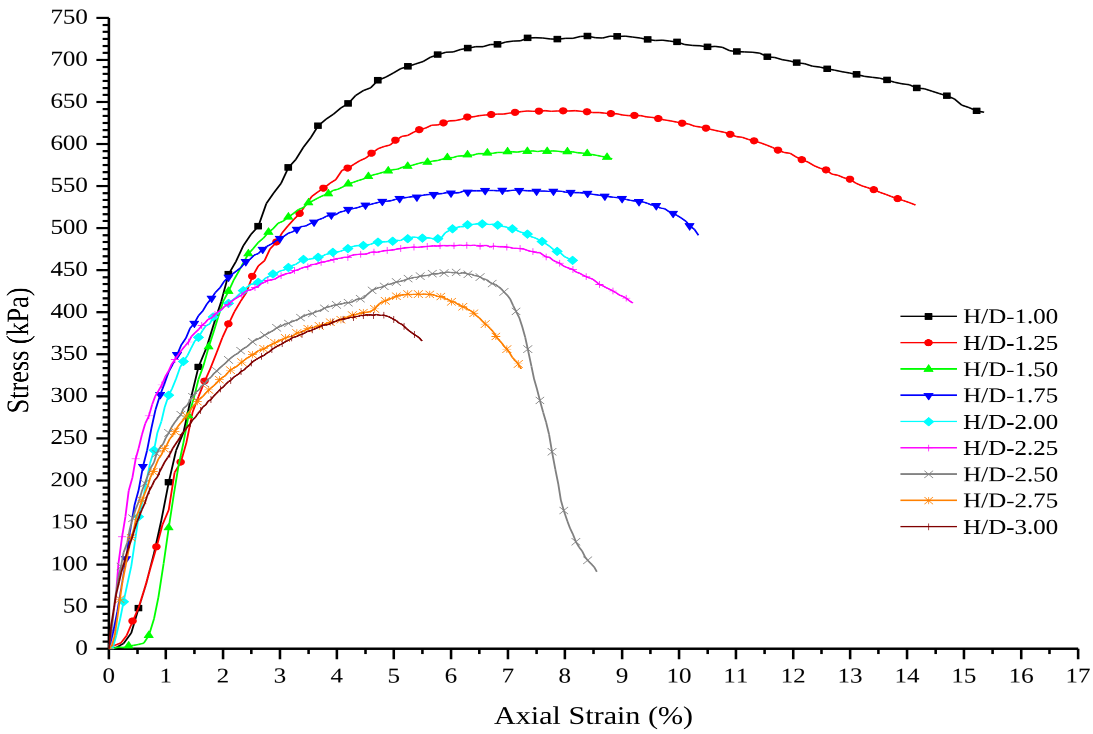
4.2.Stress strain curves for 50mm specimen diameter of Saris Sefer test pit



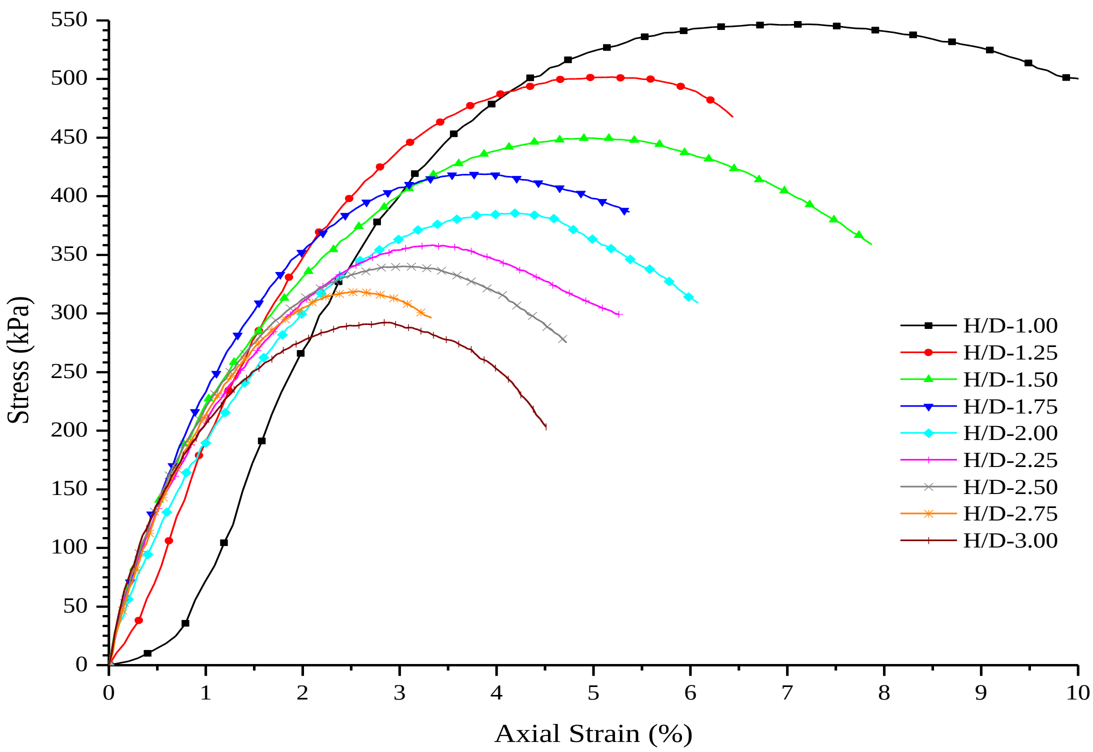
4.3.Stress strain curves for 100mm specimen diameter of Saris Sefer test pit

4.4.Table of peak UCS value (kPa) for each specimens' diameter of Saris Sefer test

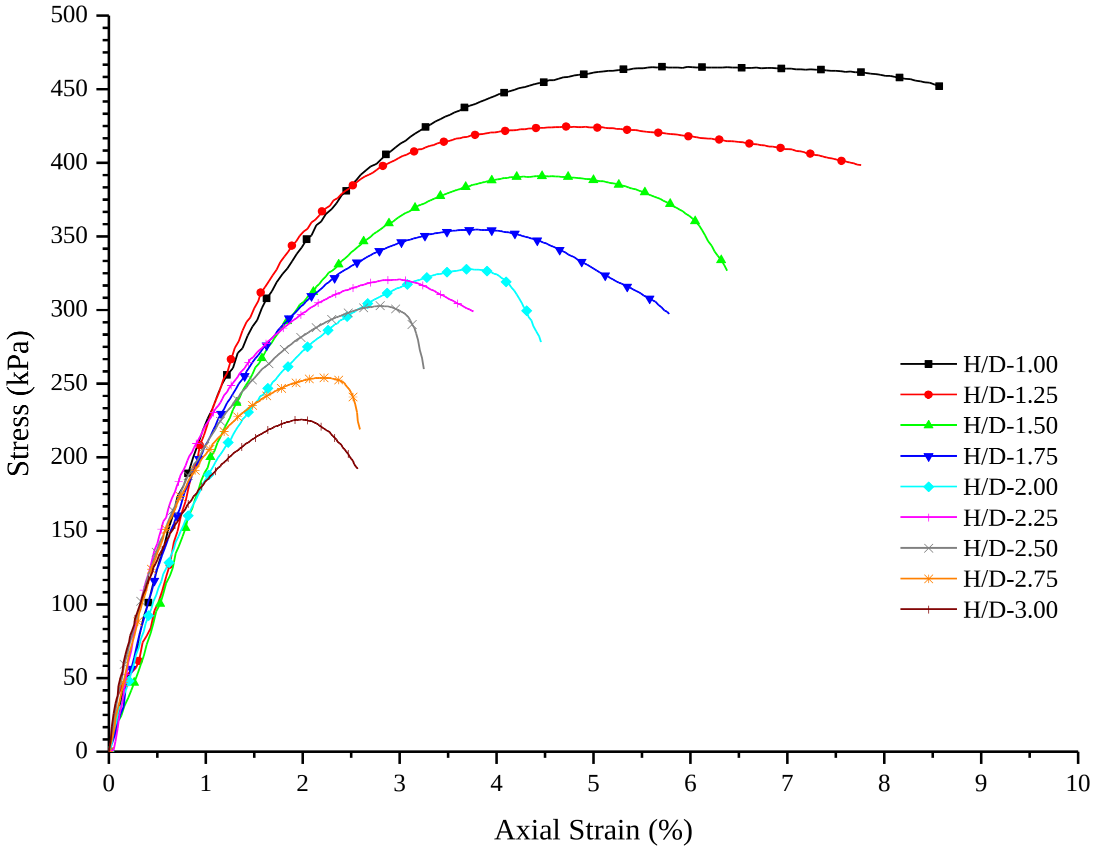
Height to diameter ratio	Specimen's diameter		
	38mm	50mm	100mm
1.00	428.74	391.34	281.43
1.25	372.16	352.22	272.95
1.50	335.57	311.93	248.93
1.75	327.13	288.63	239.48
2.00	308.69	265.02	216.96
2.25	289.24	252.35	204.87
2.50	261.36	217.89	172.55
2.75	238.42	194.48	135.58
3.00	206.53	174.75	126.06



5.1.Stress strain curves for 38mm specimen diameter of JiT Campus test pit



5.2.Stress strain curves for 50mm specimen diameter of JiT Campus test pit



5.3.Stress strain curves for 100mm specimen diameter of JiT Campus test pit

5.4.Table of peak UCS value (kPa) for each specimens' diameter of JiT Campus test

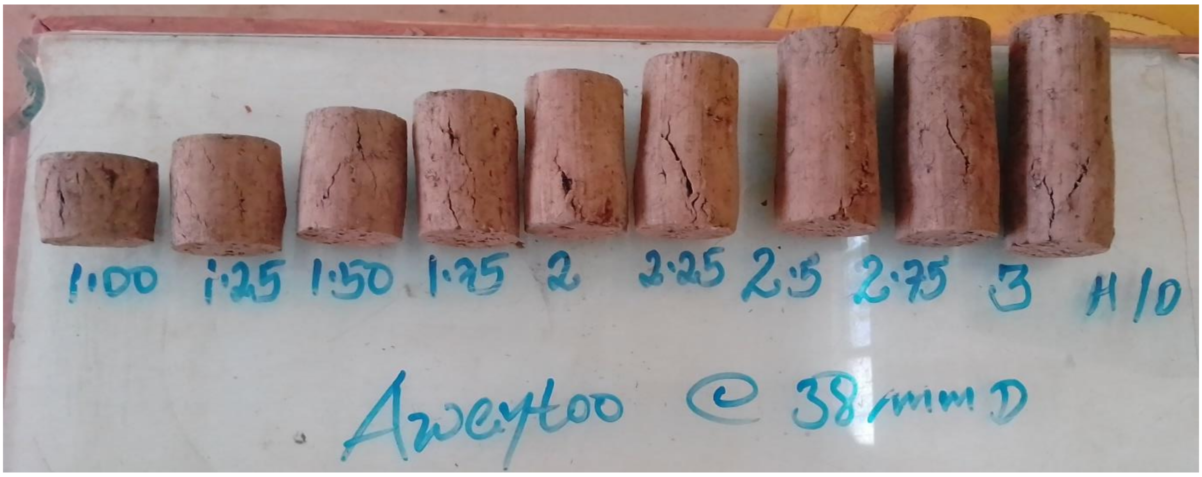
Height to diameter ratio	Specimen's diameter		
	38mm	50mm	100mm
1.00	728.32	546.82	465.21
1.25	639.82	501.83	424.58
1.50	592.01	449.63	390.98
1.75	545.29	418.91	354.64
2.00	505.55	385.46	327.63
2.25	479.84	358.27	320.76
2.50	447.71	340.14	302.84
2.75	421.82	318.95	254.00
3.00	397.37	292.35	225.62

C. Failure pattern of cohesive soil specimens

1. 38mm specimen diameter

1.1.Failure patterns of 38mm diameter specimens Mariam church test pit

1.2.Failure patterns of 38mm diameter specimens Ajip test pit



1.3.Failure patterns of 38mm diameter specimens Aweytu test pit

1.4.Failure patterns of 38mm diameter specimens Saris Sefer test pit

1.5.Failure patterns of 38mm diameter specimens JiT Campus test pit

2. 50mm specimen diameter

2.1.Failure patterns of 50mm diameter specimens Mariam church test pit

2.2.Failure patterns of 50mm diameter specimens Ajip test pit

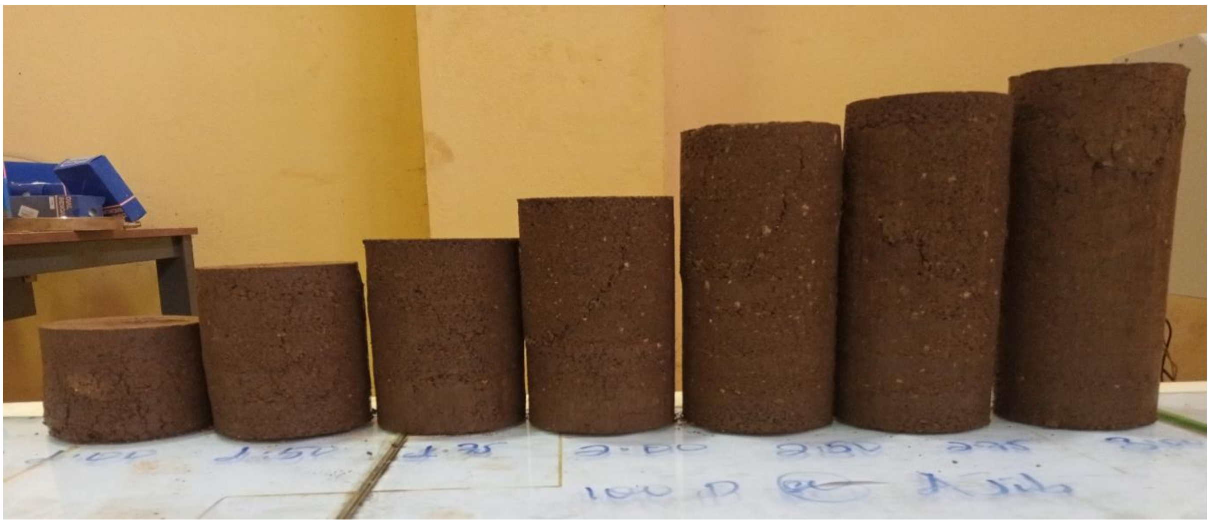
2.3.Failure patterns of 50mm diameter specimens Aweytu test pit

2.4.Failure patterns of 50mm diameter specimens Saris Sefer test pit

2.5.Failure patterns of 50mm diameter specimens JiT Campus test pit

3. 100mm specimen diameter

3.1.Failure patterns of 100mm diameter specimens Mariam church test pit



3.2.Failure patterns of 100mm diameter specimens Ajip test pit

3.3.Failure patterns of 100mm diameter specimens Aweytu test pit

3.4.Failure patterns of 100mm diameter specimens Saris Sefer test pit

3.5.Failure patterns of 100mm diameter specimens JiT Campus test pit

V. Photos of Test Produsers

Air drying of sample specimen and preventing of soaked sample specimens from evaporation before compaction

Compacting and extruding of sample specimen

Storing the extruded specimens, test the specimen and recording the specimen's deformation and load resisting reading.