JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

School of Civil and Environmental Engineering

School of Post Graduate Studies

Geotechnical Engineering Stream

INVESTIGATION OF GEOTECHNICAL CHARACTERISTICS AND SLOPE STABILITY ANALYSIS OF LANDSLIDE: THE CASE OF LALISA VILLAGE, TIRO AFATA DISTRICT, JIMMA ZONE.

A Thesis submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Geotechnical Engineering)

By: Narobika Tesema

November, 2018

Jimma, Ethiopia

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By: Narobika Tesema

Main Advisor: Dr. Ing. Fekadu Fufa

Co-advisor: Damtew Tsige (PhD fellow)

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Jimma, Ethiopia

APPROVAL SHEET

I, the undersigned certify that the thesis entitled: ''Investigation of Geotechnical Characteristics and Slope Stability Analysis of Landslide, the Case of Lalisa Village, Tiro Afata district, Jimma zone, Oromia'' is the work of Narobika Tesema and has been accepted and submitted for examination with my approval as University advisor in partial fulfillment of the requirements for Degree of Masters of Science in Geotechnical Engineering.

	Name	Signature	Date
1	Main Advisor: Dr. Ing. Fekadu Fufa		14/03/11
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As member of Board of Examiners of the MSc Thesis Open Defense Examination, we certify that we have read, evaluated the thesis prepared by Narobika Tesema and examined the candidate. We recommended that the thesis could be accepted as fulfilling the thesis requirement for the Degree of Masters of Science in Geotechnical Engineering.

1	[External Examiner]	[Signature]	[Date]
2	[Internal Examiner]	[Signature]	[Date]
3	[Chair Person]	[Signature]	[Date]

DECLARATION

I, the undersigned certify that the thesis entitled: ‘’Investigation of Geotechnical Characteristics and Slope Stability Analysis of Landslide, the Case of Lalisa Village, Tiro Afata district, Jimma zone, Oromia’’ is my own original work and that it has not been presented and will not be presented by me to any other University for similar or any other degree award.

Narobika Tesema: Signature ________________________ Date_______________________

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Abstract

Landslide is the most damaging natural disaster at global and local level. Landslide could cause damage of farm and grazing land due to ground subsidence, large cracks and soil mass movement. Livelihood, ecological and human health disturbance and destroy animal life are another impact of the study area landslide.

Recently, landslide has occurred in Lalisa village, Tiro Afata District, Jimma Zone, Oromia. Therefore, this study particularly aimed at identifying geotechnical characteristics, type of soil and their role in landslide initiation, slope stability analysis for those not failed by taking the samples within a certain distance from the failed soil to identify causes for the happened landslide in Lalisa village. The approached methods were field work, laboratory and software analysis. Measurement of slope geometry and landslide size, geophysical resistivity to know type of soil, level of ground water and thickness of weak zone without soil structure disturbance were conducted in the field. Laboratory tests using American Society Test Material standard were conducted to determine grain size, Atterberg limit, natural moisture content, shear strength and specific gravity. Slope stability was analyzed using Geostudio Slope/W. The results revealed that the soils were clay, which deform like a plastic material even semiliquid when in contact with excess water. This characteristics of soil were considered as sensitive for swelling and shrinkage when in contact with water and dries respectively and high intensity of rainfall increase the pore water pressure in the soil and decrease shear strength as a result slope failure.

The Geostudio Slope/W result shows that the factor of safety the slope less than unity even in modified slope angle, which was unsafe and this was due to the effect of slope steepness. The factor of safety for modified slope angle was higher when compared to natural slope. This could be due to the effect of steep slope. Additionally, the finding suggest that in case of modified slope gradient and increase distance from failure surface the factor of safety increased by by 47 %. These reveals that making slope gentle and free from cultivation within a certain distance are used as a prevention methods of the landslide of study area.

Keywords

Lalisa village: Investigation of Geotechnical characteristics, Landslide, Slope Stability Analysis

Table of Contents

Contents **Page**

List of Tables

List of Figures

SYMBOLS, ABBREVIATIONS AND ACRONYMS

ASTM	American Society of Test Material
CH	Inorganic Fat clay of High Plasticity
CSS	Critical Slip Surfaces
CTP1	Crest Test Pit 1
CTP2	Crest Test Pit 2
CTP3	Crest Test Pit 3
Cu	Undrained Cohesion
DDR	Deformation Dial Reading
DEM	Digital Elevation Model
E	Easting
FOS	Factor of Safety
GIS	Geographical Information System
GPS	Geographical Positioning System
GWT	Ground Water Table
LDR	Load Dial Reading
LEM	Limit Equilibrium Method
LL	Liquid Limit
LI	Liquidity Index
MS	Microsoft
N	Northing
PL	Plastic Limit
PI	Plasticity Index
PWP	Pore Water Pressure
TTP1	Toe Test Pit 1
TTP2	Toe Test Pit 2
TTP3	Toe Test Pit 3
UU	Unconsolidated Undrained
w	Water Content

CHAPTER ONE

INTRODUCTION

1.1 Background

Landslide or slope movement is the movement of a mass of rock, debris or earth down a slope (Cruden, 1991). It characterizes all varieties of ground failure and down slope movement of earth material controlled by gravity. Glade and Crozier (2005) has stated that in natural system, landslides are one of the most significant natural hazards in many areas throughout the world and cause direct and indirect costs of enormous property damage. As stated by (Terlien, 1996), even though a small percentage of individual landslides are catastrophic, it is essentially causes high total economic losses due to slope instability (direct damage to agricultural land and infrastructure and indirect damage to economic activity) to be higher than due to other hazardous natural phenomena.

Landslides and landslide-generated ground failures are among the common geo-environmental hazards in many of the hilly and mountainous terrains of both the developed and developing world. (Bryant, 2005) points out that landslides are one of the most widespread natural hazards and cause billions of dollars in damages and thousands of deaths and injuries each year around the world. Classification of slope movement is according to type of movement, the type of material and the movement phase or state of activity (Dikau et al., 1996). Five principal types of movements (fall, topple, lateral spreading, slide and flow) are distinguished based on the geomorphology classification which is proposed by (Cruden and Varnes, 1996 ;1996) Intense rainfall, earthquake shaking, water level change, deforestation, storm waves, or rapid stream erosion that causes a rapid increase in shear stress or decrease in shear strength of slope-forming materials are triggering factors of landslide by external stimulus. Jacob and Weatherly (2003), points out that heavy rainfall in tropical and temperate climatic zones results different kinds of landslides, which are rainfall-triggered landslides. Soils with high clay content are known to swell when wet and shrink in dry weather and the swelling capacity increases with increasing surface area of the clay particles and with decreasing valence of the exchangeable cations (Krhoda, 2013). As stated by Temple and Rapp (1972), deforested areas and support loss from the root system are susceptible to landslide.

In order to reduce the rate of landslide activity it is important to design proper control works. As stated by Webster's New World Dictionary (1988), retaining wall is a wall built to keep a bank of earth from sliding or water from flooding. According to Bromhead (1997), removal of all or part of the earth driving landslide to modify slope geometry which is the most efficient way of increasing the factor of safety of a slope. Drainage is the most widely used method for slope stabilization. Surface and subsurface drainage system should have the capacity to decrease pore water pressure at the failure surface in order to stabilize the landslide as much as possible. Hutchinson (1977), has indicated that drainage is the principal measure used in the repair of landslides. Not only these vegetation also has the role in stabilizing landslides by reducing pore water pressure through evapo-transpiration, by reducing infiltration and by binding the soil at shallow depth with its root system. Landslides occur when the slope changes from a stable to an unstable condition. As a number of factors acting together or alone that cause slope instability; the study will be conducted by fieldwork and soil laboratory experiment on the affected area by landslide.

The landslide in Ethiopia, Oromia, Jimma Zone, Tiro Afata District, Lalisa village was the recent occurred landslide study area. As landslide in the study area, investigation of geotechnical properties, soil type and their initial role in its occurrence was studied, effect faced the community living around due to this landslide observed, prevention and mitigation measures was also proposed.

1.2 Statement of the Problem

Landslides are one of the most significant natural hazards throughout the world and it results enormous property damage. According to UNISDR 2015; Okuyama and Sahin (2009), landslides are the cause for livelihoods disturbance, loss of human lives and damages to properties and infrastructure estimated to cost around 250 billion dollar worldwide each year. (Kitutu et al., 2011; Msilimba, 2009; Mugagga et al.,2010) suggests that landslides in East Africa highly affect smallholder farmers' income through the loss of houses, crops and soil fertility. As stated by Ayalew (1999), in between 1993 and 1998 in Ethiopia landslide destroys over 200 houses, interrupts more than 500 Km roads and kills around 300 people. Many varieties of landslides occur because of heavy rainfall in tropical and temperate climatic zones (Jacob and Weatherly, 2003). Krhoda (2013) has suggested that Soils with high clay content are known to swell when wet and shrink in dry weather and the swelling capacity increases with increasing surface area of the clay particles and with decreasing valence of the exchangeable cat ions which enhance propagation of landslides. Deforestation, which results temperature change and heavy cracking of soils, is another cause for landslide occurrence. According to National Meteorological Agency of Ethiopia (2009), mean annual rainfall in Jimma zone is ca. 2000 mm, heavy rainfall and it is the main triggering factor for almost all landslides. The high annual rainfall causes saturation of the soil, and positive pore pressure in the soil during the rainy season, which can then serve as a catalyst to allow other causal factors to act more effectively in causing a landslide. Hence, it was important to do investigation of geotechnical properties using laboratory test, measuring geometry of the slope, ground water table level, information on flooding and deforestation using interview as a primary data source and obtaining rainfall data, using secondary data source to improve their effect on the slope stability of the study area.

Lalisa village, Tiro Afata, Jimma zone, South Western part of Ethiopia about 80 km right hand side from Asandabo via Addis Ababa to Jimma main road was the study area affected by landslide. The study area was bare land no vegetation coverage and it was farm and grazing land with minimum areas covered by banana and sugarcane. Due to landslide at this area, about 104 ha of farm and grazing land become useless, livelihood and human health disturbance, animal life loss, damage banana and sugarcane about 0.81 ha. In relation to this, the aim of the research geotechnical properties role in landslide occurrence of the study area was investigated, slope stability was analyzed, it's effect, prevention and mitigation measures of the affected area was proposed.

1.3 Research Questions

1. What are the soil types, characteristics and their initiation for landslide occurrence of the study area?
2. Under which type of landslide the study area categorized.
3. What are the causes and triggering factors, consequences and remedial measures of existing landslide in the study area?
4. What is the condition of the slope from slope stability analysis?

1.4 Objective

1.4.1 General Objective

The general objective of the study is to investigate geotechnical characteristics and analysis of slope stability of landslide in Tiro Afata district.

1.4.2 Specific Objectives

The specific objectives of the study are:

1. To determine soil types, characteristics and their role in landslide occurrence in the study area;
2. To investigate the consequences, causes and triggering factors of the landslide of the study area and
3. To conduct slope stability analysis using numerical method, determine slope condition and propose the prevention or mitigation measures.

1.5 Significance of the Study

From this study information about soils type, characteristics and their role in landslide occurred in the study area, soil strength and factor of safety are carried out, consequence of landslide and factors that cause downward soil mass movement was identified and possible prevention and remedial measures also proposed. Besides, It can use as information source regarding effect of landslide on natural environment and reference for other researchers of further research on causes and mitigation of landslide and slope stability analysis.

1.6 Scope of the Study

In order to address the aforementioned purposes, six test pits were excavated, disturbed and undisturbed within a depth of 1 to 3 m were taken at different points of crest and toe of the slope. In addition to laboratory test (Atterberg limit, grain size distribution, specific gravity, density and natural moisture content, free swell and shear strength test and field test (geophysical test), literature reviews, visual observation and information from farmers were used. Software used for analysis were Arc GIS10.4.1, Geostudio12, Res2dinvx32 and WinResist.

1.7 Organization of the Study

This study was divided in to five chapters, each covering the specific topic of the study work. In this introductory chapter the background of the problem, statement of the problem, objective, research questions, limitations and scope of the study are presented. Chapter two deals with a detail literature review. Chapter three deals with material and methods used for the study, chapter four contain result and discussion gained from laboratory and field test, visual observation and software result. The last chapter was conclusion and recommendations drawn from the study. Reference comes next to conclusion and recommendation, at the end Tables and Figures of laboratory result and standard are included in appendices.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Landslide or slope movement is the movement of a mass of rock, debris or earth down a slope (Cruden 1991). Landslides or slope failures are a complex natural phenomenon that constitutes a serious natural hazard in many countries (Brabb and Harrod, 1989). The term slope movement or landslide characterizes all varieties of ground failure and down slope movement of earth material controlled by gravity.

2.2 Landslides Contributory Factors

Landslide or slope instability is due to the combination of internal changes that leads a decrease in shear resistance and external changes that results increase in shear stress see its summary in (Table 2.1). The factors that makes the slopes exposed to failure are causes of landslides, which directs the slope to becoming unstable. The term "landslide causes" is often used for long term processes leading to slope instabilities (Sowers, 1979). Knowing the factors contributing to vulnerability is the main effective for prevention and mitigation of landslide. The following factors are some landslide causes considered in literature review.

2.2.1 Slope Steepness

Relief is a principal factor in the determination of the intensity and character of landslides. Slope steepness is the main characteristic of relief that affects the mechanism as well as the intensity of the landslide. The greater the volumes of the landslides is because of the greater the height, steepness and convexity of slopes. The stability of the slope against sliding is defined by the relationship between the shear forces and the resistance to shear. As stated by UNESCO/UNEP (1988), gravity is the force that acts everywhere on the earth's surface, pulling everything in a direction toward the center of the earth. On a flat surface the force of gravity acts downward and on a slope, the force of gravity can be resolved in to perpendicular to the slope and another tangential to the slope. Thus, the perpendicular component hold the object in place on the slope but the tangential component causes a shear stress parallel to the slope that pulls the object in the down-slope direction parallel to the slope. On a steeper slope, the shear stress or tangential component of gravity increases and the perpendicular component of gravity decreases. The forces resisting movement down the slope are grouped under the term shear strength which includes frictional resistance and cohesion among the particles that make up the object. When the shear stress becomes greater than the shear strength and a critical angle is exceeded, the slope fails.

2.2.2 Periodical Swell-Shrink of Soil Behavior

As stated by Nelson and Miller (1992), dual characteristics of swelling and shrinkage under different moisture conditions of expansive soils cause deformation. According to Thomas (1939), downslope movement as well as deep weathering of soil is because of periodical swell-shrink soil behavior. Because of their cyclic swelling and shrinkage behavior during wet and dry seasons respectively, expansive soils are highly problematic, causing subsidence of soil mass as well as landslide. Clay soils have high percentage of swell-shrink potential and plasticity because of volume changes as moisture condition variation. The soil dries out during summer and it undergoes shrinkage that creates large and small cracks. This allows entrance of atmospheric gases and high infiltration or penetration of rain water into lower soil horizons during winter. Mixing of soils is due to gases while, swelling increase in clay soil when it contact with moisture that results downslope movement.

2.2.3 Geological Factors

Highly weathered rocks are composed of extremely fine pyro lasts of potash feldspar and are referred to as potash ultra-finites (Reedman, 1973). Fault zones that obviously contain fractured and crushed rocks is the reason for deep percolation of water into bedrock and subsequent weathering of crushed rocks into clay rich soil susceptible to failures. The water acting as lubricant then triggered a huge rock and soil slide. The weakness in the soil mass and rock can be enlarged by deep percolating water into the bedrock and subsequent chemical weathering of the rocks and soils. As stated by Sidle et al. (1985), the existence of a relatively impermeable layer in the bedrock that develops the formation of a perched water table during storms or heavy rainfall thus, increasing the probability for slope failures. A clayey layer in between massive rocks makes the whole mass susceptible to sliding (UNESCO/UNEP, 1988). Down slope dipping planes separating rocks of different competence, as well as planar discontinuities orientated parallel to the slope, may impede vertical drainage, root penetration and may act as a potential failure plane. Accumulation of water at some discontinuity point is because of interruption of drainage of water through the soil profile at point of discontinuity during rains.

2.2.4 Soil Type

Soils such as silt and clay are weaker and commonly have multiple planes of weakness which enhance the occurrence of landslides. Krhoda (2013), states that soils with high clay content are known to swell when wet and shrink in dry weather and the swelling capacity increases with increasing surface area of the clay particles. Sidle (1985), observed that high clay content in deeper soils may increase the water holding capacity and causes for slow or more rapid failure. The situation for clay soil is a simple reduction of strength as moisture content increases. Grain size distribution, percent fraction of coarse material, density, organic inclusions, stratification, and thickness of soil (loose) are the soil parameters influencing the interaction of soil and water. Moser (1980); Andrecs et al. (2007), states that soils with high quantities of fine grained material (silt, clay > 40 %) are prone to land sliding.

2.2.5 Human Activities

According to Dumitrascu (2006), agriculture is human activities that affect environment due to agricultural land use, chemical consumption, triggering land degradation and incorrect operated exploitation of farm land thus initiate landslide.

2.2.5.1 Deforestation for Land Use

Widespread removal of vegetation for landuse is human activity which leads to an increase in slope failures. As stated by Inganga et al. (2001), Rappet al. (1972); Nyssen et al. (2002); Davies (1996), deforestation is the main factors for landslides in most East African highlands. In the long run, cutting of trees on slopes leads to a gradual decrease in mass stability as a result of the decay of roots which previously acted as tensile reinforcements in the slope. According to Sidle et al. (2006), the rooting strength of sloping land reduced up to two decades with subsequent regeneration due to removal of forests. Conversion from trees to crops can reduce the strength of root and its depth as well as deep soils are also dried to a lesser depth and degree thus increasing landslide. According to Temple and Rapp (1972), landslides are hence commonly attributed to the loss of support from a root system and deforested areas. The removal of vegetation canopy results in the loss of interception and evapo-transpiration which tends to promote wetter and less secure slopes. Deforested areas also experience wide variations in temperatures and moisture contents thus causing heavy cracking of soils that weakens soil shearstrength and increase landslide.

Table 2.1 Summary of Landslide Causes (Call, 1992 and Msilimba, 2002)

External Causes	Internal Causes
Geometrical change • Height • Gradient • Slope length	Progressive Failure (internal response to unloading) • Expansion and swelling • Fissuring • Straining, softening • Stress concentration
Loading • Natural • Man-induced	Weathering • Physical property changes • Chemical changes
Unloading • Natural • Man-induced	Seepage erosion • Removal of cements • Removal of fine particles
Shocks and Vibrations • Single • Multiple/continuous	Water regime change • Saturation • Rise in water table • Excess pressures • Draw down

2.3 Triggering Factors of Landslide

A triggering factor is an external stimulus that triggers the movement and it is a sudden event such as an earthquake, volcanic eruption, water-level change, intense rainfall, rapid snowmelt (Wieczorek, 1996). The following are the main triggering factors of landslide:

2.3.1 Rainfall

Rainfall is the main factor in triggering landslides as it saturates the soil that have a great influence on cohesion, strength and viscosity of soil materials hence it affects the stability of slopes in such a way that decreasing suction, increase positive pore water pressure and induce seepage forces that are the causes for reduction of soil shear strength. Jacob and Weatherly (2003), points out that heavy rainfall in tropical and temperate climatic zones results different kinds of landslides which are rainfall-triggered landslides. A higher rainfall intensity leads soil saturation and increment of ground water table that generate landslides. Rainfall is one of those factors that have been found to trigger landslides due to high rainfall events result in high water saturation in soils reducing the strength of the soil. UNESCO/UNEP (1988), reported that a higher moisture content can increase the specific mass of rocks by 20 to 30 % and at the same time lower their shear resistance by 50 % and even more, due to increased pore-water pressure. This condition prominently reduces shear strength and hence slope failure. Increase in water content increases pore water pressure. Even though the effect of rainfall is more complicated, landslides are more common when rainfall is continuous and soil resistance is exceeded (Ayalew, 1999). Landslides are more likely happen when heavy rainfall preceded by ceaseless rainfall. The conditions of rainfall determine infiltration and run-off. A continuous rainfall with a lower intensity result in a higher and deeper infiltration and lower run-off in sloping areas. When the rainwater reaches the ground it starts to infiltrate, pore water pressure rises, and because of the loss of cohesion of the solid particles, their weight become increase and then results slope instability.

2.3.2 Ground Water Fluctuation

Ground water fluctuation is due to seasonal increase and decrease rainfall intensity. Decrease in intensity of rainfall cause lowering of ground water table and also decrease in weight of soil mass. But, when the intensity of rainfall increase the level of ground water table increase as well as erosion and weight of soil mass that results reduction in shear strength and slope instability. As stated by Caris and van (1991), landslide activity in the weathered marl layer and the temporal occurrence of a perched water table in the top moraine deposits is a consequence of high groundwater levels.

2.4 Classification of Landslide

Landslide classification is done according to the type of movement, material and the movement phase or activity (Dikau et al., 1996). According to Petley (2012), types of landslides involve falling, toppling, lateral spreading, sliding, flowing and during the lifetime of the landslide, a combination of different types of movements occurred alternatively in many landslides. Below are various types of landslides that can be distinguished by their mode of movement.

2.4.1 Falls

Falls are triggered by earthquakes or erosion processes and characterized by a rapid to extremely rapid rate of movement with the descent of material characterized by a free fall period. Falls in soils or soft rocks usually involve only small quantities of material because steep slopes in weak weathered materials are necessarily very short. As stated by Selby (1987), falls are usually due to undercutting of the toe or face of the slope by a river or by wave action.

2.4.2 Topples

A topple is the tilting of rock without collapse, or by forward rotation of rocks about a pivot point with the rapid rate of movement, and the failure is generally influenced by the fracture pattern in rock by abrupt falling, sliding, bouncing and rolling. Topples include the outward rotation of angular blocks or rock columns that become detached from cliffs (Crozier, 1984; Alexander, 1993).

2.4.3 Slumps

According to Moore et al. (1989), slumps are slow-moving, wide up to 110 km and have a thickness about 10 km, with transverse blocky ridges and steep toes. Slumps may be caused by currents in water undercutting the toe of a slope and embankment cut due to engineering design faulty. Slumps have curved failure planes and involve rotational movement.

2.4.4 Slides

A slide is a down slope movement of a soil or rock mass occurring on surfaces of rupture or on relatively thin zones of intense shear strain. Translational and rotational slides are two different kinds of slide. Alexander (1993); and Smith (1996), suggested that translational slides are relatively flat, planar movements along surfaces and they have pre-existing slide planes that are activated during the slide event while, rotational slides have a curved surface rupture and produce slumps by backward slippage.

2.4.5 Flow

Flow is form of rapid mass movement in which loose soil; rock and sometimes organic matter combine with water form lose cohesion and slurry that flows in down slope direction which progress destructive and turbulent form of landslide. According to Johnson and Rodine (1986); and Bryant (1991), flow is happened during the structure of the material changed into quasi – fluid.

Table 2.2 Classification of Landslide (Varnes, 1978)

Type of movement			Type of material		
			Bedrock	Engineering soils	
				Predominantly coarse	Predominantly fine
Falls			Rock fall	Debris fall	Earth fall
Topples			Rock topple	Debris topple	Earth topple
Slides	Rotational	Few units	Rock slump	Debris slump	Earth slump
	Translational	Many units	Rock slide	Debris slide	Earth slide
Lateral spreads			Rock spread	Debris spread	Earth spread
Flows			Rock flow	Debris flow	Earth flow
Complex			Combination of two or more principle type of movements		

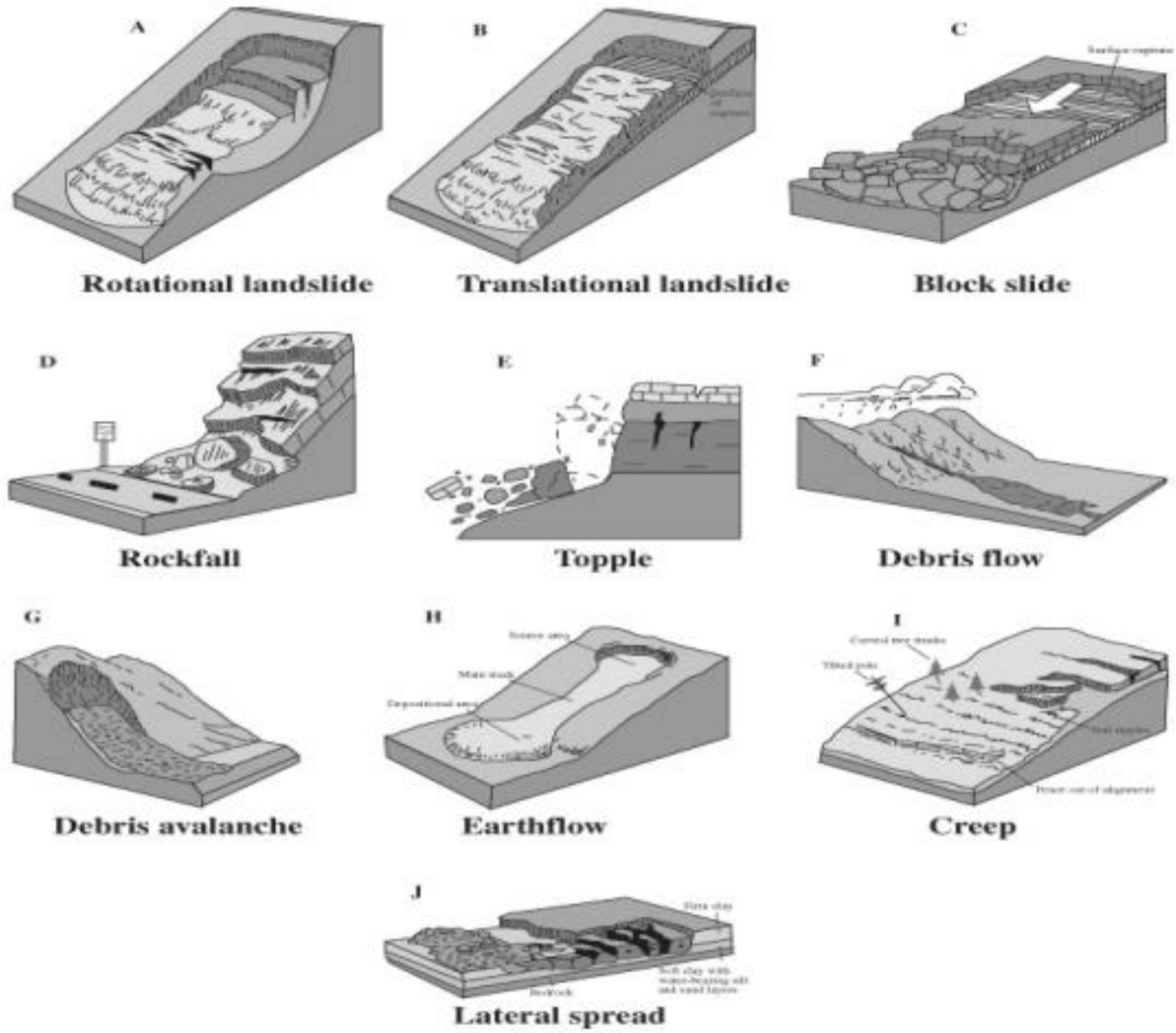


Figure 2.1 Schematic Representation of Landslide Types

2.5 Landslide Mechanics

Slope instability is the situation which contributes rise to slope failure (Alexander, 1993). In every slope there are shear stress which tend to encourage movement, and shear strength that resist movement. Sliding occurs when the resisting forces are exceeded by driving forces. The equilibrium can be disturbed by stress increments or the weakening of frictional force. Below are a discussion of different mechanics of landslide group.

2.5.1 Stress and Strain

Any mass on an inclined plane is influenced by gravity and the size of the gravitational force acting along the slope is directly related to the slope angle. According to Bryant (1991); and Alexander (1993), the influence of shear stress upon the soil is strain. As stated by Butler (1976); and Crozier (1989), strain may not occur homogenously in the soil body due to its restriction to joints where fracturing eventually occur. The ratio of resistance to shear stress provides factor of safety and the higher values of factor of safety indicates more slope stability situation.

2.5.2 Shear Strength of Soil

The strength of soil depends on the inner cohesion of the particles and internal friction or shearing resistance between individual soils grain (Bryant, 1991). When the base of the slope found to be significantly weathered, it contributes low shear strength and then slope instability (Gondwe and Govati, 1991; and Msilimba, 2002).

2.5.3 Pore Water Pressure

As stated by (Alexander, 1993; and Knapen et al., 2006), when extra an external load is applied to the soil mass on a slope in the form of water, or overburden, the pore water pressure will develop in the soil mass and water will be expelled at weak points. The development of pore water pressure results effective normal stress and shear strength of the soil mass reduction (Bryant, 1991).

2.5.4 Effects of Joints and Rock Structures on Slope Stability

Joints, fractures, spacing and rock structures have effect for slope stability (Butler, 1976; Hencher, 1987; Byrant, 1991; and Nicholas, 1995) as described below: pore spaces between particles allow water to pass by dissolving the bonding material and weathering the rocks; intersecting sets of parallel joints allow rocks to break into smaller masses that move more easily down slope; contact surfaces between beds of rocks that have different characteristics are points of weaknesses along which rocks can break; and tectonic pressures can re-orient rocks after their formation. Such dipping layers facilitate mass movement where they are inclined in the same direction as the slope of the land.

2.6 Geotechnical Investigation

Geotechnical investigation is the determination of geotechnical parameters for stability analysis and geotechnical classification using geotechnical stability analysis and classification test.

2.7 Geotechnical Classification test

According to (Ali, 2011), in relation to landslide behavior grain size distributions, specific gravity, fine grained lithological units type and characteristics, water content and consistency limits should be carried out to classify soil. Thus, this study also presents some of the geotechnical laboratory data such as particle size distribution and water content data conducted at certain selected depth of the borehole soil sample.

2.8 Geotechnical Stability Analysis

Bulk density, falling head permeability and shear strength test are laboratory tests used to determine slope stability analysis parameter such as unit weight, saturated unit weight, dry unit weight, hydraulic conductivity, cohesion and friction angle of the soil.

2.9 Field Investigation

2.9.1 Electrical Resistivity

Electrical resistivity is a geophysical survey which measures how much the flow of electricity resisted by the soil. Electrical resistivity is used to identify the condition of natural slope subsurface profile, the information on the variation of the resistivity anomaly pattern can be expected due to the contrast between geo-materials and water content present under the profile. According to Kim et al., (2010), electrical resistivity images a change of apparent resistivity with depth locally and is able to detect the water-saturated clay, which identified as a lower resistivity zone. Soil type, moisture content and temperature are the factors those affect electrical resistivity value of the soils. Resistivity value changes from low to high as the material is decreasing in water content. A high resistivity value has low conductivity which indicates that the soil is high in strength and vice versa (Figure 2.1). The resistivity values are different within soil type (Figure 2.1). Electrical resistivity or electrical conductivity of soils can be measured directly from soil surface to any depth without soil disturbance using two electrical resistivity methods of electrical profiling (EP) and vertical electrical sounding (VES). Schlumberger and Wenner array are used to conduct vertical electrical sounding and electrical Profiling or imaging respectively.

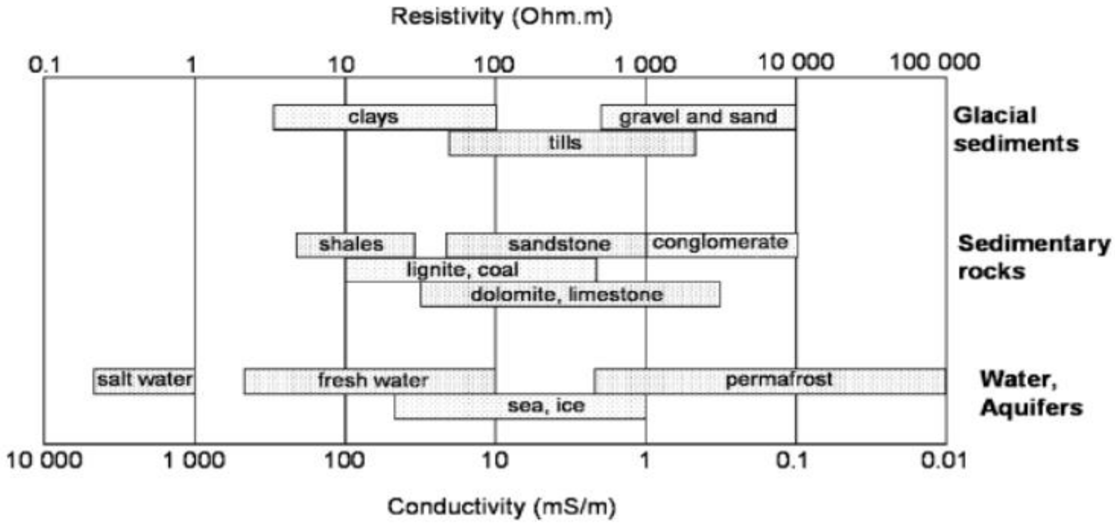


Figure 2.2 Typical Ranges of Electrical Resistivity of Earth Materials (Palacky, 1987)

2.9.1.1 Electrical Profiling (EP)

The electrical resistivity measured with this method is termed apparent electrical resistivity. The most common array used for electrical profiling is Wenner array and software used is RES2DINV. As stated by Loke and Barker (1996); Loke (1997), the purpose of RES2DINV software is to process, perform inversion of resistivity image profile data and calculate the apparent resistivity values.

2.9.1.2 Vertical Electrical Sounding (VES)

Vertical Electrical Sounding (VES) or 1D survey was conducted for the study of sub-surface soil. This method is based on the measurement of voltage of electric field that was induced by electrodes embedded in soil. Apparent resistivity values calculated from measured potential differences can be interpreted in terms of overburden thickness, water table depth, and the depths and thicknesses of subsurface strata. The most common array used for VES is Schlumberger array and it is performed for deeper imaging of subsurface profile which exhibits higher density data.

2.10 Slope Stability Analysis

Slope stability analysis deal with determination, investigation, modelling and design of natural and artificial rock and soil slopes. It also determines the factor of safety of the slope which indicates whether the slope is unstable, marginally stable and stable. A common method used to determine the slope stability for slip failure are limit equilibrium and finite element method.

2.10.1 Limit Equilibrium Method

Limit equilibrium method subdivide the landslide body into slices to calculate the forces for each slice and uses the principle that a slope is stable when driving forces exceeded by resisting forces and factor of safety, FOS is equal or larger than 1. According to Duncan and Wright (2005), the stress required to maintain just-stable slope is the equilibrium shear stress and a slope becomes unstable when shear strength exceeded and factor of safety, FOS becomes smaller than one due to (1) reduction of resisting forces and (2) increment of driving forces.

$$FOS = \frac{Resisting \quad forces}{Deriving \quad forces} = \frac{Shear \quad strength}{shear \quad stress}\frac{\tau_f}{\tau} \geq 1 \tag{2.1}$$

τ : is the equilibrium shear stress which depends on soils weight, pore water pressure and slope angle, τ_f : is the available shear strength which depends on the soils weight, cohesion, friction angle and pore water pressure.

The shear strength is the maximum shear stress which can be absorbed by the slope without failure and can be defined by Mohr-Coulomb criterion:

$\tau_f = c + \sigma \tan\phi$ Where c : cohesion, σ : normal stress at rupture surface and ϕ : angle of internal friction.

The main disadvantage of conventional LEM is that it requires pre-assumptions to complete the solution. Some of the well-known and widely used LEM methods are Bishop Method (1955), Fellenius method (1936) and Spencer method (1967). A summary of several limit equilibrium methods and their assumptions are presented in Table 2.3.

Table 2.3 Summary of Limit Equilibrium Methods (SLOPE/W 2004; Abramson et al., 2002)

Methods	ME	FE	Shape of Slip Surface	Interslice Normal E	Interslice Shear (T)	Assumption for T and E
Ordinary or Fellenius	Yes	No	Circular	No	No	No interslice forces

Bishop's Simplified	Yes	No	Circular	Yes	No	The side forces are horizontal
Janbu's Simplified	Yes	Yes	Any shape	Yes	Yes	The side forces are horizontal
Spencer	Yes	Yes	Any shape	Yes	Yes	Constant inclination T=tanθE
Morgenste rm Price	Yes	Yes	Any shape	Yes	Yes	Defined by $f(x), T = f(x).\lambda.E$

Where ME = Moment Equilibrium and FE = Force Equilibrium

2.11 Numerical Modeling

Numerical modelling of landslide is one of the possible approaches that can be used to simulate landslide instability and to estimate the slope risk level of the study area. Numerical modeling can be used to predict slope risk level and hazard zone (Hutter and Savage,1988; Sassa,1988;Hungr, 1995;Campbell, 1989;Iverson and Denlinger, 2001; Denlinger and Iverson, 2001; Rochet Bouzid, 2001).

2.12 Slope Classification

According to Misilimba (2007), three classes of slopes depending on degree of destabilizing forces are: (1) Stable slopes are those whose margin of stability is sufficiently high to withstand all destabilizing forces. (2) Marginally stable slopes are those that will fail at some time in response to the destabilizing forces attaining a critical level of activity, and (3) Active unstable slopes are those in which destabilizing forces produce continuous or intermittent movement.

2.13 Effects of Landslide

Landslides affect agricultural land by eroding and transporting fertile soils. According to Ngecu and Mathu (1999), in 1997 landslide makes the many hectares of arable land unproductive, and destroyed 25 hectares of tea bushes in Kenya. Landslides can also cause displacement, injury or death of people when their occurrence is close to populated areas, depriving societies and nations of the much required human resources. Effects of landslide in case of cost is described as below.

2.13.1 Landslide Related Costs

Personal cost, economic loss and environmental damage are the three effects of landslide considered. They can be immediate or long term.

2.13.2 Personal Costs

Death, injury, prolonged psychological and physical health problems are some personal costs. As stated by Crozier (1989), injury has long-term costs as it may not only reduce the social and productive role of an individual, but also may impose added costs for medical treatment and support.

2.13.3 Economic Costs

The direct economic costs of a given magnitude of landslide event depend on the nature of society and the type of landscape affected (Crozier, 1986). Landslide has direct damage to agricultural land and infrastructure and indirect costs such as mobilization and support of relief, and temporary or replacement of housing and supplying food; and preventative costs which are more difficult to assess, including the cost of research and costs of implementing preventative or control measures.

2.13.4 Environmental Costs

Large volume of slope movement affects environmental stability and results severe the slope and downstream (Crozier, 1989). (Crozier, 1986) states that habitat degradation, deranging of drainage system, alteration of drainage path ways, destruction of riparian vegetation, bank erosion within the stream channel, accelerated meander development and prevention of fish migration, and the loss of scenic beauty of mountainous are some environmental costs.

2.14 Landslide Prevention and Mitigation Measures

Mitigation measures for natural geological hazards aims at protecting people, property and infrastructure (Crozier, 1984). Knowledge of the nature, scale, distribution and causes of landslides are important for proposing mitigation measures of landslide. Landslide prevention and mitigation measures are activities that reduce or prevent landslide damage by providing restraining structures and/or increasing the internal strength of the soil thus, resisting slope movement. Landslide effect of the study area control and prevention, at all scales of activity can be through one or more of: avoid or remove the landslide problem, reduce the actuating forces, increase the resisting forces.

2.14.1 Afforestation Extending Over the Entire Ground Surface

Covering the bare land with vegetation prohibits mass movements in the area thus, preventing landslide. According to Sidle et al. (2006), deep-rooted trees and shrubs can reduce shallow landslide by reinforcing shallow soil layers and improving drainage see Figure 2. As stated by Guthrie et al. (2010), forests used as a barrier against sliding or movement of material and debris flows. Penetration of tree roots entire soil mantle enhance anchors into stable substrates; dense lateral root systems also stabilize soil surface layers against shallow landslide. As their tensile strength and adhesion properties, plant roots reinforce the soil and increase the confining stress and shear strength by offering additional apparent cohesion that increase the cohesiveness of the soil mass due to its closely spaced root matrix system.

2.14.2 Providing Drainage

Improving drainage has been noted as a very effective means of protecting unstable hill slopes from sliding (Pilot et al., 1988). Drainage is the most widely used method for slope stabilization. Surface water is drained from the unstable areas by surface ditches so as to reduce surface water infiltration into the potential slide mass. Surface drainage is used to divert water from flowing onto the slide area using collecting ditches. Hutchinson (1977), has indicated that drainage is the principal measure used in the repair of landslides. Ground fissures at the head of landslides should be closed by grading or ploughing to minimize the direct entry of surface water into the landslide mass. Providing less expensive structures such as wire meshed stone embankments, reduce the buildup of water in the slope (Pilot, 1988).

2.14.3 Providing Restraint or Prevention Works

Slope stability can also be increased by placing retaining structures to increase the resistance to movement. These include gravity retaining walls, gabion walls, cast-in situ reinforced concrete walls, reinforced earth-retaining structures. According to Webster's New World Dictionary (1988), retaining wall is a wall built to keep a bank of earth from sliding or water from flooding. Retaining walls can be installed downslope of landslides to stop moving landslide debris.

2.14.4 Modifying Slope Geometry

According to Bromhead (1997), removal of all or part of the earth driving landslide to modify slope geometry which is the most efficient way of increasing the factor of safety of a slope. Because of the steeper the slope, that disturb balance forces, slope reduction as a result gravitational force minimization along the slope is necessary (Crozier, 1999; and Alexander, 1993). The geometry of the slope can be modified by grading a slope angle to a uniform flatter angle, removing the material from the driving the landslide area and possible substitution by lightweight fill. Additionally, concentrate the filling at the toe of the slope, creating a berm in the section and reduce overall height and angle of the slope. The corrective slope regrading (fill or cut) is successed through determination of size or shape of the alteration and position on the slope.

2.14.5 Treating with Electro Osmosis

Electro osmotic treatment is the process of draining water in soft and unstable fine grained soil from anode to cathode, that induces negative pore pressure and results in consolidation of the soil such enhance shear strength. (Mitchell and Soga, 2005) states that the strength of the soil is improved by draining pore water and creating negative pore pressure using electro osmosis. Electro osmosis is an effective and economically viable method to drain, consolidate and strengthen loose clay and silty soils.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Study Area

The study area, Lalisa village is located on the South Western part of Ethiopia, Oromia National Regional State, Jimma zone, Tiro Afata district (Figure 3.1). It is about 345 km North East of Asandabo from Addis Ababa. It is geographically bounded between 07^0 59′ 126.0″ N 037^0 18′ 289″ E at inlet to 08^0 06′ 05″ N 038^0 05′ 012″ E at the outlet with an elevation 2176 m and 2106 m respectively. Its climatic condition was warm humid with an average temperature varies between 11 and 30 °C and inter-annual rainfall variability of 1900 ± 800 mm. The study area dealt with large cracks and subsidence, residual loose soil mass, red clay soil mass, highly weathered and fractured rock, small spring at the toe of the slope, river flow at about 3 km at the left side, infrastructure of new under construction gravel road at about 2 km at the left side of the study area, gentle topography, bare land at the inlet, sparse forest at the outlet, grass, tree and crop plantation and the livelihood community living around were agricultural based. Generation of the study area map with the existing road infrastructure, river and topography (Figure 3.1) was conducted using DEM of eth_dem_200, geographic coordinate system of Adindan_UTM_Zone_37 N and 1: 500 scale.

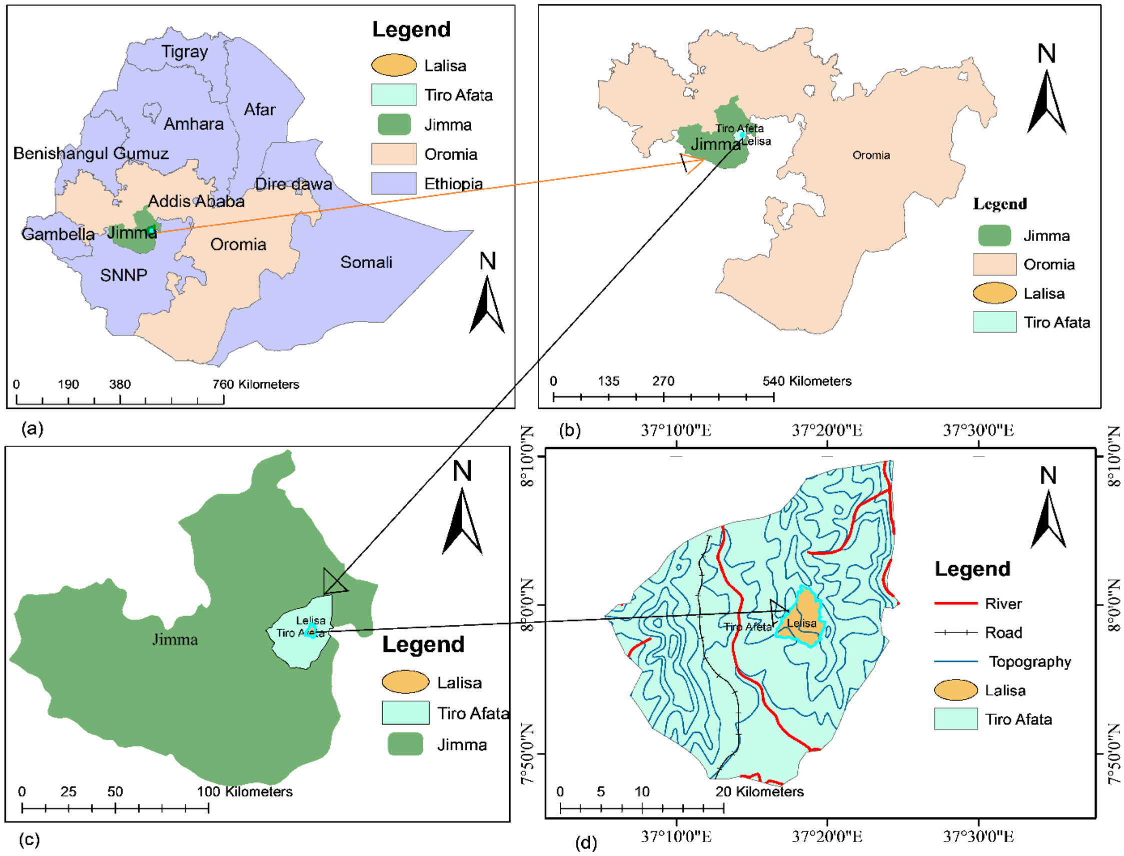


Figure 3.1 Location map of the Study Area

3.2 Site Visitation and Data Collection

Lalisa village, Tiro Afata district, Jimma zone, Oromia National Regional State, South – West Ethiopia was visited for investigation of some geotechnical characteristics and slope stability analysis of landslide situated in the site. During site visitation the presence of small spring at the toe of the slope, scars, large surface cracks, ground subsidence, destruction of natural features, damages and tilting of plants, affected farm and grazing land, topography, land use and photo graphs (Figure 3.2) of landslide affected area were taken. In landslide affected study area, the samples were collected from the slope toe and by faring away a distance of 10, 30 and 50 m from the failure surface of slope crest in order to check the stability of the soil against failure. Thus, disturbed and undisturbed representative soil samples were taken using plastic bag and cylinder tube at different depths of 1 to 3 m test pits. Coordinates of the sampling pits (Table 3.1) were taken during data collection using GPS. The collected soil samples were transported to soil laboratory. The size of the sliding area was surveyed with average length, width, depth and affected area about 3,000, 20, 8 m and 104 ha respectively.

Table 3.1 Location, Coordinates, Sampling Depth and Sampling Types of the Study Area

Test Pit	Location of the sample	Location			depth (m)	Sampling type	
		E	N	Elevation (m)		Disturbed	Undisturbed
TTP1	Toe	882936	313096	2126	1.0-1.2	Yes	Yes
TTP2	Toe	882959	313099	2138	1.0-1.2	Yes	Yes
TTP3	Toe	882689	312728	2099	1.0-1.2	Yes	No
CTP1	Crest	882992	313098	2150	1.5 & 3	Yes	Yes
CTP2	Crest	883013	313093	2151	2.3 - 2.5	Yes	Yes
CTP3	Crest	882733	312695	2124	2.3-2.5	Yes	Yes

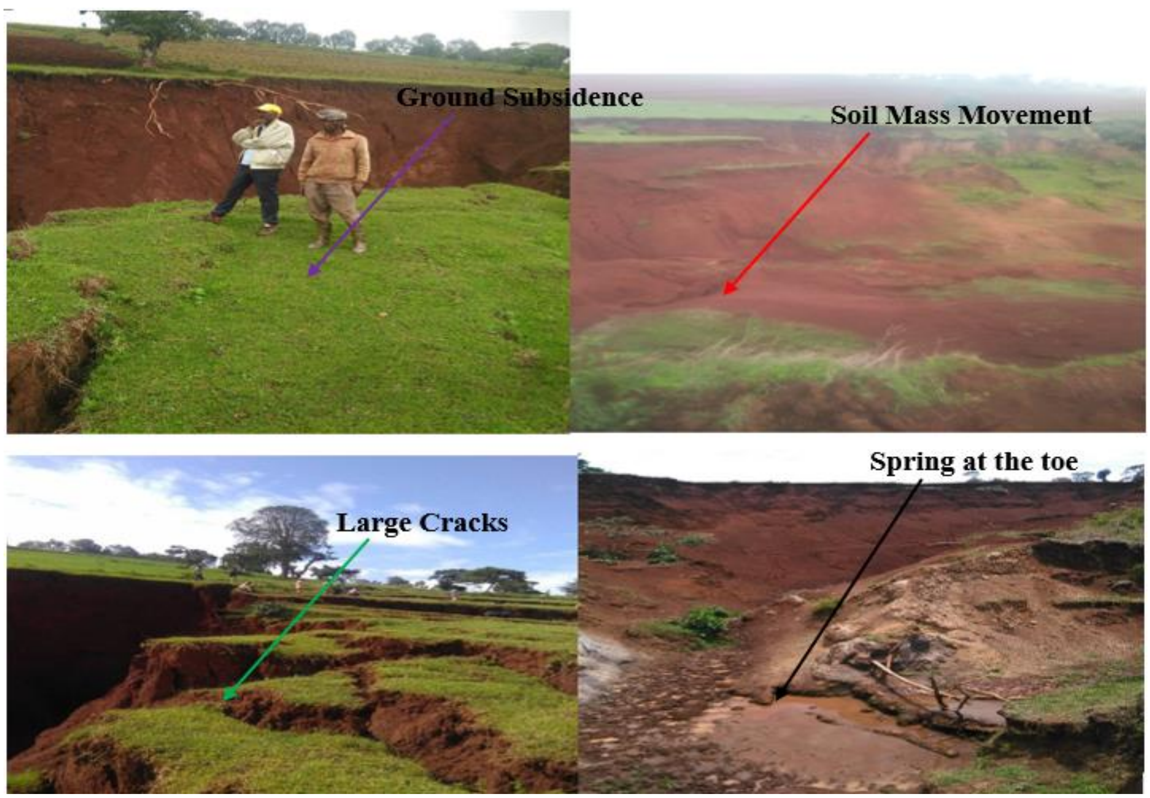


Figure 3.2 Landslide Affected Area (July, 2018)

3.3 Climate and Topography

The climatic condition of the study area was warm humid and wet characterized by high inter-annual rainfall variability (1900 ± 800 mm over the period 2000-2017) (Figure 3.4) and in the rainy season the maximum intensities of precipitation varies between 40 and 60 mm per day. Temperature is varies between 11 and 30 °C throughout the year which was high during summer and low during winter time. Even though the rainfall distribution is varies throughout the year, it has dry and wet season. The wettest period of the year was from March to October, while the dry season occurs from November to February (Figure 3.3). Towards June or from June till August there is high intensity of rainfall in these months (June, July and August) that results concentrated water flow, high runoff or erosion and temporal stream flow. There is decrease in intensity of rainfall in March, April, May, September and October months (Figure 3.3) cause water table lowering and decrease the probability of erosion. The topography of the study area was characterized by a gentle slopes with gradient 12^0 and steep slopes varies between 42 and 52^0 at the valley of landslide affected area.

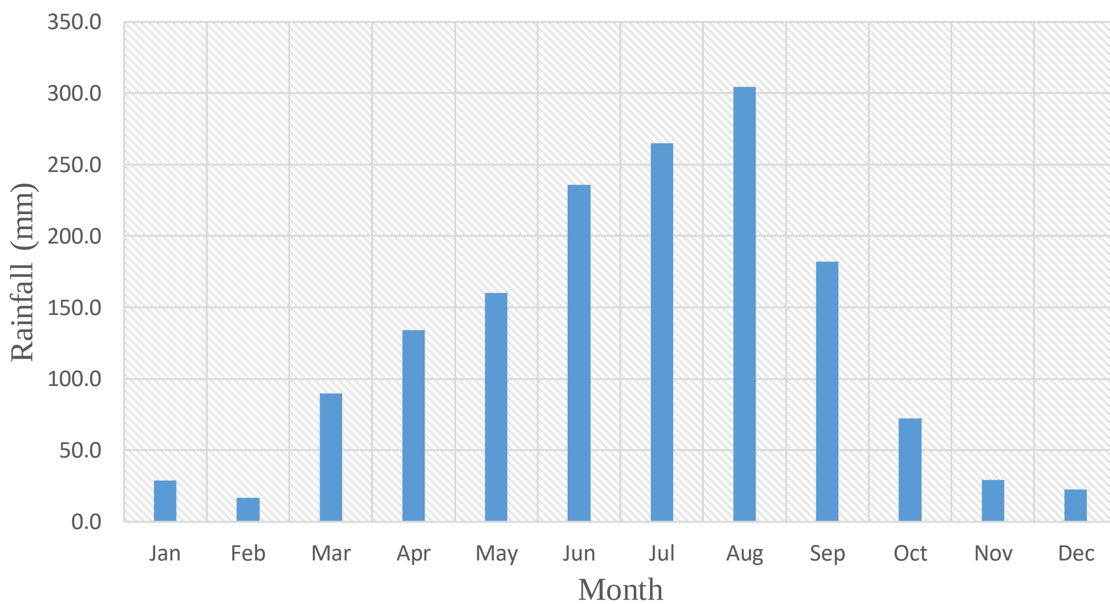


Figure 3.3 Mean Monthly Rainfall Distribution for Dimtu Meteorological Station for the years 2000-2017 (Source: Ethiopian National Meteorology, Jimma Station)

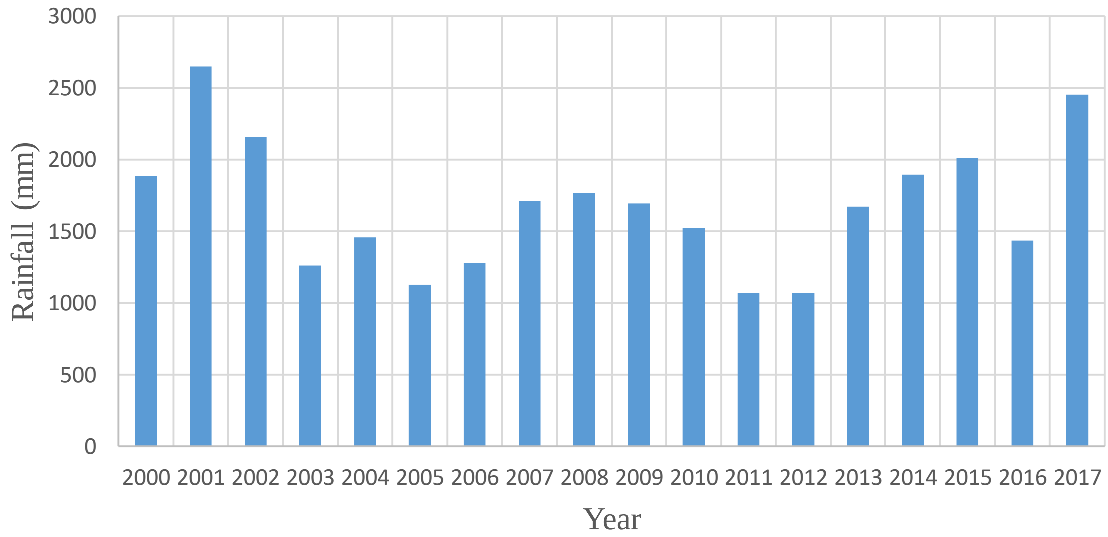


Figure 3.4 Time versus Rainfall Distribution for Dimtu Meteorological Station for the Years 2000-2017 (Source: Ethiopian National Meteorology, Jimma Station)

3.4 Study Design

The research was carried out using field survey, laboratory and software analysis. The field survey was carried out using GPS and geo-electrical resistivity survey, laboratory analysis from representative soil samples to get material properties and input parameters for software analysis were quantitative primary data while, secondary data was obtained from different literature reviews and communities living around the study area.

3.5 Software and Devices

The software (Arc GIS 10.1, Geo-studio 2012, Res2dinvx32, WinResist, MS word and Excel) and device (mobile camera, Garmin GPS 72 H and Syscal Junior) were used for the study. Arc GIS 10.1 and Geo-studio 2012 were used to delineate the study area and numerically analysis the slope stability against the landslide respectively; Res2dinvx32 was used in examination of horizontal profile of soil layers with different resistivity; WinResist for determining apparent electrical resistivity of vertical soil profile; MS word and Excel were used to analyze laboratory and display research data; mobile camera and Garmin GPS 72 H were used for documentation and determine the location of landslide affected area respectively and Syscal Junior was used to identify the subsurface profile of the soil and ground water level.

3.6 Field Work

It includes reading coordinates or location of landslide affected area, measuring the length, width and depth, assessing the presence of the river and spring at the toe of the slope, topography, investigating landslide indicators, conducting geo-electrical resistivity test and test pit excavations for subsurface soil investigations and sampling for laboratory analysis.

3.7 Data Collection Procedures

The data collection for the completion of this research were: (1) reviewing previous studies and literatures on research title related, (2) Interviewing (3) Measuring and reading the size and location of the landslide affected area and conducting field test (4) Geotechnical investigation of soils (5) Slope stability analysis using limit equilibrium method.

3.8 Sampling Preparation for Laboratory Analysis

The soil samples taken to the laboratory for investigation of some geotechnical characteristics and slope stability analysis of the affected area were: (1) The disturbed collected samples at different depths were air dried 3 – 4 days and oven dried at $\pm 105\,C^{o}$ for 16 to 24 hr before carrying out laboratory test. (2) The undisturbed collected samples using cylinder tube and tied to plastic bag to prevent moisture loss was used for unconfined compressive strength and in situ natural moisture content determination. Natural moisture contents and unconfined compressive strength were determined immediately, after the samples brought to the laboratory. After air or oven dried each samples were weighted for the required laboratory test and the test was carried out in accordance with ASTM standard.

3.9 Laboratory Analysis

To identify and characterize the problem nature of the slope material for slope stability, a range of laboratory analysis were carried out. Among those Atterberg limits (liquid and plastic limits), specific gravity and particle size distribution were conducted for geotechnical classification whereas shear strength and bulk density for slope stability analysis. Additionally, free swell, liquidity index and plasticity index were determined to identify the soil characteristics of the study area. The following below were laboratory tests analyzed for investigation of geotechnical characteristics and slope stability analysis.

3.9.1 Atterberg Limits

This test was conducted using disturbed samples in accordance with ASTM D 4318 to determine the plastic, liquid limits and plasticity index of a fine grained soils for their classification. The plasticity or compressibility of the soil samples were also determined using ASTM standard plasticity chart. The test was carried out for both air dried and oven dried samples to determine the soil type. Seven air dried samples for liquid and plastic limit determination and five oven dried samples for liquid limit determination were conducted (Appendix A, Table A.1 to A.12).

3.9.1.1 Liquid Limit, LL

This test was carried out to determine the water content at which the soil changes from liquid state to the plastic state using the standard cup method.

Test procedure

About 125 g of soil was passed through a 425 micron sieve and mixed with distilled water in the evaporation dish and soaked for 24 hours to form a paste. The Casagrande tool was adjusted and a portion of the paste was taken and placed in the Centre of the cup so that it was almost half filled, the adjustment plate was secured by tightening its screws. The top was levelled so that it was paralleled to the rubber base and the maximum depth of the soil depth (1 cm) and the paste was divided along the cup diameter using grooving tool. Then, a V-shaped gap, 2 mm wide at the bottom and 11 mm at the top and 8 mm deep was formed. The handle of the apparatus was turned at the rate of 2 revolutions per second, until the two parts of the soil came in contact with the bottom of the groove along a distance of 10 mm. The number of blows required to cause the groove to close for the approximate length of 10 mm were recorded. Some portion of the soil removed from cup and the remaining mixed with the soil left earlier on the marble plate. The portion of sample removed used for moisture content determination. Record the weight of empty can (W_c), put some portion of wet sample in can and weigh the mass of wet sample and can (W_{wsc}), then put in oven for 16 to 24 hr for moisture content determination. After 16 to 24 hr, take and record the weight of dry soil and can (W_{dsc}). The above procedures were repeated for four trials. Moisture content (w_c) determined using (Equation 3.2) from the ratio of weight of water, W_w obtained using (Equation 3.1) to weight of dry soil, W_d obtained using (Equation 3.2).

Moisture content against number of blows were plotted on a semi logarithmic chart to obtain liquid limit at 25 number of blows.

$$W_w = W_{wsc} - W_{dsc} \tag{3.1}$$

$$W_d = W_{dsc} - W_c \tag{3.2}$$

$$w_c = \frac{W_w}{W_d} \times 100 \tag{3.3}$$

3.9.1.2 Plastic Limit, PL

The plastic limit is the water content in percent at which a soil can no longer be deformed by rolling into 3.2 mm diameter threads without crumbling.

Test procedure

Some portion of the plastic soil paste left from liquid limit test sample which was dried partially on a glass rolling plate and rolled between the plate and the finger throughout its length. When the diameter of the thread decreased to 3.2 mm, the specimen was kneaded together and rolled out again. The process was continued until the thread just crumbled at 3.2 mm diameter. The rolled sample into 3.2 mm diameter threads without crumbling put in can for water content determination. The above procedure for moisture was repeated for three trials. The equation and procedure except number of blows for moisture content determination was the same with that of liquid limit. The moisture content of three trials were determined using (Equation 3.3) and average value were taken as a plastic limit value.

3.9.1.3 Plasticity Index, PI

This was determined as the difference between the liquid limit and the plastic limit (PI = LL-PL).

Soil type and soil mineral determination

The soil type and swelling characteristics of clay soil are determined using Atterberg limit values (liquid limit, LL, plastic limit, PL and plasticity index, PI) in (Table 3.2 and 3.3)

Table 3.2 Typical Atterberg for Soils

Soil type	LL (%)	PL (%)	PI (%)
Sand	Non-plastic		
Silt	30-40	20-25	10-15
Clay	40-150	25-50	15-100

Table 3.3 Swelling Characteristics of Clay Soil by PI Value (Terzagi and Peck, 1967)

PI (%)	Swelling potential
0 - 15	Low
10 - 35	Medium
20 - 55	High
> 55	Very high

3.9.1.4 Liquidity index, LI

Liquidity index is the measure of soil strength using Atterberg limits and is expressed as the ratio of the difference between natural moisture content and plastic limit to its plasticity index using (Equation 3.4) (Atterberg, 1911) .

$$LI = \frac{w_N - PL}{PI} \tag{3.4}$$

Where, $LI =$ liquidity index, $w_N =$ natural moisture content, $PL =$ plastic limit, $PI =$ plasticity index.

Table 3.4 Description of Fine-Grained Soils Strength based on Liquidity Index (Wiley, 2010)

Values of LI	Description of soil strength
LI < 0	Semisolid state- high strength, brittle, sudden fracture is expected
0 < LI <1	Plastic state- intermediate strength, soil deforms like a plastic material
LI > 1	Liquid state- low strength, soil deforms like a viscous fluid

3.9.2 Bulk Density, Unit Weight and Natural Moisture Content

Bulk density is the ratio of total mass to total volume of the sample taken for density determination, unit weight is the ratio of total weight to total volume and natural moisture content is the ratio of weight of water to weight of dry soil. Thus, unit weight is the main parameter in geotechnical slope stability analysis. The samples used were undisturbed samples obtained by cylindrical tube of 36 mm diameter. The test of natural moisture content, density and unit weight of soils of the study area is determined for seven samples (Appendix A, Table A.14 to A.15).

Test procedure

Wax and coding tube of sample test pit, extrude the sample from cylindrical tube at laboratory and trim the ends of the sample, weigh the total mass of sample (M_s), measure length (H_s) and diameter (D_s) at three different points and take the average, determine total volume (V_S) from diameter and length, determine bulk density (ρ_b), determine bulk unit weight (γ_b), take to can some samples for natural moisture content determination, weigh clean can (W_c), weigh can and wet soil (W_{wsc}), put can with natural wet soil in oven for 16 to 24 hr, weigh can and dry soil (W_{dsc}) after 16 to 24 hr, determine natural moisture content (W_N), dry unit weight (γ_d), void ratio (e) and saturated unit weight (γ_{sat}) are determined through (Equation 3.5 to 3.10) and for all samples the above procedures were performed. Based on saturated and dry unit weight of soils different soil types identified in (Table 3.5).

$$V_s = \frac{\pi {D_s}^2}{4} \times H_s \tag{3.5}$$

$$\rho_b = \frac{M_s}{V_s} \tag{3.6}$$

$$W_N = w = \frac{W_w}{W_d} \times 100 \tag{3.7}$$

$$\gamma_d = \frac{\gamma_b}{1+w} \tag{3.8}$$

$$e = (\frac{G_s \gamma_w}{\gamma_d} - 1) \tag{3.9}$$

$$\gamma_{sat} = \frac{(G_s + e)\gamma_w}{1+e} \tag{3.10}$$

Where W_w and W_d weight of water and weight of dry soil given in equation 3.1 and 3.2 respectively and the value unit weight of water, γ_w taken as 9.81 KN/m^3.

Table 3.5 Typical Values of Unit Weight for Different Soils (Wiley, 2010)

Soil Type	γ_{sat} (KN/m^3)	γ_d (KN/m^3)
Gravel	20-22	15-17
Sand	18-20	13-16
Silt	18-20	14-18
Clay	18-20	14-21

3.9.3 Specific Gravity

Specific gravity is a measure of the heaviness of material and used in determination of hydrometer analysis. It was determined in accordance with ASTM D 854 using a 50 ml density bottle, which is used for fine-grained soils. The disturbed samples were used for the determination specific gravity. It is determined for seven samples (Appendix A, Table A.16 to A.18).

Test procedure

The empty pycnometer was cleaned, dried, weighed (W_p) and a small quantity of dry soil passing 2 mm sieve was placed in it, the mass of the pycnometer and the dry soil (W_{ps}) was weighed, water added up to half of the pycnometer and agitated for 15 minutes to remove air bubble, fill water and mass of pycnometer, soil and water recorded (W_{psw}). After all air bubbles have been removed, the pycnometer was filled with de-aired water and weighed (W_{pw}). Accordingly, the specific gravity (G_s) of the soil mass is determined from (Equation 3.11), average specific gravity of three trials taken and determined at 20 C^0 using (Equation 3.12). Then after it is reported at a temperature of 20 C^0. The above procedures were repeated for other samples. Different soils have different specific gravity (Table 3.6).

$$G_s = \frac{(W_{ps} - W_p)}{((W_{ps} - W_p) + (W_{pw} - W_{psw}))} \tag{3.11}$$

$$G_s 20C^0 = K \times G_s \tag{3.12}$$

Where K is the coefficient of specific gravity at temperature T_x.

Table 3.6 General Ranges of Gs for Various Soils (Das, 2002)

Soil Type	Range of Gs
Sand	2.63 - 2.67
Silt	2.65 - 2.7
Clay and silty clay	2.67 - 2.9
Organic	less than 2

3.9.4 Grain-size distribution

Grain size analysis is the laboratory test which provides the grain size distribution required in classifying the soil and determine the percentage of different grain sizes contained within a soil. The tests are conducted on disturbed samples for both wet analysis to determine the distribution particles coarser than 75 µm (retained on No. 200 sieve) and sedimentation by hydrometer analysis for the determination of the distribution particles finer than 75 µm in accordance with ASTM standard with designations D422-63 and D1140-97. The grain size analysis of wet and hydrometer analysis of soils of the study area is determined for seven samples (Appendix A, Table A.19 to A.29).

Test procedure

The procedure followed to conduct this test was according to ASTM standard with designations D422-63 and D1140-97. The test procedure for wet sieve analysis was as: The sample is sieved with 9.5 mm sieve, the sample of 1000 g passed through 9.5 mm is weighed (M_T) and transferred to a container, soak the sample for 24 hr, after it is soaked for 24 hr it is washed on 75 µm sieve until the wash water becomes clear, the material retained on 75 µm is collected and dried in an oven, it is then sieved through the set of fine sieves of the size 9.5, 4.75, 2, 0.85, 0.425, 0.25, 0.15 and 0.075 mm. The material retained on each sieve collected and weighed as mass of retained (M_R), percentage of retained material (% R) on each sieve and passed material (% P) through each sieve is determined using (Equation 3.13 to 3.14) respectively. The above procedure wet analysis is repeated for other samples. The samples of soil passed through 75 µm sieve is transferred to large dish for hydrometer analysis and soaked until the water becomes clean, then the cleaned water was decanted. After the sample has dried in room temperature, it was pulverized and 50 g of soil is weighed (M_{th}) for hydrometer test. The 50 g (M_{sh}) taken samples was soaked for 24 hours by adding sodium hexametaphosphate dispersing agent. At the end of soaking, the sample is further dispersed using stirring apparatus. Then it is poured into 1,000 ml cylinder and make a solution again for a period of 1 min by covering it with plastic bag. The actual hydrometer reading (HR) and test temperature (T) are recorded for 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes. Corrected hydrometer reading, CHR of (R' and R'') is determined, correction factor (a), effective depth of hydrometer (L), constant which depends on the temperature of the suspension and the specific gravity of the soil particles (K), diameter of soil particle (D) and percentage of materials (P) passing each sieves are determined through (Equation 3.15 to 3.20). Percent of finer each sieve against the sieve size is plotted on a semi-logarithmic chart. The procedures mentioned above repeated for all samples of hydrometer analysis.

$$\%R = \frac{M_R}{M_T} \times 100 \tag{3.13}$$

$$\%P_i = 100 - \sum_{i=1}^{i=n} (\%M_R)_i \tag{3.14}$$

$$R' = HR + Cm \tag{3.15}$$

$$R'' = HR - Zc + C \tag{3.16}$$

$$a = 0.6226 \frac{G_s}{G_s - 1} \tag{3.17}$$

$$K = \sqrt{\frac{30\eta}{981\rho_w (G_s - 1)}} \tag{3.18}$$

$$D = K\sqrt{\frac{L}{T}} \tag{3.19}$$

$$P = a\frac{R''}{M_{sh}} \times 100 \tag{3.20}$$

Where Cm = meniscus correction, +1, Zc = zero correction, +6, C = correction based on test temperature given in (Appendix B.1, Table B.1), η = viscosity based on test temperature (Appendix B, Table B.5), ρ_w = density of water has a unity value, Gs = specific gravity, n = number starts from 1, L = effective depth of hydrometer based on actual hydrometer reading given in (Appendix B, Table B.6), M_{sh} = Mass of sample for hydrometer test.

3.9.5 Free Swell

Free swell is the reflection of soil expansion and obtained by dividing the volume difference of kerosene and water to kerosene volume. The test was carried out on an air dried disturbed samples of six test pits (Appendix A, Table A.30).

Test Procedure

The cylinder was cleaned, air dried material was sieved using No 40 sieve (0.425mm diameter) and pouring 10 cc of dry soil passing through No 40 sieve into a 100 cc different graduated cylinder one for water and the other for kerosene, add small amount of water and kerosene greater than 10 cc, mixing and shaking to take out air bubbles, fill a cylinder with distilled water and the other cylinder with kerosene, the mixture was allowed to stand for 24 hours to settle for water , the kerosene reading which has no change in swelling was taken as initial reading Li and that of water taken as final reading Lf , free swell calculated using (Equation 3.21).

$$FS = \frac{(Lf - Li)}{Li} x100 \tag{3.21}$$

3.9.6 Shear Strength

Shear strength of a soil is the maximum resistance to shear stresses just before the failure. Shear strength test is used to determine shear strength parameters (cohesion and internal friction) which are crucial for slope stability analysis. The shear strength parameter determined for slope stability analysis of the study area is undrained cohesion which is a measure of the intermolecular forces and it holds the particles of the soil together in a soil mass.

3.9.6.1 Unconfined Compressive Strength

Unconfined compression test was carried out on undisturbed samples in accordance with ASTM D 2166-00 for the determination of unconfined compressive strength and undrained cohesion of cohesive soils. It is unconsolidated undrained, UU test where lateral confining pressure is equal to zero or atmospheric pressure and it was assumed that there is no pore water lost from the sample during the shearing process. The test was carried out for five test pits (three at the crest and two at the toe) of the slope (Appendix A, Table A.31 to A.35), two trial and the average value was taken for each test pits.

Test Procedure

The sample was extruded from the sampling tube. A cylindrical sample of soil was trimmed such that the ends are reasonably smooth and the average of sample reading height (h) and diameter (D) at three point recorded, initial area (A_o) is determined. The soil sample is placed in a loading frame on a metal plate; by turning a crank and the bottom of the plate raised. The top of the soil sample was restrained by the top plate, which was attached to the calibrated proving ring. As the bottom plate was raised, an axial load was applied to the sample. The crank was turned at the specified rate so that there was constant strain rate. The load is gradually increased to shear the sample, and load dial readings (LDR) were taken periodically with 20 intervals of deformation dial reading (DDR). The loading is continued until the soils develops an obvious shearing plane or the deformations become excessive. Sample deformation ($\Delta L = 0.001*DDR$), strain (ε) from ratio of sample deformation to sample height, percentage of strain (% ε), corrected area (A'), load (P) and stress are determined through (Equation 3.26 to 3.28). The stress-strain graph plotted, unconfined compressive strength, qu which is the maximum load per unit area and undrained cohesion that is half of unconfined compressive strength are determined from stress-strain graph. The consistency of clay soil is determined based on unconfined compression strength value (Table 3.7).

$$A' = \frac{A_0}{1-\varepsilon} \tag{3.26}$$

$$P = 0.3154 \times LDR \tag{3.27}$$

$$\sigma = \frac{P}{A'} \tag{3.28}$$

Table 3.7 Consistency and Unconfined Compression Strength of Clay (Taylor and Francis, 2007)

Consistency	Unconfined Compression Strength of Clays, KN/m^2
Very soft	0 - 24
Soft	24 - 48
Medium	48 - 96
Stiff	96 - 192
Very stiff	192 - 383
Hard	> 383

3.9.7 Soil Classification

A soil classification is an arrangement of different soils into groups having similar properties. The purpose of soil classification is to make possible estimation of soil properties by association with soils of the same class whose properties are known. The soils of the study area have been classified according to ASTM D 2487-10 standard practice for classification of soils for engineering purposes (Appendix B, Figure B.1). Plasticity chart of the soil as ASTM standard on (Figure 3.5) determines the place of PI plot whether it was plotted below or above A-line that separates clay soil from silt soils and the type of soil was determined in accordance to ASTM D 2487-10 standard practice for classification of soils for engineering purposes. Average grain size classification according to ASTM -D-422, gravel particles larger than 4.75 mm ,coarse sand passing 4.75 mm and retained 2 mm, medium Sand passing 2 mm and retained 0.425mm, fine sand passing 0.425 mm and retained on 0.075 mm, Silt size 0.074 to 0.005 mm, Clay size 0.001 to 0.005 mm and colloids those less than 0.001.

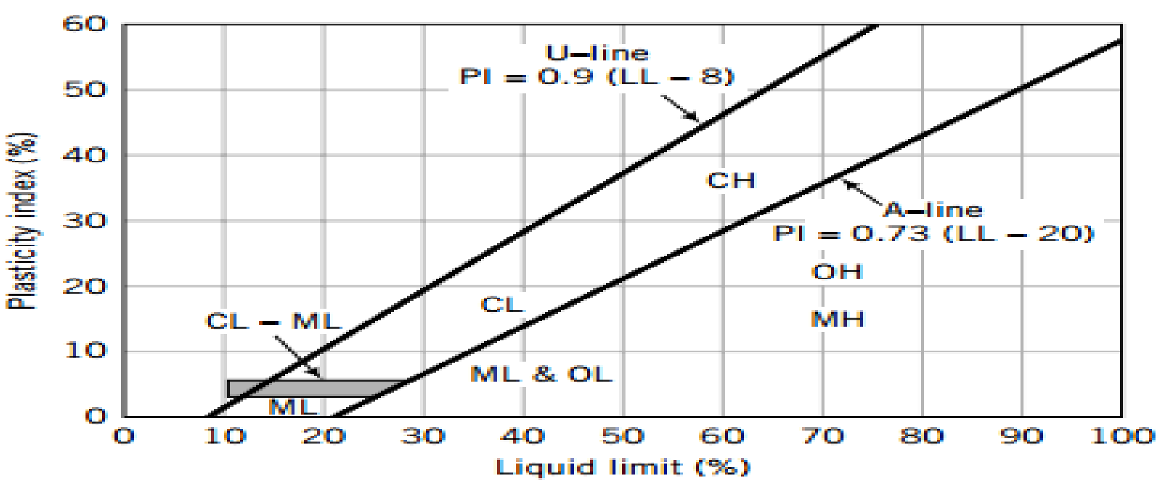


Figure 3.5 Plasticity Chart of the Fine-Grained Soil, ASTM Standard (ASTM, 2010)

3.10 Geo-physical Survey

The objective of conducting geophysical survey was to delineate the internal structure of the existing natural slope thus estimating possible thickness of different soil layer, weak zone of the subsurface soil, locate ground water level and different resistivity zone, determine direction of ground water flow depending on the structures observed during the study and identify type of soil depending on the resistivity value obtained from winResist and Res2dinvx3 software.

Test procedure

72 electrode system (Syscal Junior) taken to 10 m distance from the existing failure at the slope crest to conduct electrical resistivity tomography, all electrodes installed along the profile line at an inter-electrode spacing of 10 m with the total length about 720 m, the data were recorded on the potentiometer using Schlumberger–Wenner sequence with 72 electrodes deployed, vertical electrical sounding and electrical profiling were performed at specifically pinpointed sites based on the accessibility for layout of the instrument and respective cables, processing and inversion of resistivity image profile data performed using software Res2dinvx3 and wenner array, apparent resistivity data of iterated sounding curves was performed using software winResist and schlumberger array, determine soil type and their strength as well as their role in landslide occurrence depending on resistivity value (Table 3.10). The complete instrument and electrode installation of geo-electrical survey (Figure 3.6).

Figure 3.6 Instrument and Electrode Installation of Geo-Electrical Survey

Table 3.8 Typical Resistivity Values for Various Materials (Solberg et al., 2012)

Resistivity (Ωm)	Main characterization	Description
1 - 10	Unleached clay deposits	The clay has been exposed to little leaching since deposition. The pores in the clay still contain salt water, which stabilize the structure. Because of the large concentration of ions in the pore water, the

		conductivity of the clay is good, and thus the resistivity values are low.
10 - 100	Leached clay deposits	Sensitive clay develops as groundwater leaches ions from the marine clay. The electrical conductivity of the deposit is still high, but not as good as for the unleached clay (not quick anymore), silt, and fine-grained till.
> 100	Dry crust clay deposits, coarse sediments, (bedrock)	Dry crust clay; remoulded, dry clay from quick-clay landslides; and coarser materials like sand and gravel will have higher resistivity values than marine clay. Most bedrock types will have values of several thousand Ωm.

3.11 Slope Stability Analysis

The slope stability analysis of the study area was analyzed using Geo studio SLOPE/W software with the aim of giving the state of the slopes based on their factor of safety for circular using Limit Equilibrium Method, LEM. The method of slices are considered in relation to its application to SLOPE/W and traditional methods of analysis. Two different conditions; Condition 1 (FOS determination of natural slope at TTP1 and CTP1 and TTP2 and CTP2) and Condition 2 (FOS determination of modified slope angle from 52 to 30^0 at TTP1 and CTP1 and TTP2 and CTP2). These conditions are analyzed to identify the condition of the slope, effect of slope angle and distance from failure surface on slope stability or FOS and then propose prevention or remedial measure of the landslide of the study area. To complete the slope stability analysis of this study area, four soil layers named as upper, lower and middle soil layer and bedrock based on the result of geophysical resistivity of vertical electrical sounding (Figure 4.5) is conducted. Different soil layers have different input parameters for slope stability analysis. As vertical electrical sounding value of geo-electrical resistivity, first and third soil layer were clay soil, their resistivity also < 10 Ωm and no laboratory analysis at deeper than 3 m conducted; the input material for third layer is taken as first soil layer input material. The first, second, third and fourth soil layer thickness used for analysis were 2.8, 7.3, 2 and 1 m respectively, slope geometry of 5 m slope height, size of domain 17 (at TTP1 and CTP1 which was 10 m from failure) and 37 m (at TTP2 and CTP2 which was 30 m from failure surface) width and 10.3 m height as shown from slope geometry on (Figure 3.7 and 3.8). Due to large thickness, 25.1 m of third layer soil, for simplicity only 2 m thickness considered for analysis. The analysis used Morgenstern- Price method as it fulfills force and moment equilibrium, half sine–function for side function, piezometric line of PWP condition, entry and exit slip surface, 30 number of slices and Mohr Coulomb material model. The complete set of input soil parameters used in the analysis is shown in (Table 3.9). The unit weight of the soil used is saturated unit weight as the soil is fully saturated (Appendix A, Table A14 to A15) Because of unconsolidated undrained conducted test for shear strength determination of soils of the study area the value of internal friction angle is zero ($\phi = 0$) is used in analysis. The minimum factor of safety (FOS), critical slip surfaces (CSS) were searched by entry and exit option as well as groundwater table (GWT) level shown in the model using limit equilibrium method, LEM principle.

Table 3.9 Input Materials and Slope Angle for Determination of Slope Stability

Condition	Slope angle (^{0})	Pit	Cu (KN/m^2)	γ (KN/m^3)
1	52	TTP1	9.46	18.6
		CTP1	11.3	18.78
	52	TTP2	12.3	18.43
		CTP2	20.26	19.27
2	30	TTP1	9.46	18.6
		CTP1	11.3	18.78
	30	TTP2	12.3	18.43
		CTP2	20.26	19.27

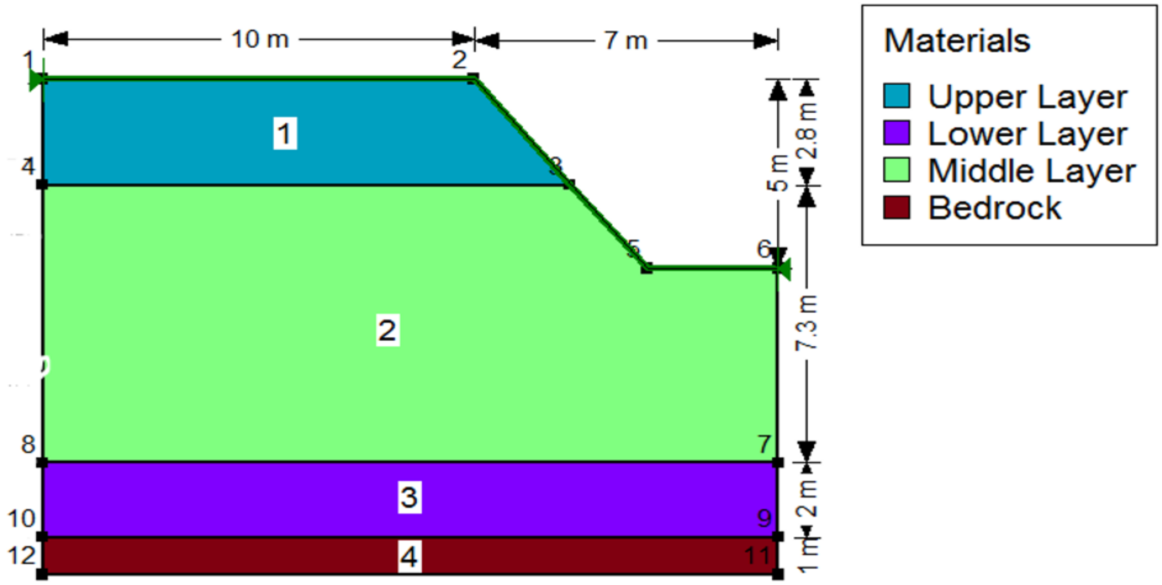


Figure 3.7 Slope Geometry at TTP1 and CTP1

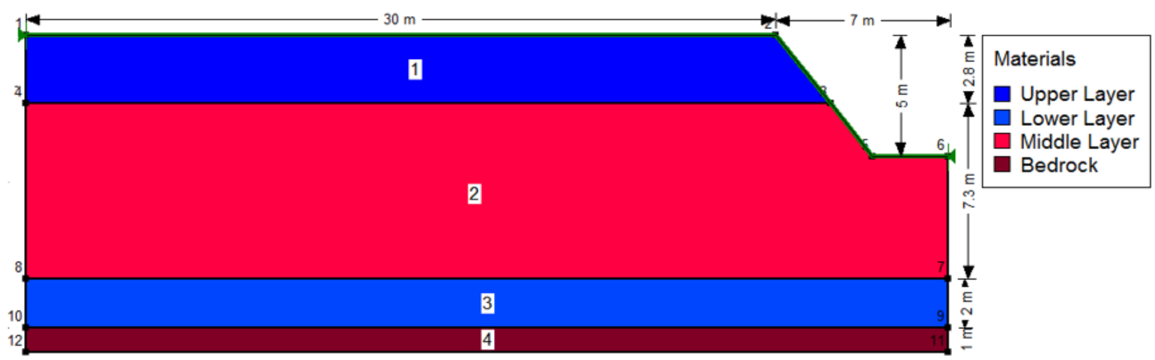


Figure 3.8 Slope geometry at TTP2 and CTP2

3.11.1 Stability of Slopes Using Method of Slices

The basic principle of slice method is the potential slide mass, which is subdivided into several vertical slices and the equilibrium of individual slice can be evaluated in terms of forces and moments. The stability of a slope in a ϕ,Cu soil is usually analyzed by discretizing the mass of critical slope surface into smaller slices and treating each individual slice as a unique sliding block (see Figure 3.9) from the field of study area. This technique is called the method of slices. In the method of slices, the soil mass above a trial failure circle is divided into a series of vertical slices of width *b* as shown on Figure 3.10 (a) for each slice, its base is assumed to be a straight line defined by its angle of inclination θ with the horizontal whilst its height h is measured along the centerline of the slice. The FOS of the slope using method of slice determined through (Equation 3.30 to 3.34). The last equation 3.4 is used for undrained condition in which the value of $\phi = 0$.

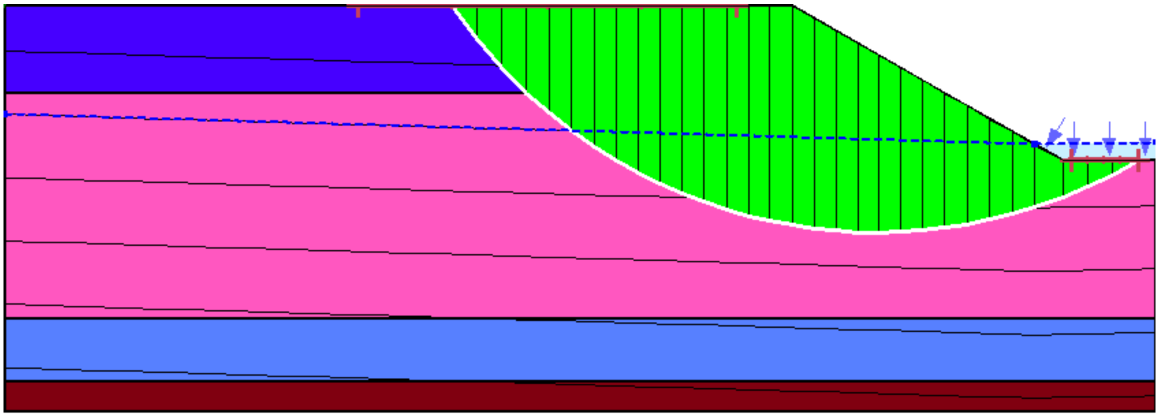


Figure 3.9 Slice Discretization and Slice Forces in a Sliding Mass

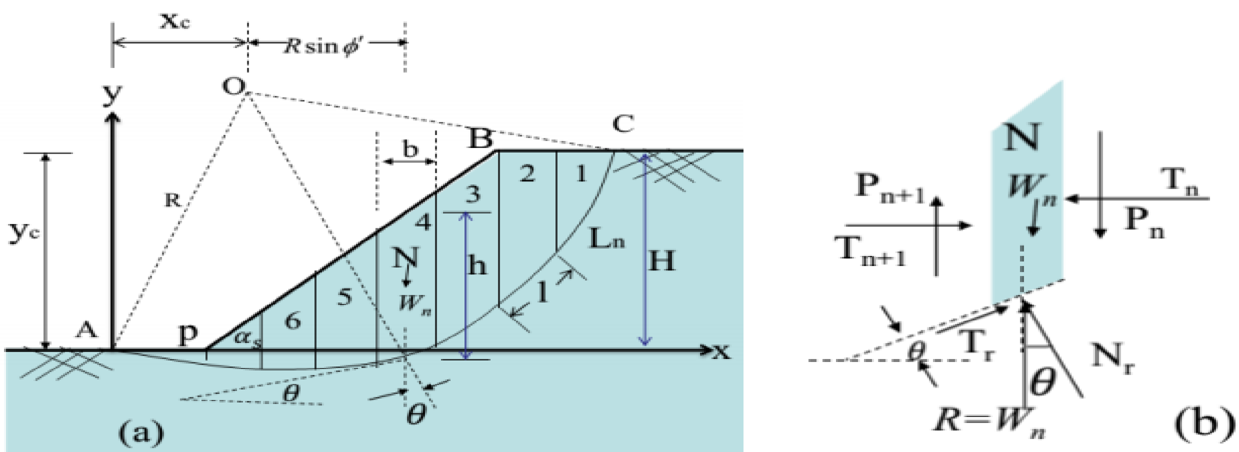


Figure 3.10 a) Method of slices in c', soil, b) Forces acting on a slice

$$W_n = \gamma * h * b \tag{3.30}$$

$$T_r = \tau_m * l \tag{3.31}$$

$$N = N' + U \tag{3.32}$$

$$\tau_m = \tau_f / FOS \tag{3.33}$$

The forces acting on a slice shown in Figure 3.10 (b) are total weight of the slice, W_n = total normal force at the base, N = effective total normal force, N' = force due to the pore water pressure at the midpoint of the base length, U = mobilized shear force at the base, T_r = minimum shear stress required maintain equilibrium, τ_m = shear strength, τ_f = Shear forces on sides of the slice, FOS = factor of safety Pn, Pn+1 and normal forces on sides of the slice, T_n, T_{n+1}. The sum of the moments of the inter-slice or side forces about the center O is zero.

$$FOS = \frac{\sum_{i=1}^{i=n}[(c' + \sigma_n \tan\phi')l]_i}{\sum_{i=1}^{i=n}(W\sin\theta)_i} = \frac{\sum_{i=1}^{i=n}[(c'l + N'\tan\phi')l]_i}{\sum_{i=1}^{i=n}(W\sin\theta)_i} = \frac{\sum_{i=1}^{i=n}[(cul)l]_i}{\sum_{i=1}^{i=n}(W\sin\theta)_i} \tag{3.34}$$

3.12 Method of Prevention or Minimization of Landslide

3.12.1 Modifying Slope Angle

Making slope gentle is effective because it increases the stability of the soil and reduces the weight of the sliding mass. Consider the slope angle of β, slope height of H and assume the failure surface is straight. Equation 3.35 stands for undrained condition of the study area. In this study the slope angle is modified from 52 to 30^0 which increased coefficient of FOS from 2.06 to 2.31 (increase by 11 %).

$$FOS = \frac{Cu}{\gamma H \cos\beta \sin\beta} \tag{3.35}$$

CHAPTER FOUR

RESULTS AND DISCUSSIONS

Introduction

This chapter contains the results and discussions of laboratory test, field work and software. From laboratory and field test result, characteristics and type of soil and also their effect on slope instability were discussed. Additionally, software results presents the state of the slope at two different distances from failure surface, FOS of natural slope and modified slope angle and also based on FOS result remedial measures proposed. The following below are the results and discussions of laboratory, field and software analysis.

4.1 Atterberg Limit

The PL and PI of soils of Lalisa village is given in Table 4.1. The value of air dried LL obtained for Lalisa village soils range from 57 to 66.21 % which is greater than 50 %, PL 29.48 to 34.44 %, PI 26.79 to 34.04 % and LI range from 0.08 to 0.28 %. According to Skempton (1953), the values fall in the range of clay soil with LL, 40 to 150 %, PL, 30 to 50 % and PI, 15 and 100 %. As observed by Terzagi and Peck (1967), swelling characteristics of clay soil by plasticity index value as it has high swelling potential, value ranges from 20 to 55 %. In relation to this, swelling potential and PI value of Lalisa village soil has fallen in the range of high swelling potential. According to (Wiley, 2010), the value of LI of Lalisa village soil falls in the range of $0 < \mathrm{LI} < 1$ which describes the state and strength of fine grained soil as plastic state, intermediate strength and soil deforms like a plastic material.

Oven dried LL to air dried LL ratio is given in Table 4.2. Accordingly, the value obtained varies from 0.78 to 0.91. The plasticity chart (the graph of LL against PI) of the study area is given on Figure 4.1. From this chart it has been seen that the soils of the study area plotted on and above A – line which has the equation $\mathrm{Ip} = 0.73(\mathrm{wl} - 20)$ separates clay soils from silt. As flow and plasticity chart of ASTM standard, the ratio of oven dried LL to air dried LL > 0.75 and plot of plasticity chart on and above A - line of Lalisa village soils falls under inorganic and high compressible clay soil which has CH symbol. Therefore, the periodical characteristics of high swelling potential and deforms like a plastic material in wet condition, shrink and highly compressible in dry condition which causes formation of large cracks of clay soil may considered as the main cause for occurrence of landslide in Lalisa village.

Table 4.1 Atterberg Limit of Air Dried Samples

Pit Designation	Depth (m)	w_N (%)	LL (%)	PL (%)	PI (%)	LI (%)
TTP1	1-1.2	44.23	66.15	32.53	33.62	0.14
TTP2	1-1.2	44.88	64.79	31.75	34.04	0.22
TTP3	1-1.2	45.02	66.21	30.00	33.01	0.28
CTP1	1.3-1.5	37.25	57.00	30.21	26.79	0.08
	2.8-3.0	40.41	65.54	34.44	31.56	0.17
CTP2	2.3-2.5	38.49	63.00	29.48	33.52	0.17
CTP3	2.3-2.5	-	62.00	31.08	30.92	-

Table 4.2 Oven Dried LL to Air Dried LL Ratio

Pit	Air dried LL, %	Oven dried LL, %	Ratio
TTP1	66.15	60.00	0.91
TTP2	64.79	58.00	0.89
CTP1	66.21	53.00	0.81
CTP2	63.00	50.00	0.78
CTP3	62.00	53.00	0.86

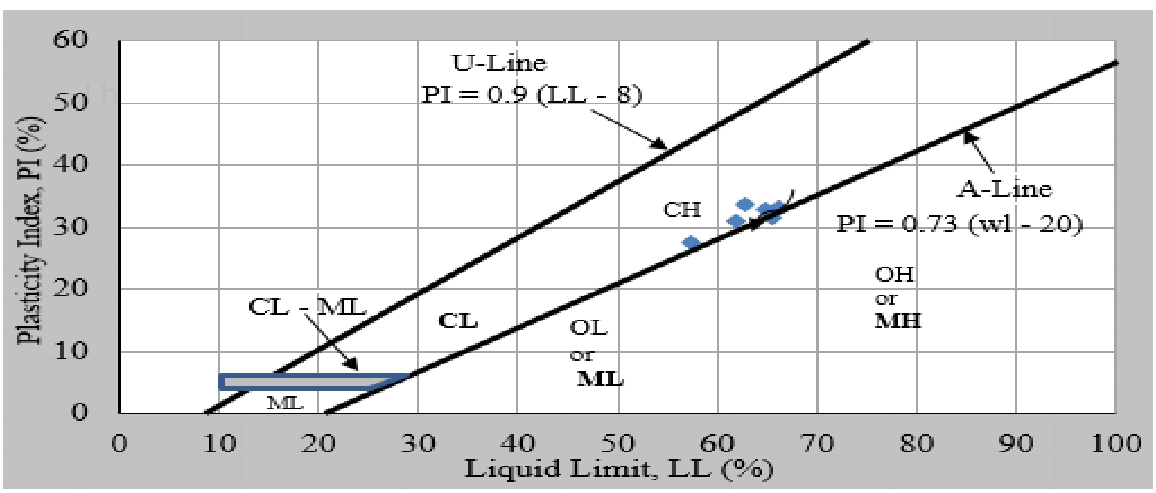


Figure 4.1 Plasticity Chart of the Study Area, ASTM Standard

4.2 Natural Moisture Content, w_N

The natural moisture content of soils of the study area is given in Table 4.3. The value of natural moisture content obtained for the soils range between 37.25 to 45.02 %. The result of natural moisture content of CTP1 increases with distance below ground surface and this indicates soil moisture content increase with depth.

Table 4.3 Natural Moisture Content, w_N of the soils of the study area

Test pit designation	Depth (m)	Natural moisture content, w_N (%)
TTP1	1-1.2	44.23
TTP2	1-1.2	44.88
TTP3	1-1.2	45.02
CTP1	1.3-1.5	37.25
	2.8-3.0	40.41
CTP2	2.3-2.5	38.49

4.3 Bulk Density and Unit Weight

Bulk density and unit weight of the soils of the study area is given in Table 4.4. The value of bulk density of the soils varies from 1.98 to 2.08 g/cm^3, unit weight 19.42 to 20.4 KN/m^3, saturated unit weight 18.43 to 19.3 KN/m^3 and dry unit weight 14.00 to 14.62 KN/m^3. The saturated and dry unit weight value of the soils fall in the range dry and saturated unit weight of silt and clay soil, dry and saturated unit weight of silt soil, 18 to 20 and 14 to 18 KN/m^3 respectively and dry and saturated unit weight of clay soil, 18 to 20 and 14 to 21 KN/m^3 respectively (Wiley, 2010). This shows that fine grained soil (silt and clay) may have a great probability to initiate the landslide in Lalisa village along with other triggering factors of landslide.

Table 4.4 Bulk Density and Unit Weight of the Soils of the Study Area

Pit designation	TTP1	TTP2	CTP1	CTP2	CTP3
Bulk density, (g/cm^3)	2.07	1.98	1.99	2.08	2.06
Unit weight, (KN/m^3)	20.31	19.42	19.52	20.4	20.21
Dry unit weight, (KN/m^3)	14.08	14.00	14.01	14.73	14.62
Saturated unit weight, (KN/m^3)	18.6	18.43	18.78	19.27	19.3

4.4 Specific Gravity

The specific gravity of soils of the study area is given in Table 4.5. The value of specific gravity gained for the soils range between 2.78 to 2.85. According to Das (2002), the value falls in the range specific gravity of clay and silty clay soil, 2.67 to 2.9. Therefore, the soils of Lalisa village are generally fine grained soils of inorganic clay that initiates the occurrence of landslide in the study area.

Table 4.5 Specific Gravity of the Soil of the Study Area

Test pit	Depth (m)	Specific gravity
TTP1	1-1.2	2.78
TTP2	1-1.2	2.8
TTP3	1-1.2	2.79
CTP1	1.3-1.5	2.80
	2.8-3	2.84
CTP2	2.3-2.5	2.83
CTP3	2.3-2.5	2.85

4.5 Grain Size Distribution

The combined grain size distribution curve for soil samples of the study area retained on No.200 sieve (0.075 mm) and passing No.200 sieve (0.0075 mm) is shown on Figure 4.2. The grain size distribution results of the soils of the study area shows, the soils of Lalisa village is the combination of coarse sand varies from 0 to 1 %, medium sand 0 to 3 %, fine sand 1 to 7 %, silt 20 to 46 % and clay varies from 43 to 77 %. The soils retained on sieve No. 200 (0.075 mm) was varies from 1 to 7 %. In relation to these results, the soils of Lalisa village is fine grained soil (silt and clay) which has high percentage of clay soil. As ASTM – D 2487 – 10, the soil type of the study area is fat clay as the soil retained on sieve No. 200 (0.075 mm) < 30 % as well as < 15 %. Thus, these soils are known to swell when wet and shrink in dry condition, as surface area of this soil particles increase water holding and swelling capacity increases, reduce the strength as moisture content increases and these factors may cause for the slope instability of Lalisa village.

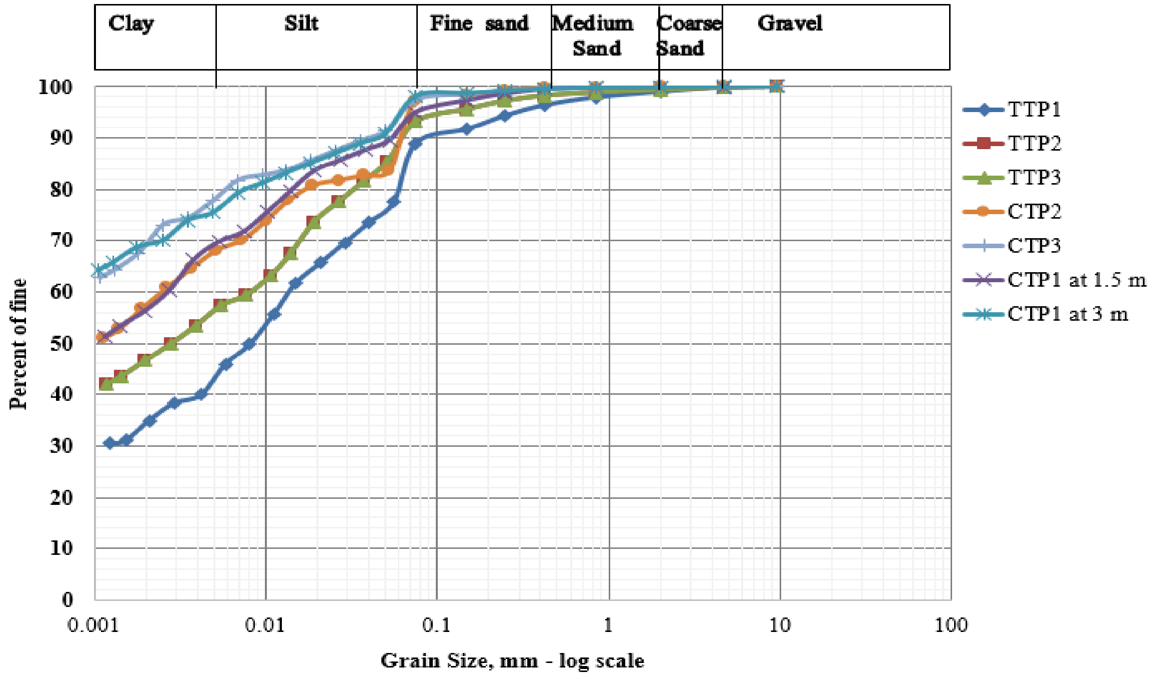


Figure 4.2 Combined grain size distribution curve for particles retained on No.200 sieve and passing No.200 sieve.

4.6 Free Swell

The free swell of soils of Lalisa village is given in Table 4.6. The value of free swell obtained for the soils of the study area varies from 47 to 55 %. The soils of the toe of the test pits has the ability to swell as compared to that of the crest test pits one.

Table 4.6 Free swell of soils of the study area

Pit Designation	Depth (m)	Free swell (%)
TTP1	1.2	48
TTP2	1.2	55
TTP3	1.2	60
CTP1	3	50
CTP2	2.5	50
CTP3	2.5	47

4.7 Unconfined Compressive Strength, UCS

The value of UCS of soils of the study area are given on Figure 4.3. As it was seen from the figure the value of UCS varies from 18.92 to 78 KN/m^2 and undrained cohesion which was half of UCS value varies from 9.46 to 39 KN/m^2. According to Taylor and Francis (2007), the value falls in the range of very soft and soft consistency of clay soil with UCS of 0 to 24 and 24 to 48 KN/m^2 respectively. The soil samples of CTP1, CTP2 and CTP3 were taken at a distance 10, 30 and 50 m from the failure surface of the slope and thus, UCS and consistency of clay increases as a distance from the failure surface increases as a result cohesion value. UCS value at both the toe of the slope and the first slope crest test pit was low and falls in very soft consistency that indicates the soil strength was affected either by water due to presence of spring at the slope toe or weathering which cause loss of shear strength hence, landslide in the study area.

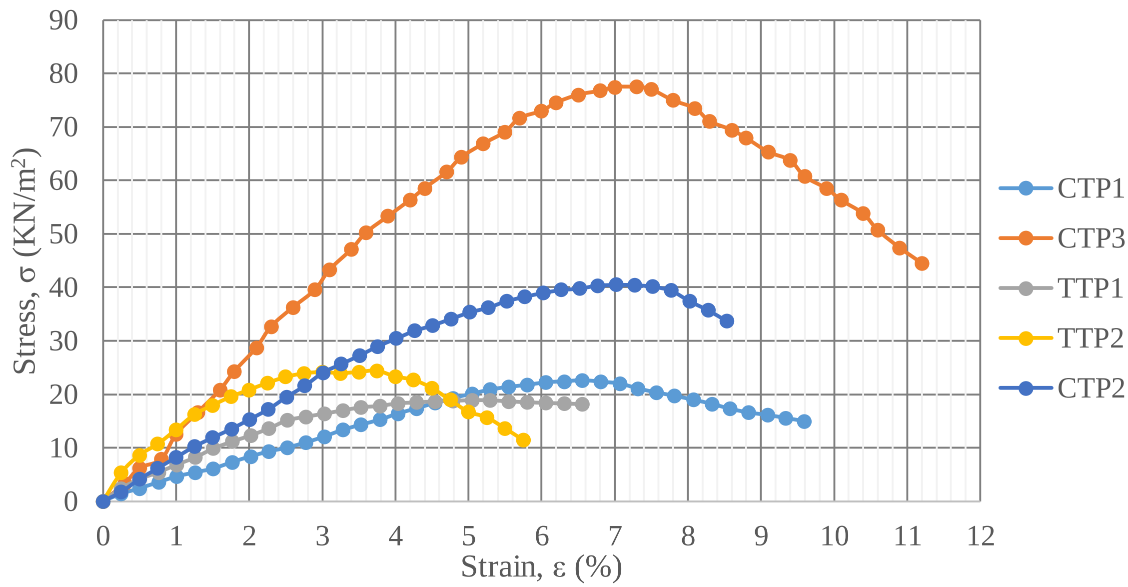


Figure 4.3 UCS Stress Strain Graph of TTP1, TTP2, CTP1, CTP2 and CTP3

4.8 Geo-Physical Resistivity

The result of geo-electrical resistivity survey at Lalisa village was explained with two methods (electrical resistivity profiling survey and vertical electrical soundings):

4.8.1 Electrical Resistivity Profiling Surveys

The electrical resistivity value of soils of the study area is given on Figure 4.4. Accordingly, the electrical resistivity value varies from 3.17 and 122 Ωm. The variation of resistivity value of soils of the study area at different depths along the profile line indicates variation in soil matrix, grain size distribution and water saturation within the depth of about 131.2 m. According to Solberg et al. (2012), the value falls in the range resistivity of unleached clay deposit that salt water, 1 to 10 Ωm, leached clay deposit, silt and fine grained till, 10 to 100 Ωm and bedrock, > 100 Ωm. Low resistivity but high conductivity value, 3.17 to 10 Ωm soils of the study area indicates presence of saturated soil, highly fractured and weathered rock which reflects weak zone or unstable zone of the soil layer between about 10 to 31 m depth at left hand side from Figure 4.4. In contrast, there was high resistivity, > 100 Ωm at a greater depth of below 43 m and near the surface soil at two points between 340 and 400 m and 40 and 80 m along the entire profile line with 7.5 m thickness. Thus, high resistivity value at a great depth indicates presence of hard strata or bedrock and unsaturated soil those are stable and at the near surface also indicates the availability of bedrocks exposed to the surface. The presence of water flow at two points, at about 3.5 and 24 m depth saturates the soil that increases soil weight, pore water pressure, decreases resistivity, soils shear strength which results landslide in Lalisa village along with other factors.

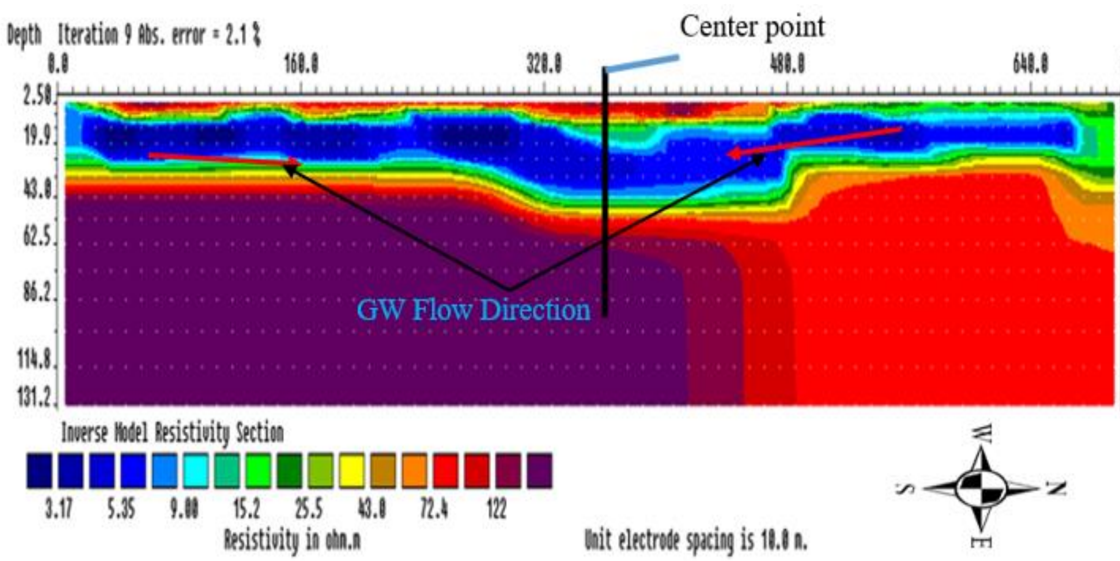


Figure 4.4 The inverse modeled resistivity section of electrical resistivity profiling along North to South of Lalisa village

4.8.2 Electrical Resistivity Sounding Surveys

The resistivity inversion model-VES along North to South of Lalisa Village is given on (Figure 4.5). Accordingly, there is four layers characterized by apparent resistivity of $\rho_1 < \rho_2 > \rho_3 < \rho_4$ and geo-electric cross- section of profile AB interpreted as: The first layer has 6.8 Ωm resistivity with 2.8 m thickness of top clay soil at 2.8 m depth, the second layer has

34.6 Ωm resistivity with 7.3 m thickness of moderately weathered and fractured rock at 10.1 m depth, the third layer has 3.8 Ωm resistivity with 25.1 m thickness of clay soil at 35.2 m depth and the last layer has a resistivity value of 247.8 Ωm. The first three layers are bedded with high resistivity of massive basaltic rock. When a clay soil and highly fractured and weathered rock in large thickness at a great depth of 35.2 m in contact with water, it increase weight and becomes slide, as well make the upper layer unstable. Hence, the existence of low resistivity value or unstable zone with a large thickness of third layer was considered as the main cause for landslide in the study area.

Table 4.7 Summarized lithology inversion result for VES at Lalisa Village

Layer	Resistivity (Ωm)	Thickness (m)	Depth (m)	Inferred Lithology	Remark
1	6.8	2.8	2.8	Top clay soil	Unstable with small thickness
2	34.6	7.3	10.1	Moderately fractured and weathered basaltic rock	Lightly stable with small thickness
3	3.8	25.1	35.2	Clay soil	Unstable with large thickness
4	247.8	-	-	Massive basaltic rock	Highly stable

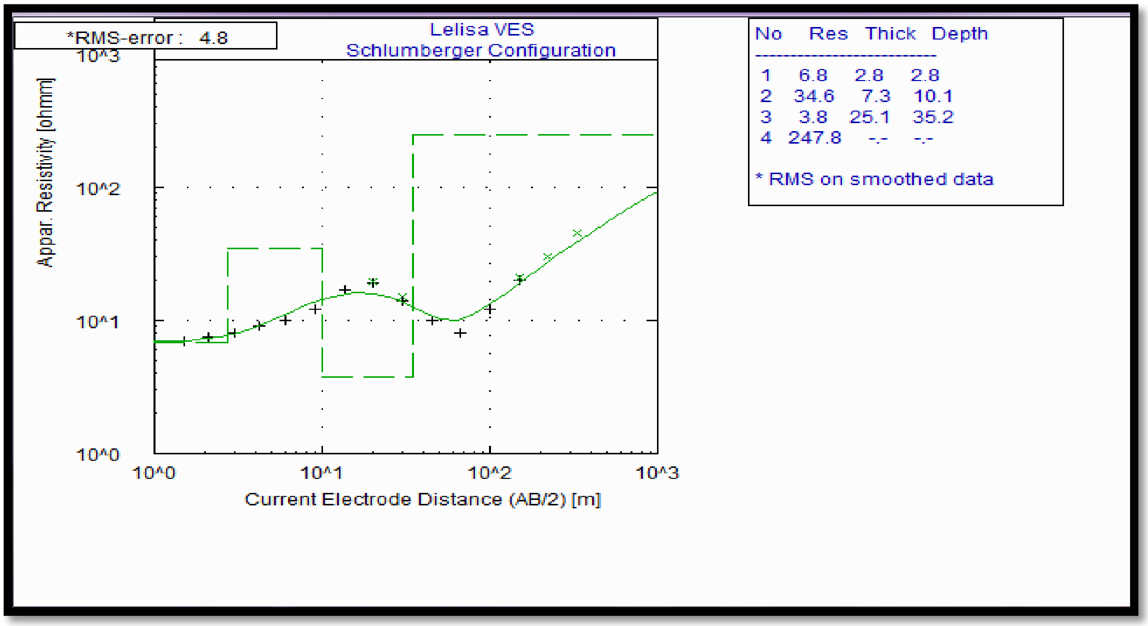


Figure 4.5 Resistivity Inversion Model-VES along North to South of Lalisa Village

4.9 Geologic Factor

The presence of weak zone (highly to moderately weathered and fractured rock) and clay soil with large thickness were the main causes for occurrence of landslide in the study area.

4.10 Slope Stability

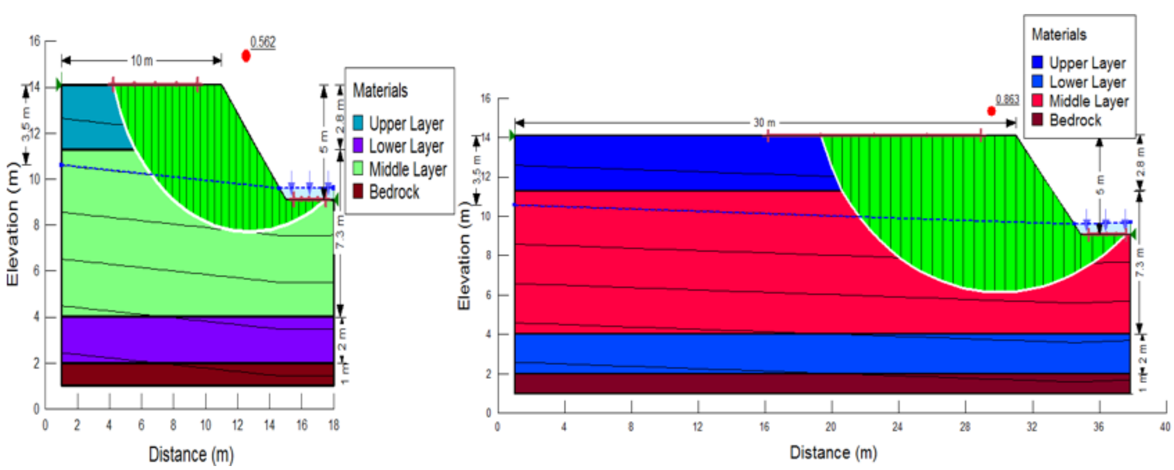
The critical slip surface, CSS and FOS of slope of the study area for two different conditions are presented on (Figure 4.6 and 4.7). The CSS passes below the toe of slope and its size also large. The bigger in size of slip surface and passing below the toe of the slope may due to ground water effect. The FOS value obtained varies from 0.562 to 1.063. According to Milimba (2007), the value falls in unsafe, FOS < 1 and approaching to failure that will fail at some time in response to the destabilizing forces attaining a critical level of activity, 1 < FOS < 1.5. FOS of natural slope of condition 1 was < 1 which is unsafe for both a and b condition but the FOS of condition b has a greater value, 0.863 (increase by 35 %) than condition a, 0.562 this may due to the distance from failure surface. Thus, as distance increase from failure surface the probability of slope damage by landslide less than the near failure surface. FOS of modified slope angle of condition 2 was < 1 for condition a is unsafe that requires making slope more flat than 30^0 and condition b is approaching to failure as its FOS < 1.5 (1.063). The increase in FOS of TTP2 and CTP2 in both natural slope and modified slope angle than TTP1 and CTP1 was as a result of distance from failure surface. Additionally, the increase in FOS from 0.562 to 0.644 (increase by 13 %) of TTP1 and CTP1 and 0.863 to 1.063 (increase by 19 %) of TTP2 and CTP2 were because of slope angle modification from 52 to 30^0. This indicates the great role of slope angle on slope stability. According to Broomhead (1997), making the slope gentle such that decreasing driving forces which causes increment of FOS of the slope. Generally, the slope of the study area is unsafe with the distance of 10 and 30 m from failure surface. Even though modifying slope angle increases FOS, it fails the slope in the state of unsafe and approaching to failure and the result tells as unsafe slope and minimum FOS of the slope of Lalisa village was due to effect of slope steepness along with other contributory factors of landslide. The finding also improves the role of geometry modification in prevention or remedial measure for landslide in the study area. Summary of slide mass, FOS and state of slope of both conditions shown in (Table 4.8).

Table 4.8 Summary of Slide Mass and FOS of Different Condition

Condition	Distance	TV	TW	TRM	TAM	FOS	Remark
1	10	45.67	911.5	1,360.70	2,420.60	0.562	Unsafe
	30	92.19	1823.6	3,372.10	3,905.30	0.863	Unsafe
2	10	45.25	906.48	1,541.90	2,391.10	0.644	Unsafe
	30	91.1	1802.8	6,437.60	6,057	1.063	AF

TV = Total volume, TW = Total Weight, TRM = Total Resisting Moment, TAM = Total Activating Moment, FOS = Factor of Safety and AF = Approaching to failure.

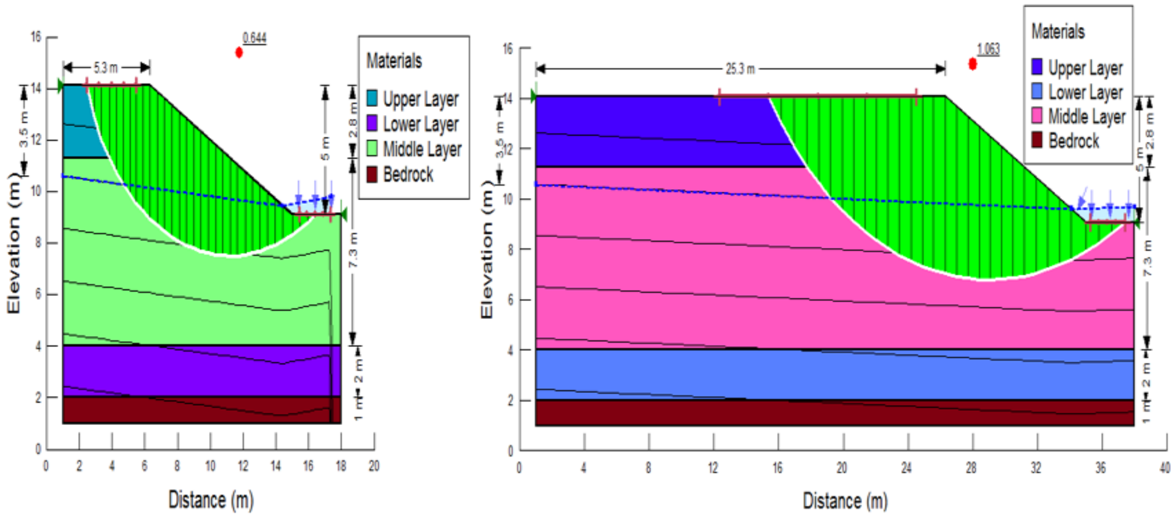
Condition 1: FOS Determination at a) TTP1 and CTP1 and b) TTP2 and CTP2 of Natural Slope



a) 10 m from failure surface condition b) 30 m from failure surface condition

Figure 4.6 Morgenstern-Price FOS for Natural Slope

Condition 2: FOS Determination at a) TTP1 and CTP1 and b) TTP2 and CTP2 of Modified Slope Angle



a) 10 m from failure surface condition b) 30 m from failure surface condition

Figure 4.7 Morgenstern-Price FOS for Modified Slope Angle

4.11 Causes and Triggering Factors of Landslide of the Study Area

Depending on laboratory, field and software result the following factors are considered as causal and triggering factor for the landslide in the study area.

4.11.1 Soil Type

The results shows that the soils of the study area clay soil. These soils are known to swell when wet and shrink in dry weather and commonly have multiple planes of weakness which initiates the occurrence of landslide in Lalisa village. Hence, the soil type of Lalisa village was the cause for landslide occurred in the study area.

4.11.2 Slope Steepness

The FOS value of the slope is unsafe in case of natural slope and it increases someone after modifying the slope angle. This indicates that the steep slope of Lalisa village is considered as another cause for slope instability in that area.

4.11.3 Deforestation

The information from farmers living around reveals that there were no landslide occurrence before 15 years which was the time were there was forest. The hazard of landslide exposed to the surface since 2010. This shows that deforestation for agricultural use considered as one factor for landslide occurred in the study area along with other factors.

4.11.4 Geological Factors

According to geophysical resistivity test result, there is weathered and fractured rock with a great depth which causes increase in weight of soil during wet condition and results slope instability in Lalisa village.

4.11.5 Rainfall

Rainfall is one of the main triggering factors to be considered in landslide of the study area. As information obtained from farmers, landslide happening was high in three months of March, April and May. These are the wettest months which have high intensity of rainfall that causes further saturation of the soil mass and displacing it further down which is the reason for support removal or undercutting of slopes then slope failures.

4.12 Type of Landslide

Landslide of the study area was classified as rotational as curved surface failure and produced slumps rotates along the slip surface by downward and outward movement of the soil mass as shown on (Figure 4.8). The slip surface was circular and critical slip surface, CSS passes below the toe of the slope (Figure 4.9); hence the failure classified as base failure which is one type of rotational failure for homogenous soil conditions; hence it was classified as rotational landslide.

Figure 4.8 Downward and Outward Movement of Soil Mass

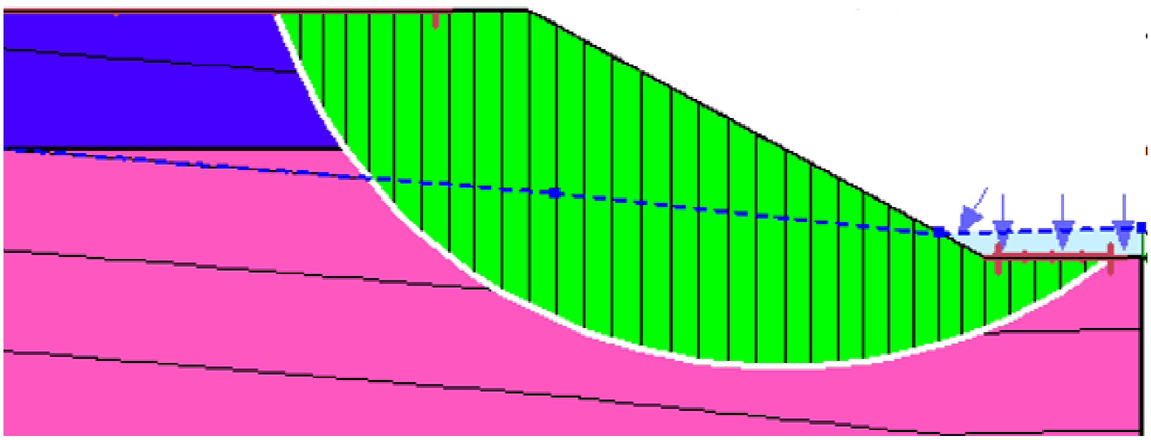


Figure 4.9 Base Failure

4.13 Consequences of Landslide

The effect of landslide of the study area as information obtained from farmers living around was evaluated. Even though no losses on human life due to landslide was reported in the study area, the impact was observed through (1) personal costs as it has made a person disable and five houses of families homeless with no damage on life in 2016. (2) Economic costs as the landslide made useless the crop and grazing land. This substantial damage to crops, grazing and farmland on the slopes is direct economic consequences of landslide at household level. In relation to this, 0.81 ha of bananas and sugarcane was damaged starting from the last 2016. Furthermore, 7 oxen in 2017, 15 oxen in 2016 and 2 goats in 2013 were killed due to landslide in this study area. These increases individual economic cost and poverty in to individuals in the society due to their limited livelihood sources and living only from their household farm. (3) Environmental damages on the society living around the study area as it has an effect on environmental through habitat degradation, removal of huge soil mass that affect farmland and fauna and flora by erosion during intense rainfall. Thus, the recent landslide covered an area of 104 ha. Due to landslide in Lalisa village about 0.48 M m^3 (multiplication of average length, width and depth of affected area surveyed) slope material was moved away till 2018.

4.14 Methods Proposed to Minimize the Effects of Landslide in the Study Area

Depending on the result of FOS of the slope of the study area, the following methods are proposed to prevent or minimize the effects of landslide in the study area.

4.14.1 Geometry modification

The slope of the study area is steep and this causes increase in tangential gravity force as a result, maximum value of shear stress which leads slope instability. Therefore, removing all or part of the earth driving landslide to modify slope geometry which is the most efficient way of increasing the factor of safety of a slope. The geometry of the slope can be modified by grading a slope angle to a uniform flatter angle, removing the material from the driving the landslide area.

4.14.2 Providing Drainage

The geo-morphology of the study area was located almost on sloped area and no drainage provided for taking erosion during intense raifall and flow on the slope valley to one direction, this makes the slope unstable against sliding. To minimize these problems providing unlined surface drainage along East to West and North to South at the upper side of slide area and controlling the runoff from upper course will minimize the continuity of landslide at Lalisa village.

4.14.3 Afforestation

There was no vegetation or the study area is bare land this indicates that the absence of support from tree root as a result unsafe against landslide in the study area. Therefore, covering the bare land with long rooted vegetation prohibits mass movements in the area thus, preventing landslide along with drainage structure.

4.14.4 Providing Engineering Structure

Providing engineering structures such as gabion and embankment for damaged area by landslide at Lalisa village. Thus, constructing gabion along both side of the slope to guide the soil movement and providing embankments along failed slope, with the size determined by the selection of gradient that produces a stable slope given the local hydraulic conditions.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Atterberg limit reveals that the soil was clay and characterized by high compressibility and high swelling potential, deforms like a plastic material; these soil characteristics have periodical swelling and shrinkage that activates landslide. Natural moisture content of the study area increase with depth improves the truth that soil water content increase as depth increase below the ground surface. The density and unit weight test result shows the soil of the study area categorized under fine grained soil (silt and clay). As soil classification, by ASTM D 2487-10 standard the soil of the study area was fallen under inorganic fat clay of high plasticity with a group symbol CH. The specific gravity value tells as the soil falls in the range specific gravity of clay and silty clay soil. According to unconfined compressive strength, cohesion value of soil near by the slope failure at the crest and toe of the slope was low as compared to those test pits far from the slope failure and this indicates that the strength of the soil close to the slope failure was affected than those at a certain distance.

Geo-electrical resistivity result shows that the geology of the study area was highly weathered and fractured rock, large thickness of clay soil at two different soil layer and massive basaltic rock at a greater depth. It also shows the presence of water flow at about 3.5 m depth. Thus, the presence of weak zone with large thickness and water flow enhance the slope instability in the study area.

From FOS result it can be understood that the slope of the study area classified as unsafe even with a distance of 30 m from failure surface. Unsafe slope or FOS < 1 obtained may be due to slope steepness, deforestation, clay soil type, rainfall and absence of drainage. .FOS also affected by distance from failure surface. As distance from failure surface increase the probability of soil strength reduction becomes less hence, high factor of safety. The FOS in gentle slope which is much greater than that of steep slope depicts as geometry modification used for prevention or remedial measure for landslide in the study area. In addition to modifying geometry providing drainage, planting and constructing engineering structures may be used as the prevention measures. The landslide type of the study area is base failure which is one type of translational slide. The failure become base failure as the CSS passes below the toe of the slope. The landslide at the study area have caused livelihood and human health disturbance, 104 ha of useless of farm and grazing land and animal life was damaged.

5.2 Recommendations

The following recommendations are forward based on the finding of the study:

1. Unsafe value of FOS was observed in the model under steep slope indicates that steep slope was a contributing factor to the slope instability. It was recommended that making the slope flatter than 30^0 used as a prevention and mitigation measures of the landslide at the study area.
2. According to laboratory and geophysical test result, the soils of the study area is clay soil which causes the periodical swell and shrink soil behavior that activates slope instability in the study area. Hence, treating soil with electro osmosis and stabilizing by grouting were recommended as a prevention and mitigation measure.
3. Making free of slope cultivation that accelerate soil erosion as a result slope failure. So making the land free from agricultural use with about 150 m radius from failure surface and planting it was recommended.

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APPENDICES

Appendix A Laboratory Analysis

A.1 Atterberg Limit Test for Air Dried Sample

Table A.1 Data Sheet for Liquid and Plastic Limit Test of TTP1

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	C8	G	T11	F1	Z-Z	I-I	T1G1
Wt. of container, W_C (g)	17.71	17.75	17.35	17.61	16.20	16.20	16.5
Wt. of wet soil + Cont., W_{wsc} (g)	48.79	53.18	46.34	51.31	28.14	27.50	28.09
Wt. of dry soil + Cont., W_{dsc} (g)	36.94	39.11	34.64	37.51	25.08	24.92	25.18
Wt. of water, W_w (g)	11.85	14.07	11.70	13.80	3.06	2.58	2.91
Wt. of dry soil, W_d (g)	19.23	21.36	17.29	19.90	8.88	8.72	8.68
No. blows	35	29	22	17			
Moisture content, W_c (%)	61.62	65.87	67.67	69.35	34.46	29.59	33.53
	LL= 66.15 %				PL= 32.53 %		
	PI= LL-PL = 33.62 %						

Table A.2 Data Sheet for Liquid and Plastic Limit Test of TTP2

	Liquid Limit, LL				Plastic Limit, PL		
Trial number	1	2	3	4	1	2	3
Container code	DN4	101	NTB	ATR	CHA	LC33	Z-Z
Wt. of container, W_C (g)	17.54	18.07	17.60	16.96	26.00	25.42	17.20
Wt. of wet soil + Cont., W_{wsc} (g)	39.89	45.78	38.71	42.32	37.50	35.21	29.36
Wt. of dry soil + Cont., W_{dsc} (g)	31.21	34.86	29.98	31.56	35.13	33.48	26.01

Wt. of water, W_w (g)	8.68	10.92	8.73	10.76	2.37	2.52	3.35
Wt. of dry soil, W_d (g)	13.67	16.79	12.38	14.60	9.13	8.06	8.81
No. blows	28	23	18	15			
Moisture content, W_c (%)	63.50	65.04	70.52	73.70	25.96	31.27	38.02
	LL = 65.79 %				PL = 31.75 %		
	PI = LL – PL = 34.04 %						

Table A.3 Data Sheet for Liquid and Plastic Limit Test of TTP3

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	N2	PL1	X	T	T67	T8	L3B2
Wt. of container, W_C (g)	6.17	6.07	6.00	6.20	6.52	6.17	6.29
Wt. of wet soil + Cont., W_{wsc} (g)	23.92	24.14	24.52	25.18	15.21	13.78	13.47
Wt. of dry soil + Cont., W_{dsc} (g)	17.00	16.92	17.00	17.35	12.06	12.01	11.72
Wt. of water, W_w (g)	6.92	7.22	7.52	7.83	2.35	1.77	1.75
Wt. of dry soil, W_d (g)	10.83	10.85	11.00	11.15	6.34	5.84	5.43
No. blows	34.00	23.00	17.00	15.00			
Moisture content, W_c (%)	63.90	66.54	68.36	70.22	37.07	30.31	32.23
	LL = 66.21 %				PL = 33.20 %		
	PI = LL – PL = 33.01 %						

Table A.4 Data Sheet for Liquid and Plastic Limit Test of CTP1 at 1.5 m Depth

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	LC31	PL13	DH	U	CO5	NTB	DN4
Wt. of container, W_C (g)	17.42	25.72	16.97	17.23	17.23	16.20	16.24
Wt. of wet soil + Cont., W_{wsc} (g)	42.78	47.05	43.92	45.14	29.00	27.42	28.13

Wt. of dry soil + Cont., W_{dsc} (g)	33.97	39.42	33.99	34.42	26.45	24.68	25.34
Wt. of water, W_w (g)	8.81	7.63	9.93	10.72	2.55	2.64	2.79
Wt. of dry soil, W_d (g)	16.55	13.70	17.02	17.19	9.22	8.48	9.10
No. blows	32.00	29.00	23.00	16.00			
Moisture content, W_c (%)	53.23	55.69	58.34	62.36	27.66	32.31	30.66
	LL = 57.00 %				PL = 30.21 %		
	PI = LL – PL = 26.79 %						

Table A.5 Data Sheet for Liquid and Plastic Limit Test of CTP1 at 3 m Depth

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	BA	L3I2	DH	OY	2	C	B2
Wt. of container, W_C (g)	18.05	17.79	16.96	17.85	6.24	5.99	6.26
Wt. of wet soil + Cont., W_{wsc} (g)	37.87	39.40	36.27	40.42	12.67	12.79	14.57
Wt. of dry soil + Cont., W_{dsc} (g)	30.11	30.82	28.45	31.15	10.98	11.07	12.47
Wt. of water, W_w (g)	7.76	8.58	7.82	9.27	1.69	1.72	2.10
Wt. of dry soil, W_d (g)	12.06	13.03	11.49	13.30	4.74	5.08	6.21
No. blows	30.00	22.00	18.00	15.00			
Moisture content, W_c (%)	64.34	65.85	68.06	69.70	35.65	33.86	33.82
	LL = 66 %				PL = 34.44 %		
	PI = LL – PL = 31.56 %						

Table A.6 Data Sheet for Liquid and Plastic Limit Test of CTP2

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	C05	C8	PL13	ATR	P2	MPL2	AD
Wt. of container, W_C (g)	18.69	17.71	26.01	19.32	5.91	6.22	6.34

Wt. of wet soil + Cont., W_{wsc} (g)	35.60	37.23	46.65	41.34	13.43	12.35	18.18
Wt. of dry soil + Cont., W_{dsc} (g)	29.28	29.74	38.23	32.27	11.70	10.89	15.64
Wt. of water, W_w (g)	6.32	7.49	8.42	9.07	1.73	1.46	2.54
Wt. of dry soil, W_d (g)	10.59	12.03	12.22	12.95	5.79	4.67	9.30
No. blows	31.00	23.00	18.00	15.00			
Moisture content, W_c (%)	59.68	62.26	68.90	70.04	29.88	31.26	27.31
	LL = 63 %				PL = 29.48 %		
	PI = LL – PL = 33.52 %						

Table A.7 Data Sheet for Liquid and Plastic Limit Test of CTP3

	Liquid Limit				Plastic Limit		
Trial number	1	2	3	4	1	2	3
Container code	HC23	NC71	G53	QWE	T1C1	W-X	I-I
Wt. of container, W_C (g)	19.15	17.91	17.46	19.12	17.65	18.06	17.44
Wt. of wet soil + Cont., W_{wsc} (g)	40.17	40.43	44.54	43.24	23.75	25.09	27.91
Wt. of dry soil + Cont., W_{dsc} (g)	32.33	31.73	33.91	33.57	22.21	23.24	25.90
Wt. of water, W_w (g)	7.84	8.70	10.63	9.67	1.54	1.85	2.01
Wt. of dry soil, W_d (g)	13.18	13.82	16.45	14.45	4.56	5.18	8.46
No. blows	29.00	24.00	20.00	16.00			
Moisture content, W_c (%)	59.48	62.95	64.62	66.92	33.77	35.71	23.76
	LL = 62 %				PL = 31.08 %		
	PI = LL – PL = 30.92 %						

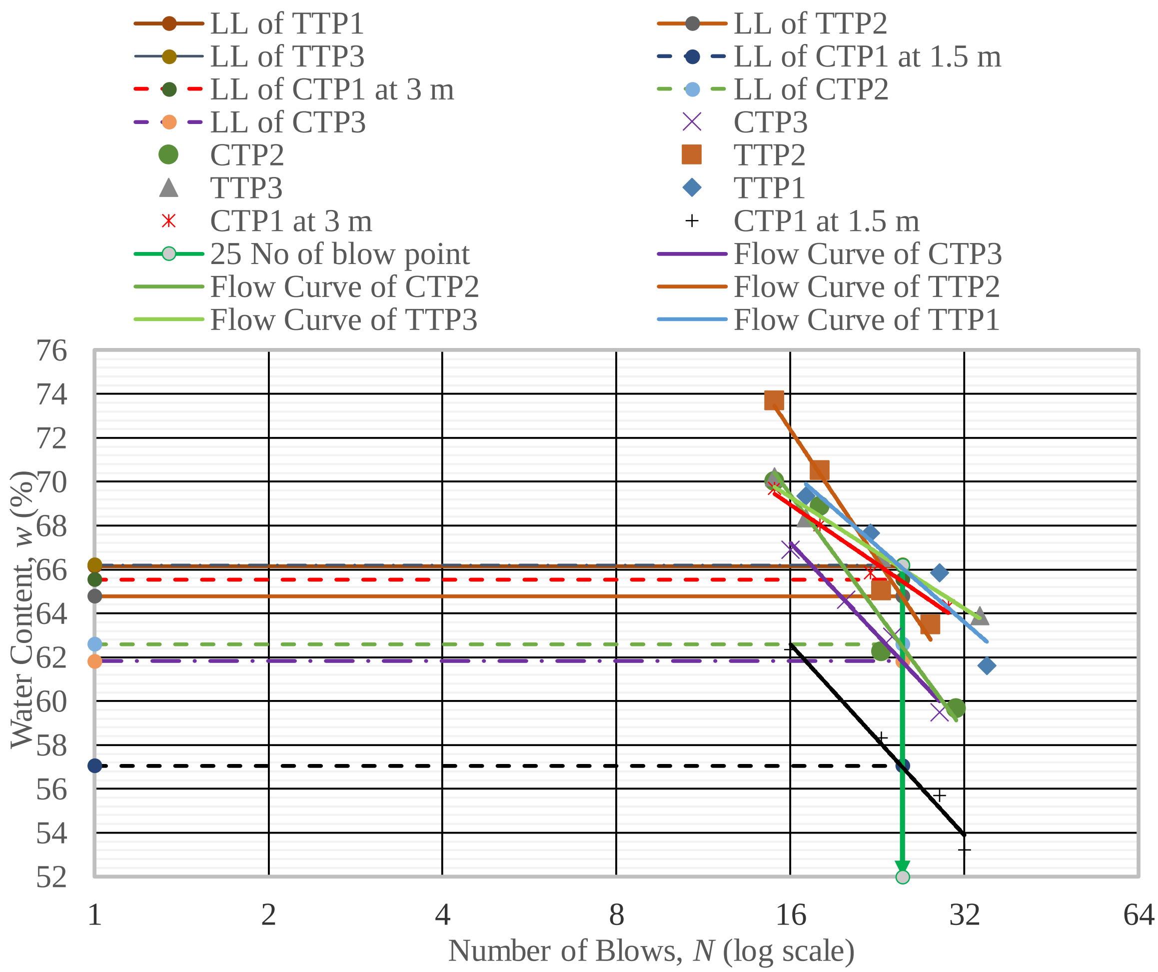

Figure A.1 Combined LL Graph of All Test Pits of Air Dried Samples

A.2 Atterberg Limit Test for Oven Dried Sample

Table A.8 Data Sheet for Oven Dried Liquid Limit of TTP1

	Liquid Limit, LL			
Trial number	1	2	3	4
Container code	206	SM	IA	T3
Wt. of container, W_C (g)	16.70	17.30	17.60	17.90
Wt. of wet soil + Cont., W_{wsc} (g)	62.90	61.10	59.40	59.90
Wt. of dry soil + Cont., W_{dsc} (g)	46.00	44.40	43.20	43.50
Wt. of water, W_w (g)	16.90	16.70	16.20	16.40
Wt. of dry soil, W_d (g)	29.30	27.10	25.60	25.60

No. blows	31.00	22.00	17.00	14.00
Moisture content, W_c (%)	57.68	61.62	63.28	64.06
	LL = 60.00 %			

Table A.9 Data Sheet for Oven Dried Liquid Limit of TTP2

	Liquid limit			
Trial number	1	2	3	4
Container code	HC12	69	P8	C2
Wt. of container, W_C (g)	18.10	25.30	19.30	17.50
Wt. of wet soil + Cont., W_{wsc} (g)	56.20	64.90	61.00	60.00
Wt. of dry soil + Cont., W_{dsc} (g)	42.50	50.40	45.30	43.40
Wt. of water, W_w (g)	13.70	14.50	15.70	16.60
Wt. of dry soil, W_d (g)	24.40	25.10	26.00	25.90
No. blows	30	24	18	15
Moisture content, W_c (%)	56.15	57.77	60.38	64.09
	LL = 58.00 %			

Table A.10 Data Sheet for Oven Dried Liquid Limit of CTP1at 3 m depth

	Liquid Limit			
Trial number	1	2	3	4
Container code	DH	42	B	I
Wt. of container, W_C (g)	17.00	22.70	17.60	18.50
Wt. of wet soil + Cont., W_{wsc} (g)	59.30	67.50	61.40	68.30
Wt. of dry soil + Cont., W_{dsc} (g)	45.00	52.00	45.50	50.00
Wt. of water, W_w (g)	14.30	15.50	15.90	18.30
Wt. of dry soil, W_d (g)	28.00	29.30	27.90	31.50
No. blows	30	26	18	15
Moisture content, W_c (%)	51.07	52.90	56.99	58.10
	LL = 53.00 %			

Table A.11 Data Sheet for Oven Dried LL of CTP2

	Liquid Limit			
Trial number	1	2	3	4
Container code	P4	LC42	WX	K-1
Wt. of container, W_C (g)	24.60	17.60	17.90	17.40
Wt. of wet soil + Cont., W_{wsc} (g)	60.50	64.70	53.10	58.20
Wt. of dry soil + Cont., W_{dsc} (g)	48.86	49.20	41.40	44.30
Wt. of water, W_w (g)	11.64	15.50	11.70	13.90
Wt. of dry soil, W_d (g)	24.26	31.60	23.50	26.90
No. blows	31	24	19	14
Moisture content, W_c (%)	47.98	49.05	49.79	51.67
	LL = 49.00 %			

Table A.12 Data Sheet for Oven Dried Liquid Limit of CTP3

	Liquid Limit			
Trial number	1	2	3	4
Container code	HC23	NC71	G53	QWE
Wt. of container, W_C (g)	17.70	18.10	17.50	17.60
Wt. of wet soil + Cont., W_{wsc} (g)	38.40	40.10	36.00	40.80
Wt. of dry soil + Cont., W_{dsc} (g)	31.50	32.50	29.50	32.10
Wt. of water, W_w (g)	6.90	7.60	6.50	8.70
Wt. of dry soil, W_d (g)	13.80	14.40	12.00	14.50
No. blows	33.00	28.00	20.00	15.00
Moisture content, W_c (%)	50.00	52.78	54.17	60.00
	LL = 53.00 %			

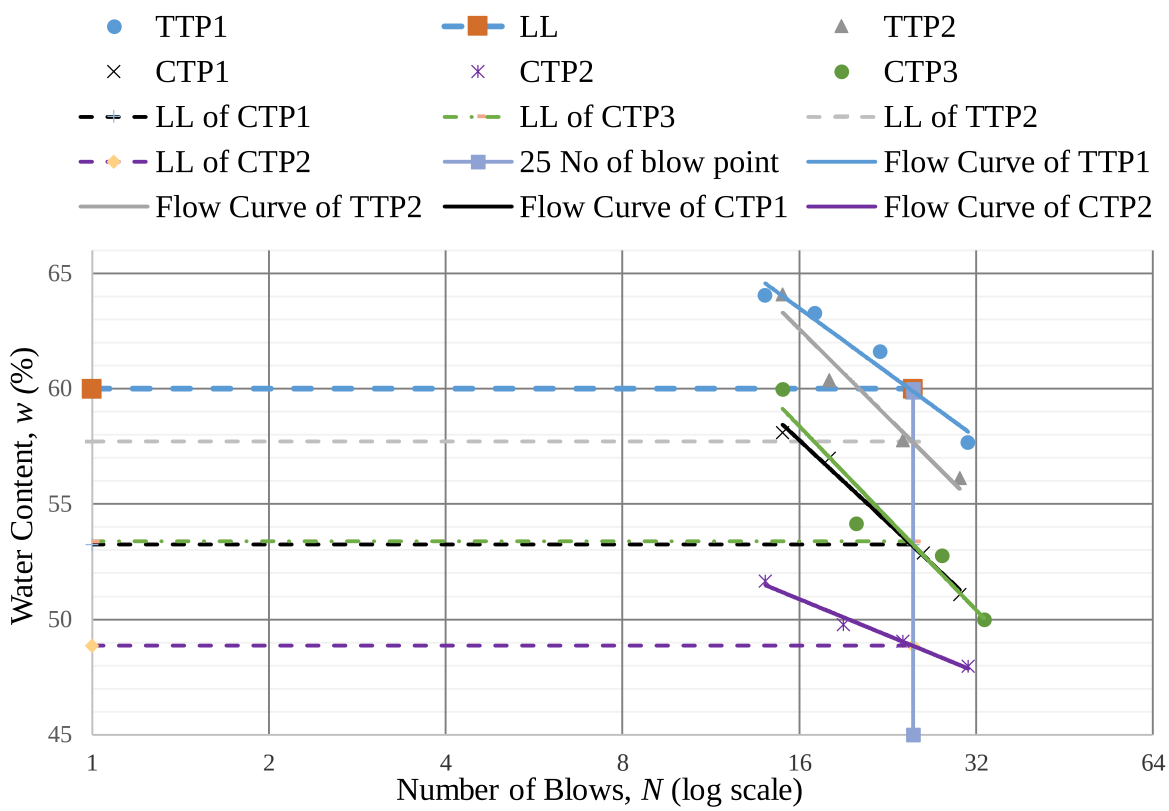

Figure A.2 Combined LL Graph of All Test Pits of Oven Dried Samples

A.3 Natural Moisture Content

Table A.13 Data Sheet for Natural Moisture Content

Pit designation	TTP1	TTP2	TTP3	CTP1	CTP1	CTP2	CTP3
Depth (m)	1-1.2	1-1.2	1-1.2	1.3-1.5	2.8-3	2.3-2.5	2.3-2.5
Container code	D12	MK	T11	T1C	T5C1	LC51	T1C1
Wt. of container, W_C (g)	18.37	17.56	17.35	17.69	17.53	21.655	17.91
Wt. of wet soil + Cont., W_{wsc} (g)	97.91	103.06	46.34	110.99	98.14	104.53	107.93
Wt. of dry soil + Cont., W_{dsc} (g)	73.52	76.59	34.64	85.67	74.94	81.50	83.88
Wt. of water, W_w (g)	24.40	26.47	11.70	25.32	23.20	23.03	24.05
Wt. of dry soil, W_d (g)	55.15	59.04	17.29	67.98	57.41	59.84	65.97

Natural moisture content W_N (%)	44.23	44.83	45.02	37.25	40.41	38.49	36.46

A.4 Bulk Density and Unit Weight Analysis

Table A.14 Data Sheet for Bulk Density and Unit Weight Analysis of TTP1 and TTP2

	TTP1		TTP2	
Trial	1	2	1	2
Diameter of sample, D_s (mm)	36	36	36	36
Height of sample, H (mm)	78.5	80	79	81
Average height of sample, H_s (mm)	79.25		80	
Volume of sample, V_s (mm^3)	80666.67		81430.08	
Total mass of sample, M (g)	165.2	168.52	164.62	158.53
Average total mass, M_s (g)	166.86		161.58	
Bulk density, ρ_b (g/cm^3)	2.07		1.98	
Unit weight, γ_b (KN/m^3)	20.31		19.42	
Water content, w (%)	44.23		44.83	
Dry unit weight, γ_d (KN/m^3)	14.08		14.00	
Void ratio, e (%)	98.57		104.83	
Saturated unit weight, γ_{sat} (KN/m^3)	18.6		18.43	
Degree of saturation, %	100		100	

Table A.15 Data Sheet for Bulk Density and Unit Weight Analysis of CTP1, CTP2 and CTP3

	CTP1		CTP2		CTP3	
Trial	1	2	1	2	1	2
Diameter of sample, D_s (mm)	36	36	36	36	36	36
Height of sample, H (mm)	78	81.5	81	77.5	80	74
Average height of sample, H_s (mm)	79.75		79.25		77	
Volume of sample, V_s (mm^3)	81175.61		80666.67		78376.45	
Total mass of sample, M (g)	157.6	165	172	162.7	158	164.34
Average total mass, M_s (g)	161.415		167.56		161.17	

Bulk density, ρ_b (g/cm^3)	1.99	2.08	2.06
Unit weight, γ_b (KN/m^3)	19.52	20.4	20.21
Water content, w (%)	40.41	38.49	38.21
Dry unit weight, γ_d (KN/m^3)	14.01	14.73	14.62
Void ratio, e (%)	101.14	89.81	91.23
Saturated unit weight, γ_{sat} (KN/m^3)	18.78	19.27	19.3
Degree of saturation, %	100	100	100

A.5 Specific Gravity Test Analysis

Table A.16 Specific Gravity of TTP1 and TTP2

Test pit designation	TTP1			TTP2		
Depth (m)	1-1.2			1-1.2		
Trial No.	1	2	3	1	2	3
Code	1	2	3	A	B	C
Mass of empty pycnometer, Wp (g)	28.56	28.41	27.66	27.95	28.31	28.68
Mass of pycnometer+ dry soil (g), Wps	38.51	38.44	37.72	37.94	38.14	38.84
Mass of pycnometer + dry soil + water (g), Wpsw	86.17	85.94	84.59	84.45	85.52	85.01
Mass of pycnometer + water (g), Wpw	79.9	79.4	78.17	78.17	79.4	78.17
Ti,	26.5	26.5	26.5	23.5	23.5	23.5
Tx	27	27	27	25.2	25.2	25.2
Specific gravity, Gs	2.7	2.87	2.76	2.69	2.65	3.06
Specific gravity, Gs at 20 C^0	2.78			2.8		

Table A.17 Specific Gravity of TTP3 and CTP1

Test pit designation	TTP3			CTP1		
Depth (m)	1-1.2			1.3-1.5		
Trial No.	1	2	3	1	2	3
Code	1	2	3	A	B	C
Mass of empty pycnometer, Wp (g)	27.68	28.90	28.10	29.00	28.32	27.60
Mass of pycnometer+ dry soil (g), Wps	37.69	39.00	38.10	39.10	38.33	37.60

Mass of pycnometer + dry soil + water (g), Wpsw	84.65	86.50	85.70	86.50	86.00	84.30
Mass of pycnometer + water (g), Wpw	78.17	79.90	79.40	79.90	79.40	78.20
Ti,	24.80	24.80	24.80	26.20	26.20	26.20
Tx	27.40	27.40	27.40	28.20	28.20	28.20
Specific gravity, Gs	2.84	2.89	2.67	2.89	2.94	2.59
Specific gravity, Gs at 20 C^0	2.79			2.80		

Table A.18 Specific Gravity of CTP1, CTP2 and CTP3

Test pit	CTP1			CTP2			CTP3		
Depth (m)	2.8-3			2.3-2.5			2.3-2.5		
Trial No.	1	2	3	1	2	3	1	2	3
Code	1	2	3	A	B	C	1	2	3
Wp (g)	28.41	27.49	28.73	28.86	28.05	28.16	28.49	28.67	27.17
Wps (g)	38.56	37.62	38.82	38.86	38.1	38.24	38.51	38.68	37.79
Wpsw (g)	86.21	84.44	86.46	86.54	84.51	85.87	85.99	86.04	84.82
Wpw (g)	79.40	78.17	79.90	79.90	78.17	79.40	79.40	79.40	78.17
Ti,	22	22	22	22.5	22.5	22.5	26.5	26.5	26.5
Tx	27	27	27	25	25	25	28	28	28
Gs	3.04	2.62	2.86	2.98	2.71	2.79	2.92	2.97	2.68
Gs at 20 C^0	2.84			2.83			2.85		

Wp = Mass of empty pycnometer, Wps = Mass of pycnometer+ dry soil, Wpsw = Mass of pycnometer + dry soil + water, Wpw = Mass of pycnometer + water and Gs = Specific gravity.

A.6 Grain Size Distribution Analysis

Table A.19 Data Sheet for Wet Sieve Analysis TTP1 and TTP2

Weight of sample before wash = 1000 g for all test

TTP1	Sieve No.	SS, mm	MR, g	% R	% P	TTP2	SS, mm	MR, g	% R	% P
	3/8"	9.5	0	0	100		9.5	0	0	100
	No. 4	4.75	1.91	0.19	99.81		4.75	1.8	0.18	99.82

	No. 10	2	8.97	0.9	98.91		2	5.62	0.56	99.26
	No. 20	0.85	10.81	1.08	97.83		0.85	4.35	0.44	98.82
	No. 40	0.425	15.13	1.51	96.32		0.425	6.52	0.65	98.17
	No. 60	0.25	20.13	2.01	94.31		0.25	10.16	1.02	97.15
	No. 100	0.15	26.22	2.62	91.69		0.15	15.79	1.58	95.57
	No. 200	0.075	27.66	2.77	88.92		0.08	24.13	2.41	93.16
		pan	889.2	88.92	0		pan	931.63	93.16	0

SS = Sieve size, MR = Mass retained, % R = percentage of retained and % P = percentage of pass

Table A.20 Data Sheet for Wet Sieve Analysis of TTP3 and CTP1 at 1.5 m Depth

TTP3	Sieve No.	SS, mm	MR, g	% R	% P	CTP1 at 1.5 m depth	SS, mm	MR, g	% R	% P
	3/8"	9.5	0	0	100		9.5	0	0	100
	No. 4	4.75	1.74	0.17	99.83		4.75	0.4	0.04	99.96
	No. 10	2	6.92	0.69	99.14		2	0.38	0.04	99.92
	No. 20	0.85	9.77	0.98	98.16		0.85	1.01	0.1	99.82
	No. 40	0.425	10.69	1.07	97.09		0.425	3.85	0.39	99.43
	No. 60	0.25	10.51	1.05	96.04		0.25	8.55	0.86	98.57
	No. 100	0.15	11.63	1.16	94.88		0.15	13.21	1.32	97.25
	No. 200	0.075	17.84	1.78	93.1		0.075	22.52	2.25	95
		pan	930.9	93.09	0		pan	950.08	95.01	0

SS = Sieve size, MR = Mass retained, % R = percentage of retained and % P = percentage of pass

Table A.21 Data Sheet for Wet Sieve Analysis of CTP1 at 3 m Depth and CTP2

CTP1	Sieve No.	SS, mm	MR, g	% R	% P	CTP2	SS, mm	MR, g	% R	% P
	3/8"	9.5	0	0	100		9.5	0	0	100

	No. 4	4.75	0	0	100	4.75	0	0	100
	No. 10	2	0.47	0.05	99.95	2	0.1	0.01	99.99
	No. 20	0.85	1.66	0.17	99.78	0.85	0.7	0.07	99.92
	No. 40	0.425	2.65	0.27	99.51	0.425	1.79	0.18	99.74
	No. 60	0.25	3.84	0.38	99.13	0.25	4.96	0.5	99.24
	No. 100	0.15	4.55	0.46	98.67	0.15	9.7	0.97	98.27
	No. 200	0.075	5.53	0.55	98.12	0.075	9.25	0.93	97.34
		pan	981.3	98.13	0	pan	973.5	97.35	0

Table A.22 Data Sheet for Wet Sieve Analysis of CTP3 and Slope

	Sieve No.	SS, mm	MR, g	% R	% P		SS, mm	MR, g	% R	% P
CTP3	3/8"	9.5	0	0	100	Slope	9.5	0	0	100
	No. 4	4.75	0	0	100		4.75	0.97	0.1	99.9
	No. 10	2	0.11	0.01	99.99		2	7.82	0.78	99.12
	No. 20	0.85	0.91	0.09	99.9		0.85	8.85	0.89	98.23
	No. 40	0.425	2.46	0.25	99.65		0.425	9.28	0.93	97.3
	No. 60	0.25	5.18	0.52	99.13		0.25	9.1	0.91	96.39
	No. 100	0.15	6.84	0.68	98.45		0.15	12.48	1.25	95.14
	No. 200	0.075	10.72	1.07	97.38		0.075	18.92	1.89	93.25
		pan	973.78	97.38	0		pan	932.58	93.26	0

SS = Sieve size, MR = Mass retained, % R = percentage of retained and % P = percentage of pass

Hydrometer analysis uses ASTEM-D-422

Sample depth (m): 1-1.2

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.78

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.23 Data Sheet for Hydrometer Analysis of TTP1

	HR	T	CHR	Cf (a)		K		% P

Time (minutes)			R'	R"		EDR (L)		DSP (mm)	
0.5	45	21	46	39.2	0.9724	8.9	0.01300	0.0548	76.23
1	43	21	44	37.2	0.9724	9.2	0.01300	0.0394	72.34
2	41	21	42	35.2	0.9724	9.6	0.01300	0.0285	68.46
4	39	21	40	33.2	0.9724	9.9	0.01300	0.0205	64.57
8	37	21	38	31.2	0.9724	10.2	0.01300	0.0147	60.68
15	34	21	35	28.2	0.9724	10.7	0.01300	0.0110	54.84
30	31	21	32	25.2	0.9724	11.2	0.01300	0.0079	49.01
60	29	21	30	23.2	0.9724	11.5	0.01300	0.0057	45.12
120	26	21	27	20.2	0.9724	12	0.01300	0.0041	39.28
240	25	22	26	19.4	0.9724	12.2	0.01285	0.0029	37.73
480	23	23	24	17.7	0.9724	12.5	0.01269	0.0020	34.42
1440	21	22	23	15.4	0.9724	12.9	0.01285	0.0012	29.95

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of finer

Sample depth (m): 1-1.2

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.80

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.24 Data Sheet for Hydrometer Analysis of TTP2

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	48	25	49	43.48	0.9685	8.4	0.01233	0.0505	84.22
1	46	25	47	41.48	0.9685	8.8	0.01233	0.0366	80.35
2	44	25	45	39.48	0.9685	9.1	0.01233	0.0263	76.47
4	42	25	43	37.48	0.9685	9.4	0.01233	0.0189	72.60
8	39	25	40	34.3	0.9685	9.9	0.01233	0.0137	66.44

15	38	21	39	32.2	0.9685	10.1	0.01293	0.0106	62.37
30	36	21	37	30.2	0.9685	10.4	0.01293	0.0076	58.50
60	35	21	36	29.2	0.9685	10.6	0.01293	0.0054	56.56
120	33	21	34	27.2	0.9685	10.9	0.01293	0.0039	52.69
240	31	22	32	25.4	0.9685	11.2	0.01278	0.0028	49.20
480	29	23	30	23.7	0.9685	11.5	0.01262	0.0020	45.91
1440	27	22	28	21.4	0.9685	11.9	0.01278	0.0012	41.45

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine

Sample depth (m): 1-1.2

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.79

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.25 Data Sheet for Hydrometer Analysis of TTP3

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	49	21	50	43.2	0.9704	8.3	0.01297	0.0528	83.84
1	47	21	48	41.2	0.9704	8.6	0.01297	0.0380	79.96
2	45.5	21	47	39.7	0.9704	8.85	0.01297	0.0273	77.05
4	44	21	45	38.2	0.9704	9.1	0.01297	0.0196	74.14
8	42	21	43	36.2	0.9704	9.4	0.01297	0.0141	70.26
15	41	21	42	35.2	0.9704	9.6	0.01297	0.0104	68.32
30	39	21	40	33.2	0.9704	9.9	0.01297	0.0075	64.44
60	38	21	39	32.2	0.9704	10.1	0.01297	0.0053	62.50
120	36	22	37	30.4	0.9704	10.4	0.01281	0.0038	59.00
240	34	22	35	28.4	0.9704	10.7	0.01281	0.0027	55.12
480	33	23	34	27.7	0.9704	10.9	0.01266	0.0019	53.76
1440	30	22	31	24.4	0.9704	11.4	0.01281	0.0011	47.36

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine

Sample depth (m): 1.3-1.5

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.80

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.26 Data Sheet for Hydrometer Analysis of CTP1

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	51	21	52	45.2	0.9647	7.9	0.01286	0.0511	87.21
1	50	21	51	44.2	0.9647	8.1	0.01286	0.0366	85.28
2	49	21	50	43.2	0.9647	8.3	0.01286	0.0262	83.35
4	48	21	49	42.2	0.9647	8.4	0.01286	0.0186	81.42
8	46	21	47	40.2	0.9647	8.8	0.01286	0.0135	77.56
15	44	21	45	38.2	0.9647	9.1	0.01286	0.0100	73.70
30	42	21	43	36.2	0.9647	9.4	0.01286	0.0072	69.84
60	41	21	42	35.2	0.9647	9.6	0.01286	0.0051	67.91
120	39	22	40	33.4	0.9647	9.9	0.01271	0.0037	64.44
240	36	22	37	30.4	0.9647	10.4	0.01271	0.0026	58.65
480	34	22	35	28.4	0.9647	10.7	0.01271	0.0019	54.79
1440	31.5	22	33	25.9	0.9647	11.15	0.01271	0.0011	49.97

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine.

Sample depth (m): 2.7-3.0

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.84

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.27 Data Sheet for Hydrometer Analysis of CTP1

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	53	21	54	47.2	0.9610	7.6	0.01279	0.0499	90.72
1	52	21	53	46.2	0.9610	7.8	0.01279	0.0357	88.79
2	51	21	52	45.2	0.9610	7.9	0.01279	0.0254	86.87
4	50	21	51	44.2	0.9610	8.1	0.01279	0.0182	84.95
8	49	21	50	43.2	0.9610	8.3	0.01279	0.0130	83.03
15	48	21	49	42.2	0.9610	8.4	0.01279	0.0096	81.11
30	47	21	48	41.2	0.9610	8.6	0.01279	0.0068	79.18
60	45	21	46	39.2	0.9610	8.9	0.01279	0.0049	75.34
120	44	22	45	38.4	0.9610	9.1	0.01264	0.0035	73.80
240	42	22	43	36.4	0.9610	9.4	0.01264	0.0025	69.96
480	41	23	42	35.7	0.9610	9.6	0.01249	0.0018	68.61
1440	39	22	40	33.4	0.9610	9.9	0.01264	0.0010	64.19

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine

Sample depth (m): 2.3-2.5

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.83

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.28 Data Sheet for Hydrometer Analysis of CTP2

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	49	21	50	43.2	0.9628	8.3	0.01282	0.0522	83.19
1	48.5	21	50	42.7	0.9628	8.35	0.01282	0.0370	82.22

2	48	21	49	42.2	0.9628	8.4	0.01282	0.0263	81.26
4	47.5	21	49	41.7	0.9628	8.5	0.01282	0.0187	80.30
8	46	21	47	40.2	0.9628	8.8	0.01282	0.0134	77.41
15	44	21	45	38.2	0.9628	9.1	0.01282	0.0100	73.56
30	42	21	43	36.2	0.9628	9.4	0.01282	0.0072	69.71
60	41	21	42	35.2	0.9628	9.6	0.01282	0.0051	67.78
120	39	22	40	33.4	0.9628	9.9	0.01267	0.0036	64.32
240	37	22	38	31.4	0.9628	10.2	0.01267	0.0026	60.47
480	35	22	36	29.4	0.9628	10.6	0.01267	0.0019	56.61
1440	32	22	33	26.4	0.9628	11.1	0.01267	0.0011	50.84

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine

Sample depth (m): 2.3-2.5

Sample: oven dried disturbed sample

Sample weight: 50 g

Hydrometer number: 152 H

Specific gravity: 2.85

Zero correction: +6

Meniscus correction: +1

Dispersing agent: Sodium Hexametaphosphate

Table A.29 Data Sheet for Hydrometer Analysis of CTP3

Time (minutes)	HR	T	CHR		Cf (a)	EDR (L)	K	DSP (mm)	DSP (mm)
			R'	R"					
0.5	53	21	54	47.2	0.9591	7.6	0.01275	0.0497	90.54
1	52	21	53	46.2	0.9591	7.8	0.01275	0.0356	88.62
2	51	21	52	45.2	0.9591	7.9	0.01275	0.0253	86.71
4	50	21	51	44.2	0.9591	8.1	0.01275	0.0181	84.79
8	49	21	50	43.2	0.9591	8.3	0.01275	0.0130	82.87
15	48.5	21	50	42.7	0.9591	8.35	0.01275	0.0095	81.91
30	48	21	49	42.2	0.9591	8.4	0.01275	0.0067	80.95
60	46	21	47	40.2	0.9591	8.8	0.01275	0.0049	77.11

120	44	22	45	38.4	0.9591	9.1	0.01260	0.0035	73.66
240	43	23	44	37.7	0.9591	9.2	0.01245	0.0024	72.32
480	40.5	22	42	34.9	0.9591	9.65	0.01260	0.0018	66.95
1440	38	22	39	32.4	0.9591	10.1	0.01260	0.0011	62.15

HR = Hydrometer reading, T = Temperature, CHR = Corrected hydrometer reading, Cf = Corrected factor, EDR = Effective depth reading, DSP = Diameter of soil particle and % P = percentage of fine.

A.7 Free Swell Test

Table A.30 Data Sheet for Free Swell Test

Designation	Depth, m	Kerosene reading, ml	Water Reading, ml	Free Swell, %
TTP1	1.2	10.00	14.80	48.00
TTP2	1.2	10.00	15.50	55.00
TTP3	1.2	10.00	16.00	60.00
CTP1	3	10.00	15.00	50.00
CTP2	2.5	10.00	15.00	50.00
CTP3	2.5	10.00	14.70	47.00

A.8 Unconfined Compression Strength Test Analysis

Visual description: red wet soft moderate homogenous clay (ASTM D 2488)

Soil classification: CH

Sample: undisturbed

Sample depth: 1 m

Project name: Adale, Lalisa Village

Sample number: TTP1

Sample diameter, mm = 36

Average height of sample, mm = 79.25

Height to diameter ratio: 2.2

Table A.31 Unconfined Compression Test of TTP1 (Deformation dial 1 unit = 0.01 mm, Load Dial 1 unit = 0.3154 lb, 1 lb= 4.448 N)

DDR	LDR	ΔL (mm)	ε	ε (%)	Ao (mm^2)	A'	Load (lb)	Load (N)	σ (KN/m^2)
0	0	0	0	0	1017.9	1017.88	0	0	0

20	1.75	0.2	0.003	0.25	1017.9	1020.43	0.552	2.455	2.405
40	3	0.4	0.005	0.5	1017.9	1022.99	0.946	4.208	4.113
60	3.95	0.6	0.008	0.76	1017.9	1025.66	1.246	5.542	5.401
80	5	0.8	0.01	1.01	1017.9	1028.27	1.577	7.014	6.822
100	6	1	0.013	1.26	1017.9	1030.87	1.892	8.416	8.161
120	7.25	1.2	0.015	1.51	1017.9	1033.49	2.287	10.173	9.844
140	8.25	1.4	0.018	1.77	1017.9	1036.22	2.602	11.574	11.166
160	9.1	1.6	0.02	2.02	1017.9	1038.87	2.87	12.766	12.291
180	10.1	1.8	0.023	2.27	1017.9	1041.52	3.186	14.171	13.602
200	11.25	2	0.025	2.52	1017.9	1044.19	3.548	15.782	15.117
220	11.8	2.2	0.028	2.78	1017.9	1046.99	3.722	16.555	15.809
240	12.25	2.4	0.03	3.03	1017.9	1049.68	3.864	17.187	16.379
260	12.75	2.6	0.033	3.28	1017.9	1052.40	4.021	17.885	16.991
280	13.2	2.8	0.035	3.53	1017.9	1055.13	4.163	18.517	17.555
300	13.45	3	0.038	3.79	1017.9	1057.98	4.242	18.868	17.832
320	13.85	3.2	0.04	4.04	1017.9	1060.74	4.368	19.429	18.324
340	14	3.4	0.043	4.29	1017.9	1063.50	4.416	19.642	18.467
360	14.2	3.6	0.045	4.54	1017.9	1066.29	4.479	19.923	18.692
380	14.3	3.8	0.048	4.79	1017.9	1069.09	4.51	20.06	18.762
400	14.45	4	0.05	5.05	1017.9	1072.02	4.558	20.274	18.922
420	14.45	4.2	0.053	5.3	1017.9	1074.85	4.558	20.274	18.862
440	14.35	4.4	0.056	5.55	1017.9	1077.69	4.526	20.132	18.671
460	14.25	4.6	0.058	5.8	1017.9	1080.55	4.494	19.989	18.499
480	14.2	4.8	0.061	6.06	1017.9	1083.54	4.479	19.923	18.379
500	14.15	5	0.063	6.31	1017.9	1086.43	4.463	19.851	18.274
520	14.1	5.2	0.066	6.56	1017.9	1089.34	4.447	19.78	18.15

DDR = Deformation dial reading, LDR = Load dial reading, ΔL = sample deformation, ε = strain, Ao = Initial area, A' = corrected area, σ = stress

Visual description: red wet soft moderate homogenous clay (ASTM D 2488)

Soil classification: CH Sample: undisturbed

Sample depth: 1-1.2 m

Project name: Adale, Lalisa village

Sample number: TTP2

Sample diameter, mm = 36

Average height of sample, mm = 80

Height to diameter ratio: 2.22

Table A.32 Unconfined Compression Test of TTP2 (Deformation dial 1 unit = 0.01 mm, Load Dial 1 unit = 0.3154 lb, 1 lb= 4.448 N)

DDR	LDR	ΔL (mm)	ε	ε (%)	Ao (mm^2)	A'	Load (lb)	Load (N)	σ (KN/m^2)
0	0	0	0	0	1017.9	1017.88	0	0	0
20	1.75	0.2	0.003	0.3	1017.9	1020.943	1.217	5.413	5.302
40	2.85	0.4	0.005	0.5	1017.9	1022.995	1.982	8.816	8.618
60	3.55	0.6	0.008	0.8	1017.9	1026.089	2.468	10.978	10.699
80	4.45	0.8	0.01	1	1017.9	1028.162	3.094	13.762	13.385
100	5.4	1	0.013	1.3	1017.9	1031.287	3.755	16.702	16.195
120	6	1.2	0.015	1.5	1017.9	1033.381	4.172	18.557	17.958
140	6.55	1.4	0.018	1.8	1017.9	1036.538	4.554	20.256	19.542
160	7	1.6	0.02	2	1017.9	1038.653	4.867	21.648	20.842
180	7.45	1.8	0.023	2.3	1017.9	1041.842	5.18	23.041	22.116
200	7.85	2	0.025	2.5	1017.9	1043.979	5.458	24.277	23.254
220	8.1	2.2	0.028	2.8	1017.9	1047.202	5.632	25.051	23.922
240	8.2	2.4	0.03	3	1017.9	1049.361	5.702	25.362	24.169
260	8.15	2.6	0.033	3.3	1017.9	1052.616	5.667	25.207	23.947
280	8.25	2.8	0.035	3.5	1017.9	1054.798	5.736	25.514	24.189
300	8.35	3	0.038	3.8	1017.9	1058.087	5.806	25.825	24.407
320	8	3.2	0.04	4	1017.9	1060.292	5.563	24.744	23.337
340	7.8	3.4	0.043	4.3	1017.9	1063.615	5.424	24.126	22.683
360	7.3	3.6	0.045	4.5	1017.9	1065.843	5.076	22.578	21.183
380	6.55	3.8	0.048	4.8	1017.9	1069.202	4.554	20.256	18.945
400	5.8	4	0.05	5	1017.9	1071.453	4.033	17.939	16.743
420	5.45	4.2	0.053	5.3	1017.9	1074.847	3.79	16.858	15.684

440	4.75	4.4	0.055	5.5	1017.9	1077.122	3.303	14.692	13.64
460	4	4.6	0.058	5.8	1017.9	1080.552	2.781	12.37	11.448

Visual description: red moist soft moderate homogenous clay (ASTM D 2488)

Soil classification: CH

Sample number: CTP1

Sample: undisturbed

Sample diameter, mm = 36

Sample depth: 2.8 - 3 m

Average height of sample, mm = 79.75

Project name: Adale, Lalisa Village

Height to diameter ratio: 2.2

Table A.33 Unconfined Compression Test of CTP1 (Deformation dial 1 unit = 0.01 mm, Load Dial 1 unit = 0.3154 lb, 1 lb= 4.448 N)

DDR	LDR	ΔL (mm)	ε	ε (%)	Ao (mm^2)	A'	Load (lb)	Load (N)	σ (KN/m^2)
0	0	0	0	0	1017.9	1017.88	0	0	0
20	1	0.2	0.003	0.25	1017.9	1020.431	0.315	1.401	1.373
40	1.75	0.4	0.005	0.5	1017.9	1022.995	0.552	2.455	2.4
60	2.6	0.6	0.008	0.76	1017.9	1025.675	0.82	3.647	3.556
80	3.4	0.8	0.010	1.01	1017.9	1028.265	1.072	4.768	4.637
100	3.95	1	0.013	1.26	1017.9	1030.869	1.246	5.542	5.376
120	4.5	1.2	0.015	1.51	1017.9	1033.486	1.419	6.312	6.107
140	5.4	1.4	0.018	1.77	1017.9	1036.221	1.703	7.575	7.31
160	6.2	1.6	0.020	2.02	1017.9	1038.865	1.955	8.696	8.371
180	6.9	1.8	0.022	2.27	1017.9	1041.523	2.176	9.679	9.293
200	7.45	2	0.025	2.52	1017.9	1044.194	2.35	10.453	10.011
220	8.2	2.2	0.028	2.78	1017.9	1046.986	2.586	11.503	10.987
240	9.05	2.4	0.030	3.03	1017.9	1049.685	2.854	12.695	12.094
260	10	2.6	0.033	3.28	1017.9	1052.399	3.154	14.029	13.33
280	10.75	2.8	0.035	3.53	1017.9	1055.126	3.391	15.083	14.295
300	11.5	3	0.038	3.79	1017.9	1057.977	3.627	16.133	15.249
320	12.4	3.2	0.040	4.04	1017.9	1060.734	3.911	17.396	16.4

340	13.15	3.4	0.043	4.29	1017.9	1063.504	4.148	18.45	17.348
360	13.95	3.6	0.045	4.54	1017.9	1066.29	4.4	19.571	18.354
380	14.7	3.8	0.048	4.79	1017.9	1069.089	4.636	20.621	19.288
400	15.35	4	0.051	5.05	1017.9	1072.017	4.841	21.533	20.086
420	16	4.2	0.053	5.3	1017.9	1074.847	5.046	22.445	20.882
440	16.4	4.4	0.056	5.55	1017.9	1077.692	5.173	23.01	21.351
460	16.75	4.6	0.058	5.8	1017.9	1080.552	5.283	23.499	21.747
480	17.15	4.8	0.061	6.06	1017.9	1083.543	5.409	24.059	22.204
500	17.35	5	0.063	6.31	1017.9	1086.434	5.472	24.339	22.403
520	17.55	5.2	0.066	6.56	1017.9	1089.341	5.535	24.62	22.601
540	17.4	5.4	0.068	6.81	1017.9	1092.263	5.488	24.411	22.349
560	17.15	5.6	0.071	7.07	1017.9	1095.319	5.409	24.059	21.965
580	16.5	5.8	0.073	7.32	1017.9	1098.274	5.204	23.147	21.076
600	15.95	6	0.076	7.57	1017.9	1101.244	5.031	22.378	20.321
620	15.5	6.2	0.078	7.82	1017.9	1104.231	4.889	21.746	19.693
640	14.95	6.4	0.081	8.08	1017.9	1107.354	4.715	20.972	18.939
660	14.4	6.6	0.083	8.33	1017.9	1110.374	4.542	20.203	18.195
680	13.7	6.8	0.086	8.58	1017.9	1113.411	4.321	19.22	17.262
700	13.2	7	0.088	8.83	1017.9	1116.464	4.163	18.517	16.585

720	12.85	7.2	0.091	9.09	1017.9	1119.657	4.053	18.028	16.101
740	12.45	7.4	0.093	9.34	1017.9	1122.744	3.927	17.467	15.557
760	12	7.6	0.096	9.59	1017.9	1125.849	3.785	16.836	14.954

Visual description: red moist soft moderate homogenous clay (ASTM D 2488)

Soil classification: CH

Sample number: CTP2

Sample: undisturbed

Sample diameter, mm = 36

Sample depth: 2.3 – 2.5 m

Average height of sample, mm = 79.25

Project name: Adale, Lalisa Village

Height to diameter ratio: 2.20

Table A.34 Unconfined Compression Test of CTP2 (Deformation dial 1 unit = 0.01 mm, Load Dial 1 unit = 0.3154 lb, 1 lb= 4.448 N)

DDR	LDR	ΔL (mm)	ε	ε (%)	Ao (mm^2)	A'	Load (lb)	Load (N)	σ (KN/m^2)
0	0	0	0	0	1017.9	1017.88	0	0	0
20	1.25	0.2	0.003	0.3	1017.9	1020.943	0.394	1.753	1.717
40	3	0.4	0.005	0.5	1017.9	1022.995	0.946	4.208	4.113
60	4.5	0.6	0.008	0.8	1017.9	1026.089	1.419	6.312	6.152
80	6	0.8	0.01	1	1017.9	1028.162	1.892	8.416	8.185
100	7.5	1	0.013	1.3	1017.9	1031.287	2.366	10.524	10.205
120	8.75	1.2	0.015	1.5	1017.9	1033.381	2.76	12.276	11.879
140	10	1.4	0.018	1.8	1017.9	1036.538	3.154	14.029	13.534
160	11.35	1.6	0.02	2	1017.9	1038.653	3.58	15.924	15.331
180	12.75	1.8	0.023	2.3	1017.9	1041.842	4.021	17.885	17.167
200	14.5	2	0.025	2.5	1017.9	1043.979	4.573	20.341	19.484
220	16.1	2.2	0.028	2.8	1017.9	1047.202	5.078	22.587	21.569
240	18	2.4	0.03	3	1017.9	1049.361	5.677	25.251	24.063
260	19.25	2.6	0.033	3.3	1017.9	1052.616	6.071	27.004	25.654
280	20.5	2.8	0.035	3.5	1017.9	1054.798	6.466	28.761	27.267

300	21.85	3	0.038	3.8	1017.9	1058.087	6.891	30.651	28.968
320	23	3.2	0.04	4	1017.9	1060.292	7.254	32.266	30.431
340	24.15	3.4	0.043	4.3	1017.9	1063.615	7.617	33.88	31.854
360	25	3.6	0.045	4.5	1017.9	1065.843	7.885	35.072	32.905
380	26	3.8	0.048	4.8	1017.9	1069.202	8.2	36.474	34.113
400	27	4	0.05	5	1017.9	1071.453	8.516	37.879	35.353
420	27.75	4.2	0.053	5.3	1017.9	1074.847	8.752	38.929	36.218
440	28.75	4.4	0.055	5.5	1017.9	1077.122	9.068	40.334	37.446
460	29.45	4.6	0.058	5.8	1017.9	1080.552	9.289	41.317	38.237
480	30.1	4.8	0.06	6	1017.9	1082.851	9.494	42.229	38.998
500	30.6	5	0.063	6.3	1017.9	1086.318	9.651	42.928	39.517
520	30.9	5.2	0.065	6.5	1017.9	1088.642	9.746	43.35	39.82
540	31.4	5.4	0.068	6.8	1017.9	1092.146	9.904	44.053	40.336
560	31.6	5.6	0.07	7	1017.9	1094.495	9.967	44.333	40.505
580	31.65	5.8	0.073	7.3	1017.9	1098.037	9.982	44.4	40.436
600	31.5	6	0.075	7.5	1017.9	1100.411	9.935	44.191	40.159
620	31	6.2	0.078	7.8	1017.9	1103.991	9.777	43.488	39.392
640	29.5	6.4	0.08	8	1017.9	1106.391	9.304	41.384	37.404
660	28.25	6.6	0.083	8.3	1017.9	1110.011	8.91	39.632	35.704
680	26.75	6.8	0.085	8.5	1017.9	1112.437	8.437	37.528	33.735

Visual description: red moist soft moderate homogenous clay (ASTM D 2488)

Soil classification: CH

Sample: undisturbed

Sample depth: 1-1.2 m

Project name: Adale, Lalisa Village

Sample number: CTP3

Sample diameter, mm = 36

Average height of sample, mm = 77

Height to diameter ratio: 2.14

Table A.35 Unconfined Compression Test of CTP3 (Deformation dial 1 unit = 0.01 mm, Load Dial 1 unit = 0.3154 lb, 1 lb= 4.448 N)

DDR	LDR	ΔL (mm)	ε	ε (%)	Ao (mm^2)	A'	Load (lb)	Load (N)	σ (KN/m^2)

0	0	0	0	0	1017.9	1017.88	0	0	0
20	1.1	0.2	0.003	0.3	1017.9	1020.94	0.765	3.403	3.333
40	2.05	0.4	0.005	0.5	1017.9	1023	1.425	6.338	6.196
60	2.6	0.6	0.008	0.8	1017.9	1026.09	1.808	8.042	7.838
80	4.15	0.8	0.01	1	1017.9	1028.16	2.886	12.837	12.485
100	5.55	1	0.013	1.3	1017.9	1031.29	3.859	17.165	16.644
120	6.95	1.2	0.016	1.6	1017.9	1034.43	4.833	21.497	20.781
140	8.15	1.4	0.018	1.8	1017.9	1036.54	5.667	25.207	24.318
160	9.65	1.6	0.021	2.1	1017.9	1039.71	6.71	29.846	28.706
180	11	1.8	0.023	2.3	1017.9	1041.84	7.649	34.023	32.657
200	12.25	2	0.026	2.6	1017.9	1045.05	8.518	37.888	36.255
220	13.4	2.2	0.029	2.9	1017.9	1048.28	9.317	41.442	39.533
240	14.7	2.4	0.031	3.1	1017.9	1050.44	10.221	45.463	43.28
260	16.05	2.6	0.034	3.4	1017.9	1053.71	11.16	49.64	47.11
280	17.15	2.8	0.036	3.6	1017.9	1055.89	11.925	53.042	50.234
300	18.25	3	0.039	3.9	1017.9	1059.19	12.69	56.445	53.291
320	19.35	3.2	0.042	4.2	1017.9	1062.51	13.455	59.848	56.327
340	20.15	3.4	0.044	4.4	1017.9	1064.73	14.011	62.321	58.532
360	21.25	3.6	0.047	4.7	1017.9	1068.08	14.776	65.724	61.535
380	22.25	3.8	0.049	4.9	1017.9	1070.33	15.471	68.815	64.293
400	23.2	4	0.052	5.2	1017.9	1073.71	16.132	71.755	66.829
420	24.05	4.2	0.055	5.5	1017.9	1077.12	16.723	74.384	69.058
440	25	4.4	0.057	5.7	1017.9	1079.41	17.383	77.32	71.632
460	25.55	4.6	0.06	6	1017.9	1082.85	17.766	79.023	72.977
480	26.15	4.8	0.062	6.2	1017.9	1085.16	18.183	80.878	74.531
500	26.75	5	0.065	6.5	1017.9	1088.64	18.6	82.733	75.997
520	27.1	5.2	0.068	6.8	1017.9	1092.15	18.843	83.814	76.742
540	27.4	5.4	0.07	7	1017.9	1094.5	19.052	84.743	77.427
560	27.5	5.6	0.073	7.3	1017.9	1098.04	19.122	85.055	77.461
580	27.4	5.8	0.075	7.5	1017.9	1100.41	19.052	84.743	77.01

600	26.75	6	0.078	7.8	1017.9	1103.99	18.6	82.733	74.94
620	26.3	6.2	0.081	8.1	1017.9	1107.6	18.287	81.341	73.439
640	25.5	6.4	0.083	8.3	1017.9	1110.01	17.731	78.867	71.051
660	25	6.6	0.086	8.6	1017.9	1113.65	17.383	77.32	69.429
680	24.5	6.8	0.088	8.8	1017.9	1116.1	17.036	75.776	67.894
700	23.65	7	0.091	9.1	1017.9	1119.78	16.445	73.147	65.323
720	23.15	7.2	0.094	9.4	1017.9	1123.49	16.097	71.599	63.729
740	22.1	7.4	0.096	9.6	1017.9	1125.97	15.367	68.352	60.705
760	21.35	7.6	0.099	9.9	1017.9	1129.72	14.845	66.031	58.449
780	20.6	7.8	0.101	10.1	1017.9	1132.24	14.324	63.713	56.272
800	19.75	8	0.104	10.4	1017.9	1136.03	13.733	61.084	53.77
820	18.65	8.2	0.106	10.6	1017.9	1138.57	12.968	57.682	50.662
840	17.5	8.4	0.109	10.9	1017.9	1142.4	12.168	54.123	47.376
860	16.5	8.	0.112	11.2	1017.9	1146.26	11.473	51.032	44.52

Appendix B Different Constant Values Depend on Different Factors

Table B.1 Values of Correction Factor, a for Different Specific Gravities of Soil Particles[a]

Specific Gravity	Correction Factor[A]
2.95	0.94
2.90	0.95
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.03
2.45	1.05

Table B.2 Values of k for use in Equation for Computing Diameter of Particle in Hydrometer Analysis

Temperature,° C	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01510	0.01505	0.01481	0.01457	0.01435	0.01414	0.01394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.1323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01255	0.01208	0.01191	0.01175
29	0.01312	0.01290	0.01269	0.01249	0.01230	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

Table B.3 Correction Value Based on Test Temperature

Temperature	21	22	23	25	25.5
Corrections	0.2	0.4	0.7	1.3	1.48

Table B.4 Different Viscosity Value Based on Temperature

Temperature	19	20	21	22	23	24	25	26
Viscosity, η	0.0103	0.01009	0.00984	0.00961	0.00938	0.00916	0.00895	0.00875

Table B.5 Values of Effective Depth Based on Hydrometer and Sedimentation Cylinder of Specified Sizes

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm
1.000	16.3	0	16.3	31	11.2
1.001	16.0	1	16.1	32	11.1
1.002	15.8	2	16.0	33	10.9
1.003	15.5	3	15.8	34	10.7
1.004	15.2	4	15.6	35	10.6
1.005	15.0	5	15.5		
1.006	14.7	6	15.3	36	10.4
1.007	14.4	7	15.2	37	10.2
1.008	14.2	8	15.0	38	10.1
1.009	13.9	9	14.8	39	9.9
1.010	13.7	10	14.7	40	9.7
1.011	13.4	11	14.5	41	9.6
1.012	13.1	12	14.3	42	9.4
1.013	12.9	13	14.2	43	9.2
1.014	12.6	14	14.0	44	9.1
1.015	12.3	15	13.8	45	8.9
1.016	12.1	16	13.7	46	8.8
1.017	11.8	17	13.5	47	8.6
1.018	11.5	18	13.3	48	8.4
1.019	11.3	19	13.2	49	8.3
1.020	11.0	20	13.0	50	8.1
1.021	10.7	21	12.9	51	7.9
1.022	10.5	22	12.7	52	7.8
1.023	10.2	23	12.5	53	7.6
1.024	10.0	24	12.4	54	7.4
1.025	9.7	25	12.2	55	7.3
1.026	9.4	26	12.0	56	7.1
1.027	9.2	27	11.9	57	7.0
1.028	8.9	28	11.7	58	6.8
1.029	8.6	29	11.5	59	6.6
1.030	8.4	30	11.4	60	6.5
1.031	8.1				
1.032	7.8				
1.033	7.6				
1.034	7.3				
1.035	7.0				
1.036	6.8				
1.037	6.5				
1.038	6.2				

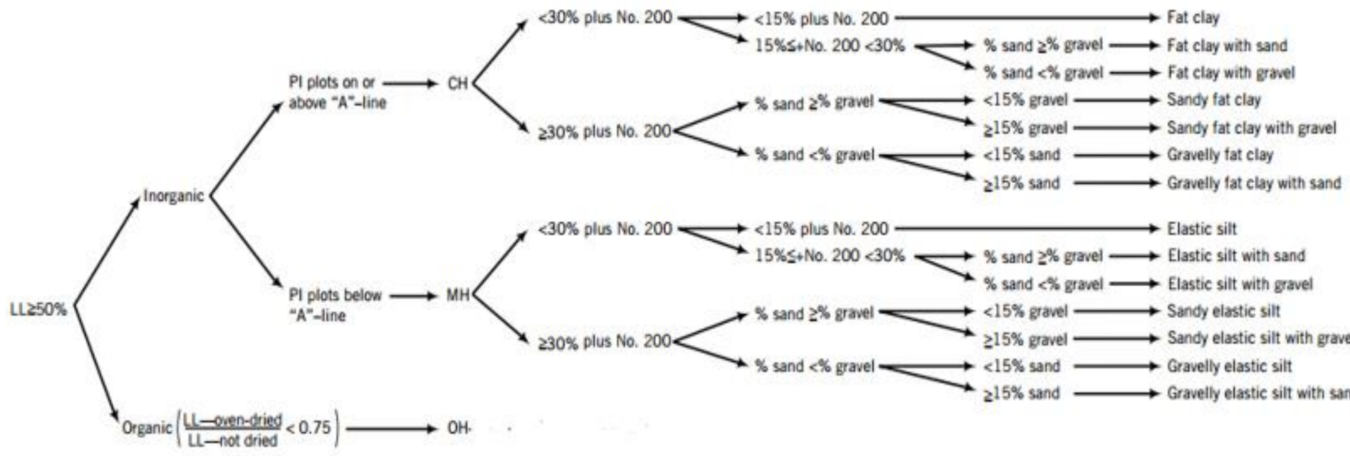

Figure B.1 Flowchart for Classifying Inorganic Fine-Grained Soils (50% or more fines). (Source: ASTM D 2487-10 standard practice for classification of soils for engineering purposes)