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JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL ENGINEERING CHAIR

MASTERS OF SCIENCE PROGRAM IN GEOTECHNICAL ENGINEERING

GEOTECHNICAL CHARACTERIZATIION AND SLOPE STABILITY ANALYSIS OF LANDSLIDE; THE CASE OF WERIE-MAYKINATAL ROAD, TIGRAY REGION, NORTHERN ETHIOPIA

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: Terhas Abrha

January, 2020

Jimma, Ethiopia

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By: Terhas Abrha

Main-advisor: Dr. Kifle Woldearegay

Co-advisor: Mohammed Yasin

January, 2020

Jimma, Ethiopia

DECLARATION

I hereby declare that the research entitled "**Geotechnical Characterization and Slope Stability Analysis of Landslide: the Case of Study Werie-Maykinatal Road, Tigray Region Northern Ethiopia**" is my work which I submit for partial fulfillment of the requirements for the degree of Master of Science in Geotechnical Engineering to Jimma University, school of graduate studies, Jimma Institute of Technology Geotechnical Engineering Chair. The research was conducted under the guidance of main advisor Dr. Kifle Woldearegay and co-advisor Mr. Mohammed Yasin (MSc.).

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(Co-Advisor)	Signature	Date

APPROVAL

The thesis entitled "**Geotechnical Characterization and Slope Stability Analysis of Landslide: the Case of Study Werie-Maykinatal Road, Tigray Region Northern Ethiopia**" submitted by Terhas Abrha is approved and accepted as a Partial Fulfillment of the Requirements for the Degree of Masters of Science in Geotechnical Engineering at Jimma Institute of Technology.

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Main Advisor:	Dr. Kifle Woldearegay	___________	________
Co-Advisor:	Mohammed Yasin (MSc.)	___________	_________

As members of the examining board of MSc. thesis, we certify that we have read and evaluated the thesis prepared by Terhas Abrha. We recommend that the thesis could be accepted as a Partial Fulfillment of the Requirements for the Degree of Masters of Science in Geotechnical Engineering.

	Name	Signature	Date
Chairman:	_____________	____________	____________
External Examiner:	____________	____________	___________-
Internal Examiner:	____________	___________	____________

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ABSTRACT

Recently, landslide has occurred in Werie – Maykinatal road section of the Mekelle - Abi Adi - Adwa road project in Tigray Regional State, northern Ethiopia. This study aimed at investigating the geotechnical characteristics of type soil/rock, evaluating the stability condition of the slopes, and provision of recommendations on remedial measures in order to address the landslide problems. The approached/ methods used include: field work, laboratory tests and slope stability analysis. The field investigation involved description of soils/rock, inventory and detailed characterizations of the landslide affected sites (slope angle, dimensions of failed slopes, and any signs of instabilities) as well as sampling of soils for laboratory analysis. For the present study, based on the field manifestation of instability two most critical slope sections were identified for detailed slope stability analysis. ASTM (American Society Test Material standard) method was used to determine the properties of the soils in the laboratory. Tests carried out include: grain size, Atterberg limit, natural moisture content, shear strength and specific gravity. A total of six samples representing from two most critical slope sections in Werie – Maykinatal road were studied, with a view of determining their sliding potential. The results of geotechnical analysis revealed that the soils contain 1.62-1.97 % clay and silt, 43.56-65.37 % sand and 31.28-54.16 % gravel. Based on the unified soil classification system, the soil samples were classified as well-graded gravel and Poorly-graded gravel with clay and sand with group symbols of GW and GP-GC respectively. Results of the direct shear tests revealed that the cohesion and angle of internal friction varies between 25.3-73.65 kPa and 17.16-27.93 degrees respectively. The stability of the slopes were analyzed using Geo studio Slope/W software and result show that the factor of safety (FOS) values from site1 and site2 are 1.203 and1.372 respectively. This show the slopes are marginally stable and are being affected by slope instability. A modified slope angle (gentle slope) is recommended with a FOS value of 1.683 and 1.793 forsite1 and site2 respectively. The factor of safety (FOS) for modified slope angle was higher when compared to the FOS values from natural slope due to the effect of steep slope on the natural slope. In order to mitigate the problems provision of gabion retaining walls integrated with drainages are recommended. Moreover, surface water drainages (road surface and side ditches) are suggested. Benching of the upslope can also minimize the slope instability of the road sections.

Key Words: Geotechnical Characteristics, Landslide, Mitigation Measures, Slope Stability Analysis

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SYMBOLS, ABBREVIATIONS AND ACRONYMS

a.s.l	above sea level
AASHTO	American Association of State
ASTM	American society of test material
C	Cohesion
CSS	Critical Slip Surfaces
E	Easting
ERA	Ethiopian road authority
FOS	Factor of Safety
GPS	Geographical Positioning System
GS	Specific gravity
LEM	Limit Equilibrium Method
LL	Liquid Limit
MS	Microsoft
N	Northing
N	No. of blows for Liquid limit
ϕ	Angle of internal friction
PI	Plasticity Index
PL	Plastic Limit
UNESCO	United Nation, Educational,
Science and Cultural Organization	
USCS	Unified Soil Classifications
standards	
W	Moisture content
γd	Dry unit weigh
GW	well-graded gravel soil
GP-GC	Poorly-graded gravel with clay
and sand	

CHAPTER ONE

INTORDUCTION

1.1 Background

The developments in soil and rock mechanics play an important role in the evolution of slope stability analyses in geotechnical engineering. The increasing demand for the engineered cut and fill slopes in construction projects has enhanced the needs for deepened understanding on the analytical methods, investigation tools and stabilization methods in order to solve slope stability problems. (Abramson et al., 2002)

A slope is defined as a surface of which one end or side is at higher level than another; a rising or falling surface. An earth slope is an unsupported, inclined surface of a soil mass. The failure of a mass of soil located beneath a slope is called as slide. It involves a downward and outward movement of the entire mass of soil that participates in the failure. The failure of slopes takes place mainly due to, the action of gravitational forces, and Seepage forces within the soil. They may also fail due to excavation or undercutting of its foot, or due to gradual disintegration of the structure of the soil. Slides may occur in almost every conceivable manner, slowly or suddenly, and with or without any apparent provocation (Salunkhe and Chvan,2017).

The movement of mass of a soil in a downward direction of a slope is called a slide or a slope failure. The failure of a natural slope is a common geological phenomenon occurring whenever an imbalance takes place between shear strength and shear stress in the ground. The first sign of an imminent landslide is the appearance of surface cracks in the upper part of the slope, perpendicular to the direction of the movement. The instability is either due to increase in seepage pressure, due to excavation of slope toe material, due to increase of shear stress from surface loading as a result of construction or train traffic. The slip may occur through the fill, through the base or through foundation (Abramson et al., 2002).

Landslides have widespread distribution in Ethiopia and result in different hazard level and extents often occur in hilly and mountainous terrains and they were triggered by different influencing factors, such as rugged morphology, physically weak lithologies, very scarce land cover (barren land), poor land use practices and wide distribution of surface and groundwater associated with seasonal floods (Kifle Woldearegay, 2013 and Bekele Abebe et al, 2010). In many parts of Ethiopian highlands landslide-induced hazards are the most destructive natural phenomena that cause property damages, including failures of engineering structures, human sufferings, environmental degradation and loss of fertile agricultural farm lands (Lulseged Ayalew, 1999, kifle Woldearegay, 2013, Tenalem Ayenew and Giulio Barbieri, 2005, Lulseged Ayalew and Hiromitsu Yamagishi, 2004 and Lulseged Ayalew 2000).

The present study area is located in northern part of Ethiopia, along the way between WerieLehe and Maykinetal, within the Werie - Maykinetal road section of The Mekele - Abi Adi - Adwa Road project.

There are numerous methods currently available for performing the slope stability analysis. The majority of these methods may be categorized as limit equilibrium method. The limit equilibrium method is widely used due to its simplicity. There are numerous limit equilibrium methods available for evaluation of slope stability, such as Ordinary Method, Bishop Simplified Method, Janbu Simplified Method, Janbu Corrected Method, Spencer’s Method, Corp’s of Engineers Method, Morgenstern and Price’s Method, Lowe-Karafiath Method and Generalized Limit Equilibrium Method (GLE). The most widely used limit equilibrium method of analysis for slope stability is the Bishop's Simplified Method (Chitra and Gupta, 2016).

This research attempted to: investigate: the geotechnical condition of the site, analyze stability of the landslide affected area around WerieLehe town of EdagaArbiworeda, and recommend possible mitigation measure that may help to alleviate the observed problem.

1.2 Statement of the Problem

Slope stability analysis is a vital tool for the design and construction of slopes. This analysis is performed to assess the safe design of slope and the equilibrium conditions. Improper slope analysis and design might cause slope failure which has been acknowledge as one of the most frequent disaster that can lead to great loss of properties and life. Thus, the initial soil investigation has to be done properly in order to achieve the actual soil condition for the certain place where we want to start construction.

In hilly areas landslide are main concern where movements of existing or planned slopes could have an effect on the safety of people and property .One of the causes of the incorrect assessment of slope stability may be inaccurate determination of the geological structure of the slope in question (Das, 2011).

Landslide-induced hazards are the most destructive natural phenomena that cause property damages, including failures of engineering structures, human sufferings, environmental degradation and loss of fertile agricultural farm lands (Woldearegay, 2013; Ayalew and Yamagishi, 2004; Ayalew, ,2000).

The Werie – Maykinatal road section of the Mekelle-Abi Adi-Adwa road project connects Mekelle and Adwa town. It passes through steep slopes and highly dissected topography, adverse geological formations, complex structural features and dense to sparse vegetation cover. The route is highly affected by surface water erosions as well as slope instabilities. It is generally characterized by poor and at times with no construction of retaining structures and inadequate surface drainage systems like ditches and culverts. It is common to observe debris/earth slides, scouring of road sections, rock fall, and rockslides. Due to landslide problem in this area, damage of the road (asphalt), hamper traffic, rarely car accident, repeated failures leading to repeated, blocked at three different places and as a result traffic has been hampered for the several days, ditches eroded and culverts are blocked this leads to further erosions and initiate slope failures.

The above problems have been causing both direct and indirect impacts. The direct impacts are cost of reconstruction for the destroyed infrastructure and for resettlement of the displaced communities while the indirect impacts are disruption of economic activities and other social services. This research focuses on the slope stability evaluation of the Werie – Maykinatal road section of the Mekelle - Abi Adi - Adwa road project, Ethiopia.

1.3 Research Questions

The study attempted to answer the following research questions:

1. What are the existing geotechnical properties of the soil/rock masses in the landslide affected sites?
2. What are the triggering factors and effects of landslides in Weria- Maykinatal road section?
3. What is the stability condition of the selected slopes along Weria - Maykinatal road section?
4. What type of remedial measurements would be recommended of the existing landslide in Weria - Maykinatal road section?

1.4 Objectives

1.4.1 General Objective

The general objective of this research was to characterize the geotechnical condition of the selected sites and analyze the slope stability of the landslide along The Werie – Maykinatal road (Mekelle-Abi Adi-Adwa road project, 50m length), Ethiopia.

1.4.2 Specific Objectives

The specific objectives of the study are:

- To determine and characterize the geotechnical properties of soils /rocks in the landslide prone side of the study area.
- To identify the main cause of slope failure along weire – Maykintala road section.
- To determine slope condition and analyze the stability of the slope using Geo studio Software
- To recommend possible remedial measures in order to minimize risks from a landslide in the study area.

1.5 Scope of the Research

Though several sites are affected by landslides, this research dealt with two selected road sections along the Mekelle-Adwa road (mainly along Werie – Maykinatal road section) in Tigray National Regional State, Ethiopia.

In order to address the aforementioned purposes, six test pits were excavated and disturbed as well as undisturbed samples (within a depth of 2.0 to 3 m) were taken at different points of crest, middle and toe of the slope. In addition to field investigations (visual observation and description), laboratory test were carried out which include: atterberg limit, grain size distribution, specific gravity, dry density and natural moisture content and shear strength test. Finally slope stability analysis was carried out and mitigation measures recommended.

1.6 Limitations

Though there were limitations in financial resources, all efforts were done to undertake the research in an ethical and scientific manner Due to the budget and time constraints, it was difficult to extend areas of study, carry out field experiments/instrumentations like inclinometer, tilt meter and piezometers. Despite all these difficulties and limitations, all efforts were being made to present the results and findings in a systematic manner, which were all supported by the actual field observation and laboratory testing. However, despite all these difficulties and limitations, all efforts were being made to present the results and findings in a systematic manner, which were all supported by the actual field observation and laboratory testing.

1.7 Significance of the Study

The road from the Werie – Maykinatal had been constructed over the past few years (2010) and subsequently been plagued with serious slope instability (landslide) problem. Since every natural hazard in the world has different triggering causes, consequences and mitigation measure, so this research has an important point to know what the exact reasons that makes the study area susceptible for soil mass movement or sliding and instability of slopes of the road. 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. So that this study tried to assess the causes and recommended mitigation measures for further upgrading of the road. This study also highlighted the significance of landslides and the need for further research and capacity building.

1.8 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, scope and Significance of the Study 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

2. LITERATURE REVIEW

2.1 Introduction

Landslides, the downward and outward gravitational displacement of slope-forming materials, may damage any structure and may even cause the loss of lives when they occur in a catastrophic way.

Two relevant peculiarities of this hazardous geomorphologic process are its widespread spatial distribution. There are landslide prone slopes almost everywhere, and its high sensitivity to human and natural induced changes in the slopes and controlling factors (Jamil, 2009).

Landslides cause habitat degradation, derange drainage systems, alter drainage path ways, destroy riparian vegetation, bank erosion, accelerate meander development and loss of scenic beauty of mountain environments and additionally landslides threaten people, their property and livelihood source (Gadinala, 2007).

Landslides can also be divided into shallow and deep-seated based on the depth of the slip plane. Slides with a sliding depth of less than 3m-5m are considered to be shallow. Often, these types of landslides involve the soil mantle deep-seated landslides are those slides in which the slide plane is more likely to be within weathered rock. These are usually deeper than 5m (ERA, 2013).

2.2 Concepts of slope stability

Slopes can be man-made or natural. The study and quantification of their safety has been recognized as very essential for the economical prevention of life and property loss. Civil engineers and in particular Geotechnical Engineers have devoted much effort and study to the understanding of the mechanisms leading to failure of slopes. The failure of soil in a down ward and out ward movement of a slope is called a slide or slope failure (Abramson et al., 2002).

Slides occur in almost every conceivable manner, slowly or suddenly and with or without any apparent provocation .They are usually caused by excavation, by undercutting the foot of an existing slope, by a gradual disintegration of the structure of the soil, by an increase of the pore water pressure in a few exceptionally permeable layers, or by a shock that liquefies the soil Problems associated with failures of natural and artificial slopes often pose formidable challenges in geotechnical engineering (Nelson, 2010)**.** In general, an exposed inclined ground surface that is unrestrained may be prone to mass movement due to gravitational forces. The resulting shear stresses, induced along a potential or known failure surface, slope failure occurs when the shear stress along failure plane exceeds the shear strength of the soil. The ratio of available shear strength to induced shear stress in a potential failure surface is referred to as the factor of safety (Nelson, 2010).

Generally, this chapter emphasize on, types of slope, Factors influencing slope stability, types of slope failure, Causes of landslides, slope stability analysis methods and different type of slope stabilization methods used currently.

2.3 Types of slope

Slopes can be categorized as natural or manmade (cut and fill) slope related to geology and civil engineering knowledge. And also divided into three classes: - (1) stable slope is 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 (Msilimba, 2002).

2.4 Causes and Triggering Factors of Landslides

Slope stability is affected by many factors. A change in any one or in the combination of these factors can alter the steady state condition of the slope, decreasing its stability and leading to slope failure. When the slope is in a critical state of stability the destabilization can be generated by a relatively sudden triggering event of natural (such as an earth quake, soil saturation) and human events (undercutting slope for construction purpose). The most important factor controlling slope stability is explained here after (Duncan and Wright, 2005). The following factors are some landslide causes considered in literature review.

2.4.1 Force of gravity

The primary factor influencing shear stress is the pull of gravity. Its influence on slope stability is related to the slope gradient. The forces of gravity can be resolved into two components: a component acting perpendicular to the slope and component acting tangential to the slope. On a steeper slope, the shear stress or tangential component of gravity increases and the perpendicular component of gravity decreases. Therefore, the down-slope movement of a material is affected by steep slope angles which increase the shear stress and reduce shear strength. Shear stress of a material can be promoted by undercutting, mining activity, tectonic tilting and removing of lateral support. Shear strength is governed by inherent factors of rock or regolith such as; angle of internal friction, cohesion and binding action of plant roots between particles **(**Edward and Keller, 2008).

2.4.2 Hydrologic factor

Water plays major roles in both solid rock and soil mass. Water can reduce shear strength and thereby promoting the movement of rocks and sediment down slope under the pull of gravity. Water reduces shear strength by creating positive pressure in the pore spaces of earth materials. Water infiltrating into slope materials can saturates the soil particles at depth by filling the pore spaces.

Slope failures often occur after heavy rainfall over a prolonged time period (Long, 2008). This is a triggering factor which is usually considered for dynamic models of slope failure. Besides rainfall, erosive action of streams also contributes to slope instabilities. Streams erode the lower valley slope by undercutting which leads to increased slope gradient and local slope instability.

2.4.3 Erosion

Two aspects of erosion need to be considered from slope stability point of view. The first is large scale erosion, such as river erosion occurring at the base of a slope. The second is relatively localized erosion caused by groundwater or surface runoff. In the first type, erosion changes the geometry of the potentially unstable rock mass. The removal of material at the toe of a potential slide reduces the confining stress that may be stabilizing the slope.

Localized erosion of joint filling material, or zones of weathered rock, can effectively decrease interlocking between adjacent rock blocks (Oyedepo and Oluwajana, 2013). Loss of such interlocking significantly reduces the rock mass shear strength. The resulting decrease in shear strength may allow a previously stable rock mass to move causing slope failure. In addition, localized erosion may also result in increased permeability and ground-water flow thus affecting the stability of rock slope (Oyedepo and Oluwajana, 2013).

2.4.4 Seismicity

Explosions, earthquakes or volcanic eruptions can increase shear stress and trigger slope failure. These conditions occurred naturally, but can be accelerated by human influence. Intense shaking can cause water pressure in the pore spaces of sediments, leading to liquefaction. The vibration released during earthquakes can cause failure of slopes which were previously stable through the influence of increased vertical acceleration. According to Muthu and Petrou (2007) the possibility of an earthquake triggering a landslide event depends on the shaking of the ground rather than on the actual magnitude of the earthquake.

2.4.5 Land cover change

In developing countries people have cut down trees and removed vegetation to build their houses. The roots of this vegetation bind the soil together and protect it from heavy rainfall keeping the slope stable but, if vegetation is removed the slope is exposed to risk of slope failure. Many slope failures occur on areas that have undergone significant deforestation (Tenalem Ayenew, 2005).

2.4.6 Anthropogenic (human) factors

Anthropogenic factors are related to human activity in response to slope changes. Human activity can shape the slope of landscapes, finally leads to failure. Human activities that can induce landslides are discussed below;

- ❖ Undercutting and slope modification during construction of highways and roads creating an artificial slope that exceeds the angle of repose. This results in increasing the average slope gradients. As a result, increases the chance of slope failures.

- ❖ Overloading of slopes during mining and quarrying operations. This extra weight may increase the chance of slope failure; altering the hydrology may have dramatic effects on slope stability (Long, 2008).
- ❖ Deforestation of trees due to human intervention promotes soil slope failure, when the roots of tree are no longer binding the soil together and unable to protect it from heavy rainfall.
- ❖ Vibrations resulted from artificial causes such as; machine activities and underground explosions cause serious flooding and sediment failure in big reservoirs.

2.5 Geologic factors

Geology is one of the important factors considered in slope stability analysis (Varnes, 1996). Depending on the type of regolith, there is strong relationship between geology of the material and slope instability in specific area. Weathering alters the mechanical, mineralogical and hydrological attributes of the regolith, and, hence, is an important factor of slope instability in many settings. Type of lithology and geological structures plays a great role in slope stability.

2.5.1 Lithology

lithology is among the most important factors commonly considered in slope stability and are used practically in all works dealing with landslide hazard assessment (Clerici et al., 2002; Saha et al., 2002). Lithology is strongly related with slope instability by weathering processes, water percolation and interaction of rock mass. Weathered rock mass allow percolation of water via joints and fractures which can promote slope failure. The properties of rock materials such as; strength and permeability are related to degree of weathering and internal structure of the lithology. Lithological units, such as; basalts, shales, sandstones and limestones have different shear strength characteristics because of the varying conditions under which they are formed.

2.5.2 Geological structures

Geological structures such as; bedding, joints, foliation, cleavage, schistosity, and faults are potentially weak planes in a slope. Their strength is generally less as compared to the strength of surrounding intact rock. Tectonic setting of the area also contributes slope instability by fracturing, faulting, jointing and foliation structures. Fault and fracture zones indicate weak zones; therefore they may be favorable planes of weakness where failures occur. Therefore, it is important to know their orientation in relation to slope angle, direction, and strength along such potential weak planes (Sidle and Ochiai, 2006).

2.6 Geomorphic factors

Geomorphic factors have significant influence on slope instability initiation. These features are directly related with the topography and slope of the area such as; gradient, aspect and shape of the slope.

- ✓ **Slope gradient;** with increasing slope gradient, the shear stress increases due to the effect of gravity thus, down slope movement of material is enhanced. According to Carson and Petley(1970) slope gradient is taken as the main driving forces of mass movement, especially for shallow landslides. In most cases of landslide assessment, slope gradient is taken as main causative factor (Sidle and Ochiai, 2006).
- ✓ **Slope aspect and shape:** Slope aspect refers the direction to which the slope is facing. Aspect is closely related to the bedrock structure particularly in metamorphic rocks. Failures of rocks are common on slope aspect oriented parallel to the direction of foliation and lineation planes (Vieira and Fernandez, 2004).
- ✓ **Shape of the slope** has great influence on the slop stability of steep terrain by concentrating or dispersing surface and primarily subsurface water in the landscape (Sidle and Ochiai, 2006). Three hydro geomorphic slope units important in assessing terrain stability are; (a) divergent, (b) planar and (c) convergent slope shapes have different degree of infiltrating water. In divergent slope, subsurface (and surface) water is dispersed rapidly; thus pore pressure is typically lower than other slope forms. Convergent slope tends to concentrate subsurface water into small area of the slope, thereby high pore water pressure develops which reduces shear strength of the material, thus,

finally promotes down slope failure. Planar slopes are intermediate in susceptibility to landslide between divergent and convergent slopes.

2.7 Geotechnical character of the landslide prone areas

Geotechnical characterization of the study area needs to review of available geologic, topographic, soils maps and ground water condition information (well logs, hydro-geologic maps, documented local project experience) .For basic geotechnical characterization: laboratory analysis consisted of physical, chemical, and mineralogical tests, strength tests and also field instrumentations are utilized such as piezometer, inclinometer, tilt meter and rainfall measurement are considered to characterize the site material geo-technically. The output of all above refer to a synthesis of the program of geotechnical characterization, flow and stability analysis, and proposals for stabilization of the area are in an attempt to significantly reduce the level of risk (Coutinho, 2011).

Before a geotechnical analysis can be performed, the parameter values needed in the analysis must be determined (Rahman, 2012). Some of the parameters which need to be determined are described as follows:

- **Unit weight:** Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight, γ (KN/m3), is usually determined in the laboratory by using undisturbed sample.

- **Shear strength parameters:** If a slope contains more than one soil layer, it may be necessary to calculate the factor of safety for circles at more than one depth (Duncan, 2005).

2.8 Type of soil at prone side

Based on a few laboratory and field tests are the common works to predict the engineering properties of material and as the inputs for geotechnical engineers to classify. The dominant and major categories of soil types are residual (remain at the place of formation), colluvium (transported by gravity), alluvial (transported by water) and aeoline (transported by air). Those are the more susceptible and exposed material types for mass movement. Geologically types of material which is not good for slopes to stay stable are weathered basalt, tuff and scoracious basalt (ERA, 2013). Colluvium is a general term used to describe soil and rock material that has been transported through rain wash, sheet wash and down slope creep that collect on or at the base of slopes. Colluvium is typified by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders. Talus is a gravitationally derived deposit that forms down slope of steep rock slopes, comprised of a generally loose assemblage of course, angular rock fragments of varied size and shape. Talus is commonly collectively referred with the term colluvium. It is a very common deposit of the ground surface in mountainous areas. Colluvial deposits are typically shallow (less than about 25 feet to 30 feet thick), with thickness increasing towards the base of slopes. It commonly directly overlies bedrock on unglaciated slopes and intermixes with alluvial material in stream bottoms (WSDOT, 2013).

2.9 Type of slope failure

According to Hoek E., (2009) failures in rock and soil slopes can be classified into four types; (i) Plane mode of failure, (ii) Wedge mode of failure, (iii) Circular or Rotational mode of failure, and (iv) Toppling mode of failure.

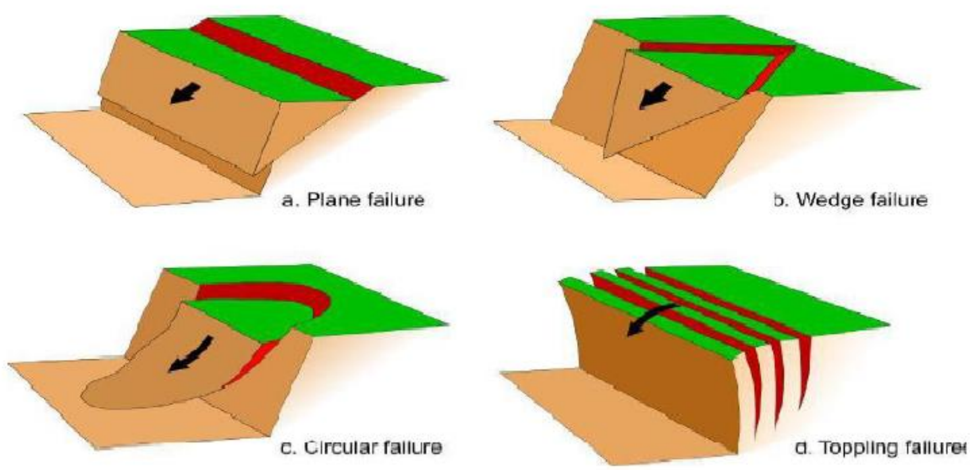

Figure2. 1 The Most Common Slope Failure Modes

2.9.1 Plane failure

In planar failure (Fig. 2.1a) the mass progresses out or down and out along a more or less planar or gently undulating surface. The movement is commonly controlled structurally by (a) surface weakness, such as; faults, joints, bedding planes and variations in shear strength between layers of bedded deposits or (b) the contact between firm bedrock and overlying weathered rock.

Conditions for appearance of planar failure:

- ❖ The strike of the slope doesn´t differs more than ± 20° from the strike of the weakness plane.
- ❖ The toe of the failure plane has to cross the slope between toe and crest.
- ❖ The dip of the failure plane must be less than the dip of the slope face, and the internal angle of friction for the discontinuity must be less than the dip of the discontinuity (Hoek and Bray, 1981).

2.9.2 Wedge failure

Wedge failure (Fig. 2.1b) occur when two discontinuities strikes obliquely across the slope face and their line of intersection daylights in the slope face. The wedge of rock resting on these discontinuities will slide down the line of intersection, provided that the inclination of this line is significantly greater than the angle of friction (Hoek and Bray, 1981). Necessary structural conditions for wedge failure can be summarized as follows (Norrish and Wyllie, 1996):

- ❖ The trend of the line of intersection must approximate the inclination direction of the slope face.
- ❖ The plunge of the line of intersection must be less than the inclination of the slope face and thereby the line of intersection must daylight in the slope.
- ❖ The plunge of the line of intersection must be equal or greater than the angle of friction of the intersecting surfaces (discontinuities).
- ❖ Cohesion (C) equals to zero

2.9.3 Circular (Rotational) Failure

Circular failure surfaces (Fig. 2.1c) are found to be the most critical in slopes of homogeneous materials. This type of failure occurs mainly in soils, but also in weak rock mass, when the rock mass is heavily jointed or fractured. In this case, the failure will be defined by a single discontinuity surface but will tend to follow a circular failure path. This path will follow curved surface of least resistance within the rock mass or soil.

The conditions under which circular failure will occur start when the individual particles in a soil or rock mass are very small as compared with the size of the slope and when these particles are not interlocked as a result of their shape. Hence, crushed rock in a large waste dump will tend to behave as soil and large failures will occur in a circular mode (Hoek, 2009).

The simplest circular analysis is based on the assumption that a rigid, cylindrical block will fail by rotation about its center and that the shear strength along the failure surface is defined by the undrained strength. The factor of safety for such a slope may be analyzed by taking the ratio of the resisting and overturning moments about the center of the circular surface.

A purely circular failure surface on a rotational failure is quite rare because frequently the shape of the failure surface is controlled by the presence of preexisting discontinuities, such as; faults, joints, bedding, shear zones, etc. The influence of such discontinuities must be considered when a slope stability analysis of rotational failure is being conducted. The location of the critical failure surface is found by determining the lowest value of safety factor obtained from a large number of assumed failure surface positions (Kliche, 1999). This study is mainly concentrated on circular mode of failure because all the failures exist along the road are circular types.

2.9.4 Toppling

Toppling failure (Fig. 2.1d) occurs when the weight vector of a block of rock resting on an inclined plane falls outside the base of the block. This type of failure may occur in undercutting beds. A toppling is overturning of a rock block about a pivot point located below its center of gravity .Toppling failure most commonly occurs in rock masses that are sub divided into a series of slabs or columns formed by a set of fractures that strike approximately parallel to the slope face and dip steeply into the rock mass(Hunt, 2006). Toppling failure most commonly occurs in rock masses that are sub divided into a series of slabs or columns formed by a set of fractures that strike approximately parallel to the slope face and dip steeply into the rock mass (Norrish and Wyllie, 1996).

2.10 Slope stability analysis methods

Slope stability is one of the most important engineering practices, particularly encountered in large and important projects such as; dams, highways (roads) and tunnels. Many techniques exist for evaluation of the stability of a given slope. Earlier methods for slope stability analysis were generally based on hand-performed and therefore simplified computations. Now a day, more and more powerful computers becoming commonly available, experts have developed complicated but more accurate methods.

Slope stability analysis deal with determination, investigation, modeling 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. The most common slope stability analysis methods discussed as follow

2.10.1 Limit equilibrium Methods on 2D slope stability analysis

The limit equilibrium method of analysis is a well-established method and widely used by the engineering geologist and engineers. This method mainly provides an assessment of stability of the slope in terms of its safety factor. For determining the factor of safety of a particular slope the primary requirement is the strength properties of the soil material involved and does not consider its stress – strain behavior. The limit equilibrium method provides only an estimate of the stability of a slope but doesn't provide any information about the magnitude of movement of the slope (Duncan and Wright, 2005).

The analysis is based on determining applied stresses and mobilized strength over a trial slide surface in the soil slope, and then a factor of safety is determined by considering these two quantities. Typically many trial failure surfaces are considered to find the most critical surface, or the minimum value. There are various alternative methods that are available in this category. The main difference between different limit equilibrium methods is in the assumptions made about shape of slide surface (circular, plane, wedge, etc.) and equilibrium equation that can be satisfied (force or moment equilibrium or both). Although the "third" dimension, i.e., perpendicular to the plane of the cross-section, is sometimes considered it is usually assumed to be insignificant on the final results. These methods are more commonly used in limit equilibrium approach for slope stability analysis (Duncan and Wright, 2005). This study entirely focused on limit equilibrium slope stability analysis method.

2.10.2 Finite element Method on 2D slope stability analysis

The finite element method is a powerful calculating method in engineering sciences. This method is by far method used for analyzing geotechnical problems. Unlike the limit equilibrium method, the finite element method considers linear and non-linear stress – strain behavior of the soil in calculating the shear stress for the analysis. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffith, 2001).

Today, new analysis method in engineering can be studied with 'FEM' as reference of exact solution. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffith, 2001).

2.10.3 Numerical Analysis Methods

Numerical analysis methods give reasonable approximations to the correct mathematical solution of the governing equations of the mechanics of slope stability. They are, however, much more sophisticated and complicated than limit equilibrium methods: they take into account deformations (strains) and not just forces (stresses) like the more conventional limit equilibrium methods do. Numerical methods have been extensively used in the past several decades due to advances in computing power. In a broad sense, numerical methods can be classified into continuum and discontinuum methods. There are quite a large number of numerical methods that have been presented in the literature to estimate the behavior of systems made of geo materials (Griffith, 2001).

2.10.4 Slice Methods

The slice methods can be divided into two groups: non rigorous and rigorous. Non-rigorous methods satisfy either force or moment equilibrium, whereas rigorous methods satisfy both force and moment equilibrium. The factor of safety estimated from rigorous methods is relatively insensitive to the assumptions made to obtain determinacy (Abramson et al., 2002).

A number of limit equilibrium methods of analysis have been developed to study slope stability problems. The methods are generally divided into three categories, based on the number of equilibrium equations to be satisfied:

- Overall moment equilibrium methods,
- Force equilibrium methods, and
- Moment and force equilibrium methods

2.10.4.1 Ordinary Method of Slices (OMS)

The Ordinary method (OM) satisfies the moment equilibrium for a circular slip surface but neglects both the interstice normal and shear forces. The advantage of this method is its simplicity in solving the FOS since the equation does not require an iteration process. The FOS is based on moment equilibrium and computed by. (Kifle Woldearegay, 2014)

$$F_m=\frac{\sum(C'l+N'\tan\emptyset'')}{\sum W\sin\alpha}\text{...(2.1)}$$

Where: - $N' = (W\cos\alpha - ul)$

u = pore pressure,

l = slice base length and

α = inclination of slip surface at the middle of the slice.

2.10.4.2 Bishop's Simplified Method (BSM)

Bishop's simplified method (BSM) is very common in practice for circular shear surface (SS). This method considers the interslice normal forces but neglects the interslice shear forces (Abramson et al. 2002). It further satisfies vertical force equilibrium to determine the effective base normal force (N'), which is given by

$$P = \frac{1}{m\alpha}\sum(W - \frac{C'l\sin\alpha}{F} \text{-ulcos}\,\alpha) \qquad (2.2)$$

Where, $m\alpha = \cos\alpha(\,1 + \tan\alpha\frac{\tan\emptyset'}{F}$

The Bishop rigorous method (BRM) considers the interslice shear forces (T) in addition to interslice normal forces (E). The method further assumes a unique distribution of their resultant forces and satisfies moment equilibrium of each slice. The interslice T and E forces, and hence the FOS are determined by an iteration procedure.

2.10.4.3 Janbu's Method

The simplified method, generalized method (GPS) and direct method developed by (Janbu, 1954, 1968) are very common instability analysis. The fundamental differences in these methods are briefly reviewed below.

2.10.4.4 Janbu's simplified method

Janbu's simplified method (JSM) is based on a composite SS (i.e. non-circular) and the FOS is determined by horizontal force equilibrium. As in BSM, the method considers interslice normal forces (E) but neglects the shear forces (T). The base normal force (N) is determined in the same way as in BSM and the FOS is computed by:

$$\mathrm{Ff} = \frac{\sum(\mathrm{C'l}+(\mathrm{N}-\mathrm{ul})\tan\emptyset')\sec\alpha}{\sum \mathrm{W}\tan\alpha+\sum\Delta\mathrm{E}} \qquad (2.3)$$

Where, $\sum\Delta\mathrm{E} = \mathrm{E2} - \mathrm{E1}$

2.10.4.5 Janbu's generalized method

Janbu's generalized method (JGM) or Janbu's generalized procedure of slices (GPS) (Janbu ,1973) considers both interslice forces and assumes a line of thrust to determine a relationship for interslice forces. As a result, the FOS becomes a complex function with both interslice forces

$$\mathrm{Ff} = \frac{\sum[(C'l+(N-ul)\tan\emptyset')\sec\alpha]}{\sum(W-(T2-T1))\tan\alpha+\sum(E2-E1)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.4)$$

Where, $\sum \Delta E = E2 - E1$

Similarly, the total base normal force (N) becomes a function of the interslice shear forces (T) as:

$$N = \frac{1}{m\alpha}(W-(T2-T1) - \frac{1}{F}(C'l - ul\tan\emptyset')\sin\alpha) \ldots\ldots\ldots\ldots\ldots.. \quad (2.5)$$

This is the first method that satisfies both force and moment equilibrium. The moment equilibrium for the total sliding mass is explicitly satisfied by considering an infinitesimal slice width (dx) and taking moments about the midpoint of the slice base (Janbu 1957, 1973). The infinitesimal slice width was introduced to avoid the confusion about the point of application of base normal force. This equilibrium condition, in fact, gives the relationship between the interslice forces (E and T) as:

$$T = \tan\alpha t E - \frac{dE}{dX} ht \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.6)$$

Where, $\tan \alpha t$ = slope of the line of thrust

ht = height from the midpoint of the slice base to dE

2.10.4.6 Morgenstern and Price Method

According Morgenstern, N.R and Price, 1965 the Morgenstern Price method (M PM) also satisfies both force and moment equilibriums and assumes the interslice force function.

The method suggests assuming any type of force function, for example half sine, trapezoidal or user defined. The relationships for the base normal force (N) and interslice forces (E, T) are the same as given in JGM. For a given force function, the interslice forces are computed by iteration procedure until Ff (equation 2.3) is equals to Fm which is shown below in equation 2.1 (Janbu, 1973).

$$F_m = \frac{\sum (C'l + (N - ul)\tan \emptyset'')}{\sum W \sin \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.8)$$

2.11 Slope Stabilization methods

If the result of the stability analysis indicate that the roadway slope does not meet the factor safety requirement, then it may be necessary to use slope stabilization methods. Now of day, with advance of technology and development of construction industry several slope stabilization methods to mitigate slope failure along the road and others civil structures developed. Slope stabilization methods can be placed in one of two broad categories:

- Preventive stabilization methods, applied to stable, but potentially unstable natural slopes and slopes to be cut.
- Remedial or corrective treatments applied to existing unstable, moving slopes, or to failed slopes.

According to Abramson, (2001) the stability of any slope will be improved if certain actions are carried out. To be effective, first one must identify the most important controlling process that is affecting the stability of the slope; second, one must determine the appropriate technique to be sufficiently applied to reduce the influence of that process. The mitigative prescription must be designed to fit the condition of the specific slope under study. The analysis of these alternative remedial measures for soil slope problems requires experience and sound decision on the part of the experts. The following sections provide a general introduction to techniques that can be used to mitigate soil slope instability.

2.11.1 Slope Reduction

An increase in slope height may result in increased weight over a potential shear plane (Crozier, 1999). Slope angle can be graded into gentle ones; and if there is not enough room for such extensive grading, terraces or benches may be excavated into the slope.

Benches are required for heights greater than 10 meters in rocks. Benches should be wide enough to contain falling loose materials. Drainage ditches are provided along the benches and toe of slope to control slope run off and minimize erosion and it is required for heights greater than 5 meters in soils. Width of benches should at least be 3.0 meter. A ditch of 1.0 meter depth and 1.5 to 2.0 meter wide is required to catch falling debris from slopes. About benching the following are average slope values (horizontal: vertical) for excavations in different materials. Slope angles are indicative and require site-specific assessment (ERA, 2013).

Table2. 1 Type of Soil and Slope Reduction

Type of Material	Cut slope total height (m)			Remark
	3-6	6-10	10-15	
Residual clay soils	1:1	1:1	2:1	Consider benching when the slope height is above 6 m. Vegetation cover is highly recommended.
Heavy, plastic clay soils	1.5:1	2:1	-	Keeping the slope dry is extremely important.
Granular soils with some clays	1.5:1	2:1	-	Keep a constant slope. Appropriate drainage and vegetation is necessary
Dense transported soils (sub-angular cobbles, gravels and sands in a fine matrix)	0.75:1	1.5:1	2:1	Reduce the upper portion to 1:1 to limit gully formation or widening.
Loose to medium dense transported soils (boulders, sub-angular cobbles and gravels in a fine matrix), or talus	1:1	1.5:1	2:1	Cover the slope with grass and other suitable plants and keep the slope dry.

(Source: ERA, 2013)

2.11.2 Slope Modification

Modification of a slope either by humans or by natural causes can result in changing the slope angle as indicated below in figure 2.7, so that it is no longer at the angle of repose. A mass movement event can then restore the slope to its angle of repose. Undercutting - streams eroding their banks or surf action along a coast can undercut a slope making it unstable (Nelson, 2010)**.**

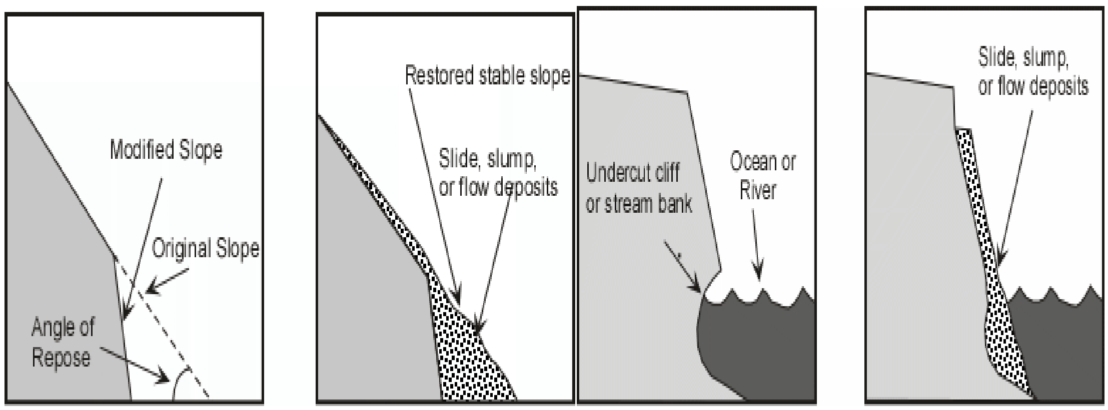

Figure2. 2 Slope Modification

It is known that mass movement can be extremely hazardous and result in extensive loss of life and property. But, in most cases, areas that are prone to such hazards can be recognized with some geologic knowledge, slopes can be stabilized or avoided, and warning systems can be put in place that can minimize such hazards (Nelson, 2010)**.**

2.11.3 Engineering Structures for Mitigation

The general plan to propose a risk management program that included both structural and non-structural mitigating actions must consider the social and economic conditions of the area (Coutinho, 2011). Engineering structures for slope stability are different types of retaining wall, surface or sub surface drainage, individual geo-synthetic or geo-composite materials those are selected depending on site condition.

2.11.4 Retaining Structures

These structures are basically constructed at base and toe of the sliding surface. Accordingly, support at the base can be achieved by the provision surcharge at toe by construction of rock or earth fill while common or crib retaining wall structures are constructed at the base/toe of the slide mass to restrict movement of the slide mass. 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 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.The most common use of retaining walls for slope stabilization is when a cut or fill is required and there is not sufficient space or right- of -way available for just the slope itself. The wall should be deep enough so that the critical slip surface passes around it with an adequate FOS (Abramson, et al., 2001).

2.11.5 Stabilization by Drainage

Stabilization by drainage has been noted as a very effective means of protecting unstable hill slopes and from further sliding. Water was noted to have infiltrated into the weathered and jointed beds which in turn increase both pore and cleft water pressures. The local ground level can be lowered through the installation of curtains every five meters. Stone embankments, using wire meshes, will reduce significantly the buildup of water in the slope such structures are less expensive and more applicable to developing countries (ERA, 2013).

The drainage system is designed to collect storm water runoff from the road surface and right-of-way, convey it along and through the right-of way, and discharge it to an adequate receiving body without causing adverse on- or off-site impacts. Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Rapid removal of storm water from the pavement minimizes the conditions which can result in the hazards of hydroplaning (ERA, 2013).

2.11.6 Stabilization Using Vegetation

Seeding with grasses and legumes reduces surface erosion, which can under certain conditions lead to slope failure. Planting with shrubs adds vegetative cover and stronger root systems, which in turn will enhance slope stability. If not controlled, surface erosion and small, shallow slope failures can lead to larger problems that cannot be controlled. Large scale erosion requires applied engineering technology to correct and control. The terms bioengineering and biotechnical slope protection refers to the use of vegetation as slope protection to arrest and prevent slope failure and surface erosion (Selby, 1993).

2.11.7 Rock slope stabilization methods

Stabilization methods for rock slopes depend on the type of failure mode identified during the field reconnaissance and through stability evaluation. The size of the feature requiring stabilization often is another important consideration when selecting the most cost effective stabilization method. In many situations the preferred approach for stabilizing unstable rock block or mass is to force a controlled failure of rock mass.

In many cases, cut slopes require stabilization to ensure their long-term viability and reduce localized slope failure. Generally, the most effective strategy is to prevent the failure at the source through stabilization, not to install structures to protect against them in the future. There are many methods that can be used to stabilize a rock slope. These include altering the slope geometry, installing drainage, adding reinforcement, or a using combinations of these methods (Abramson et al., 2002).

2.11.8 Reinforcement systems

Most reinforcement systems work to strengthen the rock mass internally by increasing its resistance to shear stress and sliding along fractures. Other systems work externally to protect the rock from weathering and erosion and to add a small amount of structural support.

Rock anchors is most common type of reinforcement, which threaded steel bars or cables that are inserted into the rock via drilled holes and bonded to the rock mass by cement grout or epoxy resins.

Rock anchors can be used to secure a single loosened block or to stabilize an entire rock slope that is affected by a prevalent rock structure. Disadvantages include relatively high cost, susceptibility to corrosion, and lengthy installation times, which can slow down the construction of the rock slope.

The another type of reinforcement is tensioned anchors (a rock bolts), which are used on rock masses that already show signs of instability or on newly cut rock slopes to prevent movement along fractures and subsequent decrease of shearing resistance. Rock bolts are considered a type of active reinforcement due to the post-tensioning they provide, and are used to add compressive stress to joints within a rock mass.

Tensioned rock bolts can require more time to install than dowels because installation involves several steps: drilling, grouting the bond length and inserting the bar or cable, then tensioning the anchor and grouting the free length. Because the tension in the bolt can reduce over time due to creep and become seized by small shears in the rock mass, rock bolts may need periodic retensioning (Abramson et al., 2002).

2.12 Slope stability modeling with Geo-Studio SLOPE/W

Field visiting and laboratory tests are the backbone work of slope stability analysis. Slope stability analysis is needed to know the slope either it is stable, moderately stable or not stable and to provide appropriate mitigation. According to Slope/W software manual published at 2008, for the analysis some input parameters are required which are shear strength parameters (c, ϕ) and unit weight ($\boldsymbol{\gamma}$). The area is undoubtedly and visibly surrounded by unstable and slightly stable slopes, but it needs geotechnical proof with numerical modeling software package Geo-Studio - Slope/W 2007.

CHAPTER THREE

MATERIAL AND METHODS

3.1 Location of the study area

The study area is located along the Werie – Maykinatal road section of the Mekelle - Abi Adi - Adwa road project in Tigray Regional State, northern Ethiopia (Figure 3.1). It is 42 kilometers southeast of Adwa and 788 km North of Addis Ababa. The road section is located at14°16′N 39°27′E latitude and 11°51′N 38°12″E longitude and the elevation varies from 1,370 m to 2,650 m above sea level.

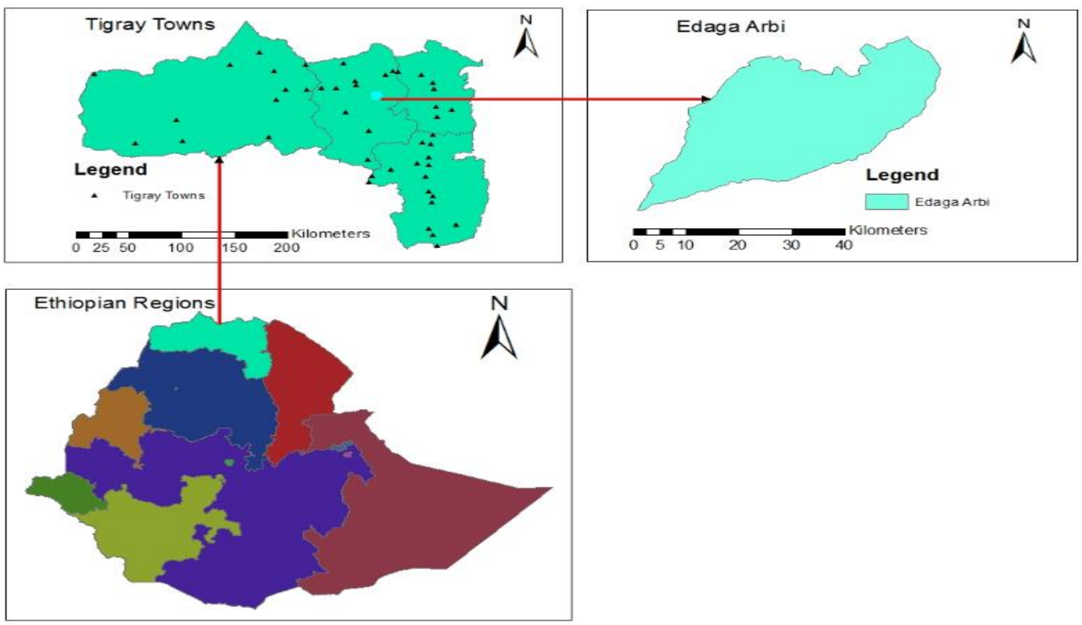

Figure3. 1 Geographic Location of the Study Area

3.2 Study design

The research was conducted by using both experimental and analytical methods. The research was carried out using field survey, laboratory and software analysis. The field survey was carried out using GPS, 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. The field survey included description of soils/rocks, documentation of landslide affected sites (using GPS), test-pit excavations and sampling of soils for laboratory analyses. Laboratory analysis was done on representative soil samples to get material properties and input parameters for slope stability analysis (software analysis). Secondary data was obtained from different literature reviews and communities living around the study area.

3.3 Site Visitation and Data Collection

Werie – Maykinatal road section of the Mekelle - Abi Adi - Adwa road project in the northern part of 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 Surface(localized) erosion, slope failures , ground cracking, ground subsidence, destruction of natural features, damages and tilting of plants, Cracking of roads, displacement of the culverts and deformation of drainage structures ,displacement of trees from high slopes into the vicinity of roads and affected farm and grazing land and photo graphs (Figure 3.2) of landslide affected In landslide affected study area, the samples were collected from the slope crest, middle and toe by faring away a distance of 8 and 6 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 2 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.

Table 3. 1 Location Coordinates of the Study Area

pit No	Location of the sample	Location			Depth (m)	Sampling type	
		N	E	Elevation (m)		Disturbed	Undisturbed
TP1-1	Top	13°54.15′	39°00.37′	1595.4	2.12	Yes	NO
TP1-2	Middle	13°64.45′	39°10.57′	1605.32	2.35	Yes	NO
TP1-3	Toe	13°54.15′	39°00.37′	1618.45	3.0	Yes	NO
TP2-1	Top	13°50.97′	39°00.70′	1388.5	2.1	Yes	NO
TP2-2	Middle	13°59.16′	39°04′	1398.26	2.45	Yes	NO
TP2-3	Toe	14°00.26′	39°14′	1478.36	3.0	Yes	NO

Figure3. 2 Many Cracks on road surface, ground subsidence and Slope instability

- **Collection of secondary data**

Collection of secondary data is very important particularly to know about general conditions of the study area. Even though there was scarcity of secondary data such as; borehole data, ground water level and laboratory tests data, still attempts were made to collect existing data/information which were relevant for the present study. Such secondary data was collected from published and unpublished sources. The secondary data, thus collected for the present study includes; (i) Geological map, (ii) Topographic map, (iii) Metrological data and (iv) Hydrological condition.

- **Reconnaissance survey**

In order to gather general information about the study area a reconnaissance field survey was conducted during May 25/2019 up to May 30/2019. During reconnaissance survey information on landslides and related slope instability problems in the study area was mainly obtained through visual observations and by interviewing the local peoples and concerned organization (Ethiopian road authority north Ethiopia road maintenance directorate direct)

3.4 Climate and Topography

The climatic condition of the study area was warm humid and wet characterized by high inter-annual rainfall variability (800 to 500 mm over the period 1992 - 2007) and in the rainy season the maximum intensities of precipitation varies between 500 and 600 mm per day. Temperature is varies between 11 and 28.1 °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. Rainfall is seasonal with about 70% of total annual rainfall concentrated in July and August, The topography of the study area was characterized by flat terrain, rolling, mountainous and escarpment are 1.4%, 28.1%, 36.7% and 33.9% respectively of landslide affected area.

3.5 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 and test pit excavations for subsurface soil investigations and sampling for laboratory analysis.

3.6 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.

3.7 Software and instruments used for slope stability analysis

The software Geo-studio 2007, MS word and Excel and device mobile camera and GPS were used for the study. Geo-studio 2007 was used to delineate the study area and numerically analysis the slope stability against the landslide respectively; MS word and Excel were used to analyze laboratory and display research data; mobile camera and Garmin GPS were used for documentation and determine the location of landslide affected area respectively.

3.7.1 Method of slices using SLOPE/W software

The slope model was analyzed using SLOPE/W software with the aim of giving the state of the slopes with their factor of safety using Limit Equilibrium Method (LEM).The software computes the factor of safety (FOS) for various shear surfaces (SS), for example circular and non-circular. However, only the circular SS was automatically searched. The method of slices was considered in relation to its application to SLOPE/W and traditional methods of analysis. According to Abramson et al. (2002) slices method is widely used by much computer software because it can accommodate geometry of complex slope, different soil conditions and influence of external boundary loads. Conventionally, the weight of soil lying at a particular point should influence the stress acting normal to that point on sliding surface.

Theoretically, the basic principle of slices 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. This would allow easy estimation of the allowable safety factor of a slide mass. In this study, three soil layers for site1 and site2 obtained from shear strength test, with different strength parameters were used for slope stability analyses. These same shear strength parameters were used in dry conditions. Similarly unit weight of soils for each site above the groundwater table (GWT) considered. The input parameters used in the study are shown in appendix B.

3.8 Geotechnical analysis

Preliminary geotechnical classification and identification tests such as moisture content, bulk density, specific gravity, grain size distribution, liquid limit, plastic limit, and plasticity index and shear strength tests were carried out on the soil samples based on According to ASTM, Each geotechnical test was performed thrice on the same soil sample under the same condition in order to determine the reliability of the geotechnical test results.

3.9 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: (i) the disturbed collected samples at different depths were air dried 3 – 4 days and oven dried at $\pm 105C^{\circ}$ for 16 to 24 hr before carrying out laboratory test. (ii) The undisturbed collected samples using cylinder tube and tied to plastic bag to prevent moisture loss was but not used for direct shear strength and in situ natural moisture content determination. Natural moisture contents and shear strength were determined immediately, after the samples brought to the laboratory. After air or oven dried each sample were weighted for the required laboratory test and the test was carried out in accordance with ASTM standard.

3.10 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 parameters and bulk density for slope stability analysis. The following below were laboratory tests analyzed for investigation of geotechnical characteristics and slope stability analysis.

3.10.1 Water Content Test

This test is performed to determine the water content of soils. The water content is the ratio, expressed as a percentage, of the mass of "pore" or "free" water in a given mass of soil to the mass of the dry soil solids. The knowledge of water content is necessary in soil compaction control in determining consistency limit of soil and for the calculation of stability all kinds of earth works and foundations. ASTM D 2216 - Standard Test Method for Laboratory the moisture contents of the three test samples from the study area was determined in the laboratory following ASTM D2216 testing procedures. An oven drying temperatures of l05°C was used to dry the test samples. For every test sample three sets of samples were prepared in order to avoid error. An appropriate amount of sample was taken for the moisture content determination. Finally the moisture contents for the three sets of samples were calculated using the normal procedures and the natural moisture content of soils of the study area is determined for six samples in appendix A.

3.10.2 Specific Gravity Test

This test is performed to determine the specific gravity of soil by using a pycnometer. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by density bottle. The specific gravity of soil from three test pits was determined in the laboratory according to ASTM D854 testing procedures. Particular representative samples have been taken with oven dried. The given sample has been separated over No.10 sieve 10gm sample and prepared for testing. Then the sample is processed though the test methods. Finally the specific gravity for the three sets of samples was calculated using the normal procedures and the specific gravity of soils of the study area is determined for six samples in appendix A.

3.10.3 In-situ Density (Field Density)

One of the simplest methods of determining the field unit weight of compaction is by the sand replacement method; an in-situ density measurement was conducted by field in -situ apparatus methods at each failed slope where samples have been recovered for laboratory tests. The bulk density was computed upon completion of each test. The field dry densities were later computed based on the natural moisture content from field. Finally the field dry densities for the three sets of samples were calculated using the normal procedures and the unit weight of soils of the study area is determined for six samples in appendix A.

3.10.4 Atterberg Limits

This lab is performed to determine the plastic and liquid limits of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling. 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.

3.10.4.1 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 Casagrande cup method and Fall Cone Method. To determine the liquid limit of the soil samples, the fraction of the soil that passed through the 425μm sieve was weighted (125 g) on a weighing balance and carefully mixed with clean water in order to form a thick homogeneous paste. A groove was cut through the paste (soil sample) that was placed inside the Casangrade's apparatus cup and the numbers of blows were counted and recorded until the groove in the soil closes. The moisture contents were determined and the moisture contents were plotted against the numbers of blows in order to determine the liquid limit. Finally the liquid limit for the three sets of samples was calculated using the normal procedures and the liquid limit of soils of the study area is determined for six samples in appendix A.

3.10.4.2 Plastic limit, PL

This test was carried out to determine the water content at which the soil changes semi-solid to a plastic (flexible) state. The plastic limit is determined by rolling, soil sample was also taken from the soil sample that passes through the 425µm sieve was weighted (30 g) on a weighing balance. Then it was thoroughly mixed with water using the hand until it becomes homogenous and plastic enough to form ellipsoidal-circular shape (i.e. ball). The ball-shaped soil was rolled in a rolling device until the thread cracks or crumbles at about 3 mm diameter. The crumbled sample (3 mm) was then air-dried thus the moisture contents were determined. Two or more determinations are made, and the average water content is reported as the plastic limit. Finally the plastic limit for the three sets of samples was calculated using the normal procedures and the Plastic limit of soils of the study area is determined for six samples in appendix A.

3.10.4.3 Plasticity Index (PI)

Plasticity index was calculated from plastic limit and liquid limit as follows:

PI = LL – PL

3.10.5 Grain Size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils

The sieve analysis of soil from six test pits was determined in the laboratory according to ASTM D422 testing procedures. The soil particles were gently separated from each other. The sieve set (stack of sieves) were arranged in descending order from the top with a retainer beneath it. By taking a measured amount of dry, pulverized soil and passing it through stack of progressively finer sieves with pan at the bottom. The soil filled sieve stack was placed on the mechanical sieve shaker for about 10 minutes. The amount of soil retained on each sieve is measured, and cumulative percentage of soil passing through each is determined. The percentage of finer for each sieve, determined by sieve analysis, is plotted on semilog arithmetic graph paper. The grain size analysis of soils of the study area is determined for six samples in appendix A.

3.10.6 Direct Shear Test

The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall. ASTM D 3080 - Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

Direct shear test is generally conducted on cohesion less soils and it is convenient to perform and it gives result for the strength parameters. Determining the c and ϕ value for the selected slope materials has a contribution to analyze the stability of the slope. This conducted from eight test pits were determined in the laboratory according to ASTM D3080 testing procedures. The initial mass of soil was weighted in the pan. A square sampler was then gently used to collect a representative sample. Each collected sample was placed in a shear box and a load was placed on it both in horizontal and vertical positions and the deformation dial gauges were set at zero. A set of normal loads of 2 kg, 4kg and 6 kg were applied one after the other in successive tests and place it in the direct shear device, Then placed a porous stone and a filter paper in the shear box, the soil sample placed into the shear box and level off the top. Place a filter paper, a porous stone, and a top plate (with ball) on top of the sample and Completed the assembly of the direct shear device and initialized the three gauges (Horizontal displacement gage, vertical displacement gage and shear load gage) to zero. The vertical load was set (or pressure) to a predetermined value, bleeder and the valve then closed and the load was applied to the soil specimen by raising the toggle switch, The motor was started with selected speed so that the rate of shearing was at a selected constant rate, and the horizontal displacement gauge, vertical displacement gage and shear load gage readings. The readings were taken. Readings were Continue until the horizontal shear load peaks and then falls. The readings on the load dial units were recorded, and the procedure was repeated for other samples. The shear strength results were presented as stress-strain curves and the shear stress was plotted against the normal stress, thus the cohesion and angle of internal friction were determined. 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 is considered in relation to its application to SLOPE/W and traditional methods of analysis. Two different conditions; Condition 1 (FOS determination of natural slope) and Condition 2 (FOS determination of modified slope) the slope angle varies from 54.25^0 to 39.81^0 for site1 and 50.44^0 to 34.88^0 for site2.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, three soil layers named as upper, lower and middle soil layer .Different soil layers have different input parameters for slope stability 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 numbers of slices and Mohr Coulomb material model. The complete set of input soil parameters used in the analysis is shown in Table 3.1.The unit weight of the soil used is dry unit weight as the soil is dry (Appendix A) Because of consolidated drained conducted test for shear strength determination of soils of the study area . 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.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

In general, the present study area slope instability is the main problem of the study area. 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, geology, physiography, and hydrological condition and also their effect on slope instability were discussed. Additionally, software results presents the state of the slope at three 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

4.2 Laboratory Test Results and Discussion

For the selected study area six types of laboratory tests are done those tests are water content, grain size analysis, atterberg limits, unit weight, specific gravity and direct shear. For all tests the standard ASTM has been used. For selected scar sides of the study area, all the soil samples are collected from six test pits at different depth and tested by own specific standard their results and procedures are mentioned in the appendix A.

4.2.1 Natural Moisture Content

The natural moisture content of soils the study area is given in Table 4.1. The value of natural moisture content obtained for the soils range between 6.34 to 14.16 %. Based on the result of natural moisture content the soil is gravelly material and tested by own specific standard ASTM D-2216 their results and procedures are mentioned in the appendix A.

Table 4. 1Natural Moisture Contents of the Soils

Test pit No	Depth of sample taken(m)	Natural moisture Content, w (%)
TP1- 1	2.12	13.68
TP1- 2	2.35	12.67
TP1- 3	3.0	14.17
TP2- 1	2.08	7.19
TP2- 2	2.30	6.94
TP2- 3	3.0	6.34

4.2.2 Unit Weight of Soils

Bulk density and unit weight of the soils of the study area is given in Table 4.2. The value of bulk density of the soils varies from 1.69 to 2.45 g/cm^3 and the unit weight 15.03 to 22.95 KN/m^3 . This shows that very loose and loose granular material may have a great probability to initiate the landslide in Werie – Maykinatal road section with other triggering factors of landslide. Their results and procedures are mentioned in the appendix A.

Table 4. 2 Unit Weight of The Soils Samples From Study Area

pit No	Depth of sample taken(m)	Bulk density of the soil(g/cm^3)	Unit weight of the soil (kN/m^3)
TP1- 1	2.12	1.69	15.03
TP1- 2	2.35	1.81	15.81
TP1- 3	3.0	2.023	17.78
TP2- 1	2.08	1.83	17.3
TP2- 2	2.30	2.26	21.13
TP2- 3	3.0	2.45	22.95

4.2.3 Specific Gravity

The specific gravity of soils of the study area is given in Table 4.3. The value of specific gravity obtained for the soils range between 2.58 to 2.63. Based on the result of specific gravity the soil is gravelly material and tested by own specific standard ASTM D-854 their results and procedures are mentioned in the appendix A.

Table 4. 3 The Specific Gravity of the Soils from Study Areas

pit No	Depth of sample taken(m)	The specific gravity of soils
TP1- 1	2.12	2.58
TP1- 2	2.35	2.60
TP1- 3	3.0	2.62
TP2- 1	2.08	2.63
TP2- 2	2.30	2.58
TP2- 3	3.0	2.62

4.2.4 Grain Size Distribution and Soil Classification

The combined grain size distribution curve for soil samples of the study area is shown on Figure 4.1. Most of the study area materials are coarse lessen than 50 percent are passed by sieve number 200(0.075mm). Results indicate generally the materials are categorized as GW and GP-GC and A-2-7 and A-2-6 from USCS and AASHTO respectively .The results and procedures are mentioned in the appendix A.

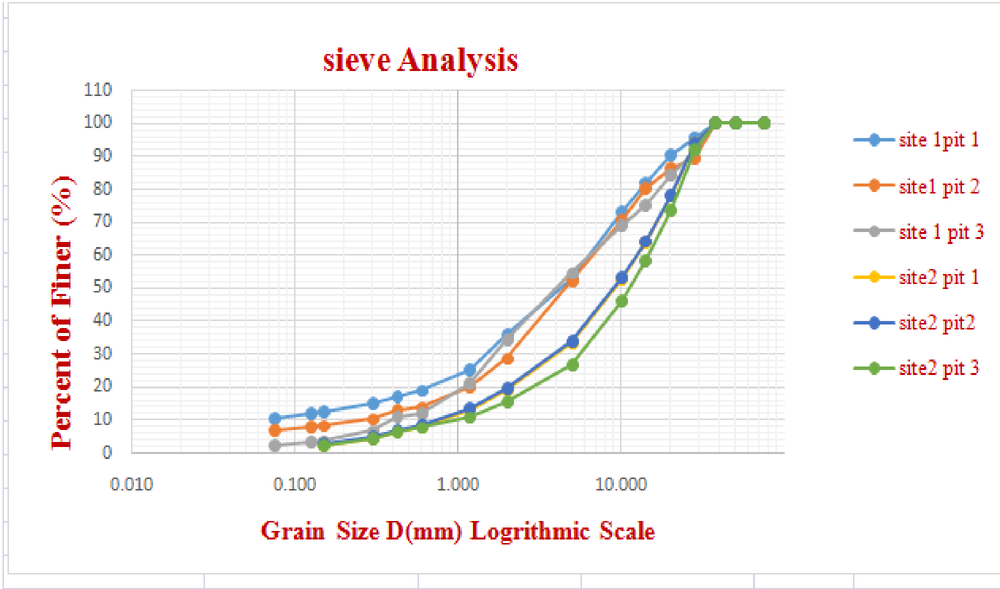

Figure4. 1 Size Distribution Curves for the Soil Samples

Table 4. 4 Summary of the Grain Size Analysis and Soil Classification

Pit No	Specific Gravity (GS)	% Finer (clay and silt)	% Sand	% Gravel	C_u	C_c	Group Symbols
TP1-1	2.58	2.55	61.25	35.7	75	4.687	GP-GC
TP1-2	2.60	1.62	62.48	32.06	30.85	5.5	GW
TP1-3	2.62	1.97	65.37	31.28	23.43	1.54	GW
TP2-1	2.63	1.16	50.56	47.2	16.25	1.18	GW
TP2-2	2.58	1.52	50.56	46.84	17.14	1.63	GW
TP2-3	2.62	1.52	43.56	54.16	12.78	1.96	GP-GC

4.2.5 Atterberg limit of Soil

The Atterberg limit value for soil samples was determined in accordance to AASHTO T89-96. This laboratory is performed to determine the plastic and liquid limits of soils the results, it can be seen that almost all soil types are gravel with clay and sand soil with non plastic up to medium plastic, which covers most part of the cross section of the slope is medium plastic.

The summary of results obtained from moisture content, liquid limit, plastic limit and plasticity index analyses are presented in Figure 4.2 and 4.3 and Table 4.5 and the results and procedures are mentioned in the appendix A.

Table 4. 5 Liquid Limit, Plastic Limit and Plasticity Index of Soils from Study Areas

Pit No	Depth of sample taken(m)	Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)
TP1-1	2.12	13.68	40.45	27.8	12.65
TP1-2	2.35	12.66	38.37	28.56	10.41
TP1-3	3.0	14.19	41.15	30.61	10.54
TP2-1	2.08	7.19	28.99	24.38	4.66
TP2-2	2.30	6.94	28.54	20.50	8.04
TP2-3	3.0	6.37	26.94	19.13	7.81

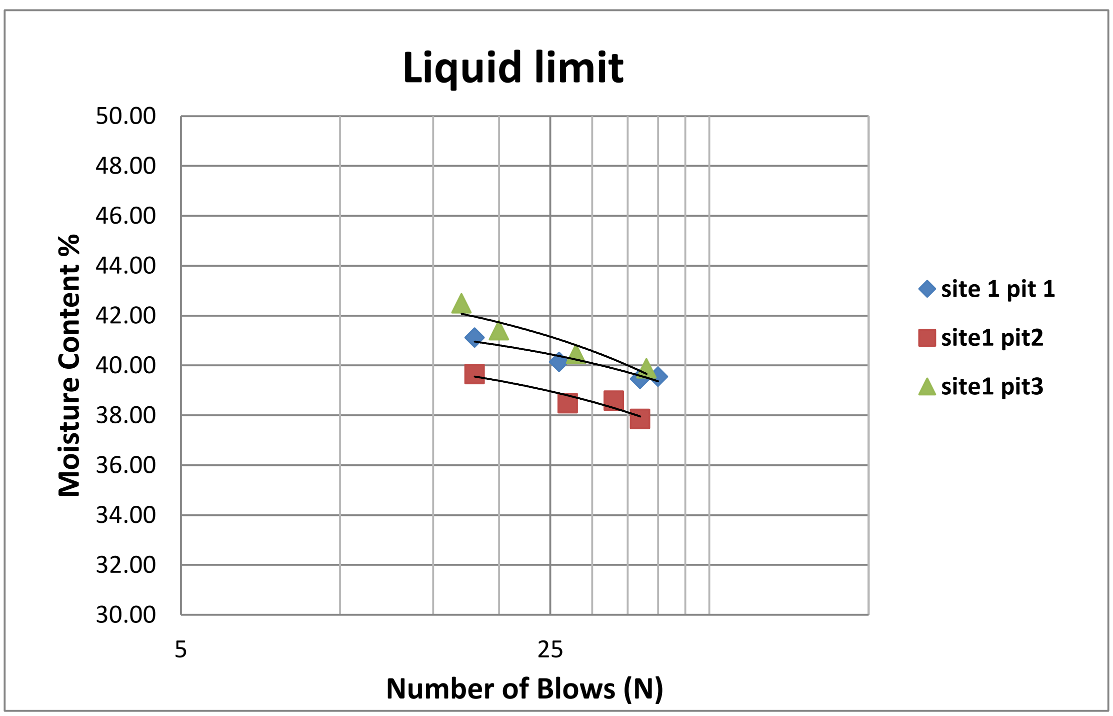

Figure4. 2 Liquid Limit Test Results by Casagrande Cup Method of Site1

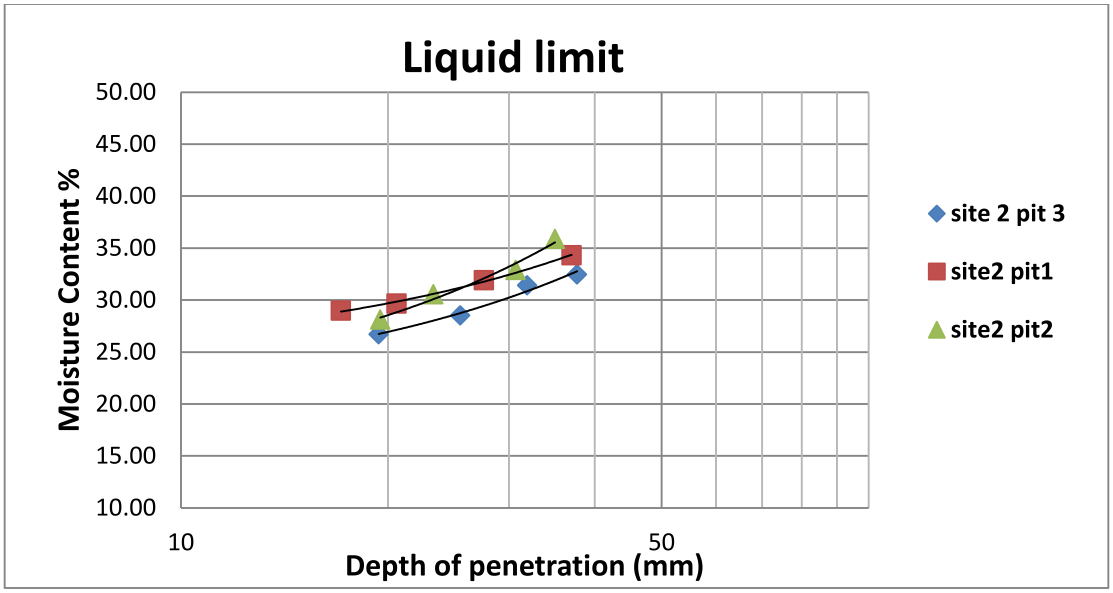

Figure4. 3 Liquid Limit Test Results by Fall Cone Method of Site2

4.2.6 Shear Strength Parameter Determination

The summaries of direct shear test results, as well as their interpretations are tabulated in Tables 4.6. From the direct shear test the following shear strength parameters were obtained which helps in analyzing of slope stability. From the laboratory results, the angle of internal friction (ϕ) of soils varies from 17.16^o to 27.93^o and the cohesion (C) 25.37KN/m^2 to 73.65KN/m^2 respectively. Based on direct shear test results value the soil is loose granular material and tested by own specific standard ASTM D3080 also their results and procedures are mentioned in the appendix A.

Table 4. 6 Summary Direct Shear Test Result of Soils from Study Area

Pit No	Soil layer	angle of internal friction ,ϕ (°)	cohesion ,C (kN/m^2)	Unit weight ,r (kN/m^3)
TP1-1	Upper	24.23	25.37	15.03
TP1-2	Middle	25.24	44.43	15.81
TP1-3	Lower	26.52	49.47	17.78
TP2-1	Upper	17.16	52.73	17.3
TP2-2	Middle	27.93	60.41	21.13
TP2-3	Lower	21.51	73.65	22.95

4.3 Slope Stability Analysis Results

The critical slip surface, CSS and FOS of slope of the study area for two different conditions are presented on (Figure 4.4 and4.5). The CSS passes through the toe of slope and its size also large. The bigger in size of slip surface and passes through the toe of the slope may due to the slope material is weaker than base material.

The results obtained by SLIDE are given in Appendix B and C as before; the LEM software SLOPE/W and SLIDE was found to FOS-values varies from 1.203 to 1.793. According to Milimba (2007), the value actively unstable, FOS < 1: approaching to failure that will fail at some time in response to the destabilizing forces attaining a critical level of activity, marginally Stable, 1 < FOS < 1.5: likely to fail at some time in response to destabilizing forces reaching a certain level of activity and Stable, FOS > 1.5: The margin of stability is sufficiently high to withstand all destabilizing forces.

FOS of natural slope of site1 and site2 was 1 < FOS < 1.5 which are marginally Stable but the FOS of modify slope of site1 and site2 was FOS > 1.5 which are Stable.

Additionally, the increase in FOS from 1.203 to 1.683 (increase by 50 %) of site1 and 1.372 to 1.793 (increase by 54.7 %) of site2 were because of slope angle modification from $54.25^{0}$to $39.81^{0}$of site1 and $50.44^{0}$to $38.88^{0}$of site2. 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 6,8 and 10 m from failure surface.

The result tells as unsafe slope and minimum FOS of the slope of Werie – Maykinatal road section 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 Tables

Table 4. 7 Input Data for Slope Stability Analysis on Site 1

Condition	Soil profile	Slope angle (^{0})	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Angle of internal friction ($^\circ$)	Ground water conditions
1	Upper soil layer	38.66	15.03	25.37	24.23	At great depth
1	Middle soil layer	59.05	15.81	44.42	25.24	At great depth
1	Lower soil layer	63.43	17.78	49.47	26.52	At great depth
2	Upper soil layer	38.66	15.03	25.37	24.23	At great depth
2	Middle soil layer	45.00	15.81	44.42	25.24	At great depth
2	Lower soil layer	34.99	17.78	49.47	26.52	At great depth

Table 4. 8 Summaries of Slide Mass and FOS for Natural Slope of Site1

Method	Total volume (m3)	Total weight (kN)	Total resisting moment (kN-m)	Total activating moment (kN)	Total resisting force (kN)	Total activating force (kN)	FOS
Morgenstern – price	189.72	3014.4	1.1227e+005	93350	1749.8	1454.1	1.203
Ordinary	189.72	3014.4	1.111e+005	93350	-	-	1.19
Bishops	189.72	3014.4	1.124e+005	93350	-	-	1.204
janbu	189.72	3014.4	-	-	1736.1	1464.9	1.185

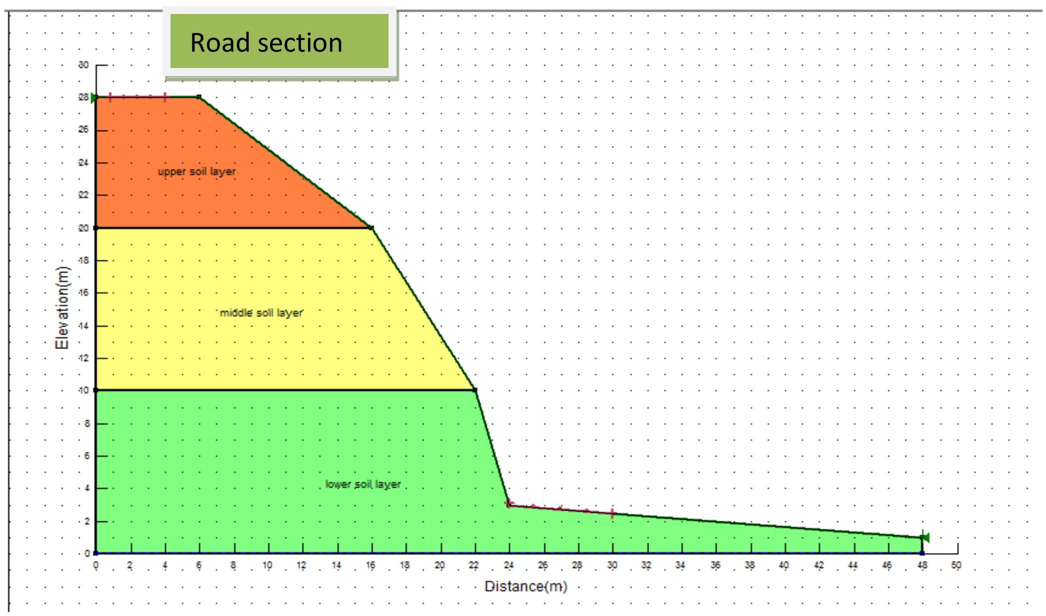

Figure4. 4 Slope Profile at Natural Condition for Site1

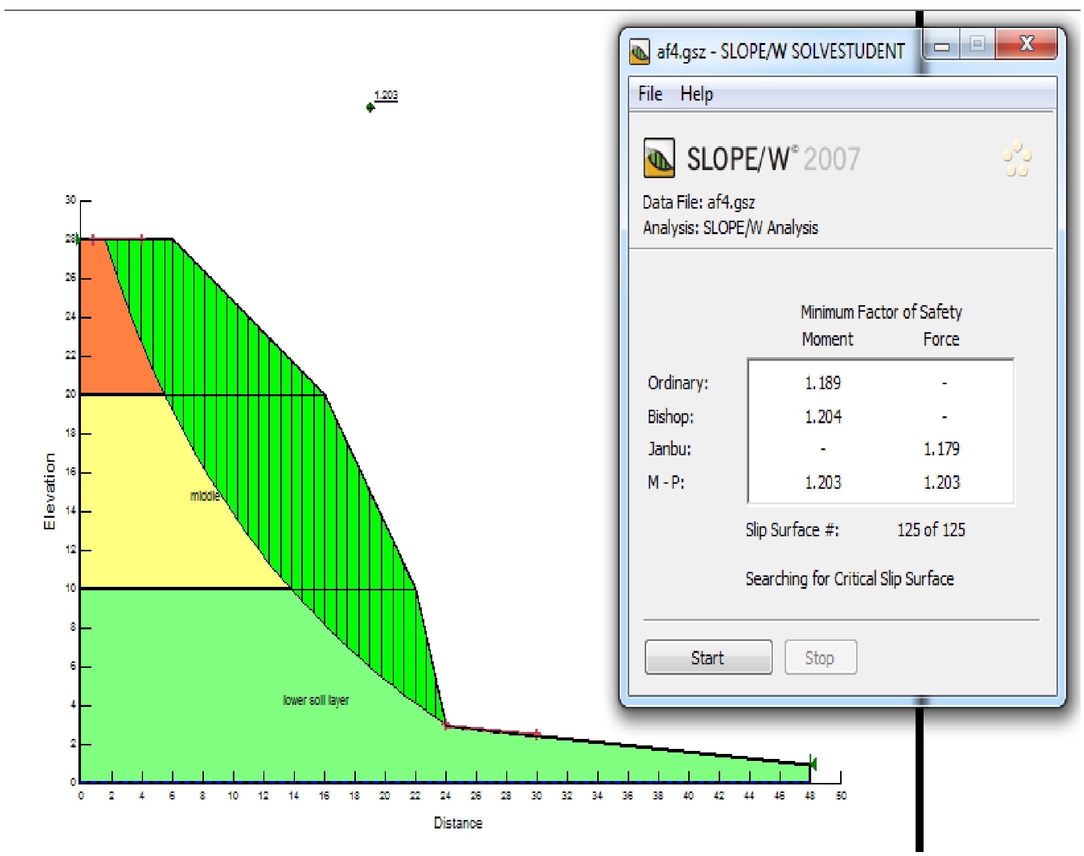

Figure4. 5 The Critical Slip Surface, CSS and FOS for Natural Slope of Site1

Table 4. 9 Summary of Slide Mass and FOS after Modification Slope of Site1

Method	Total volume (m3)	Total weight (kN)	Total resisting moment (kN-m)	Total activating moment (kN)	Total resisting force (kN)	Total activating force (kN)	FOS
Morgenstern – price	286.69	4640	1.3372e+005	79449	3317	1971.6	1.683
Ordinary	286.69	4640	1.2653e+005	79449	-	-	1.593
Bishops	286.69	4640	1.344e+005	79449	-	-	1.692
janbu	286.69	4640	-	-	3238.6	2103.9	1.539

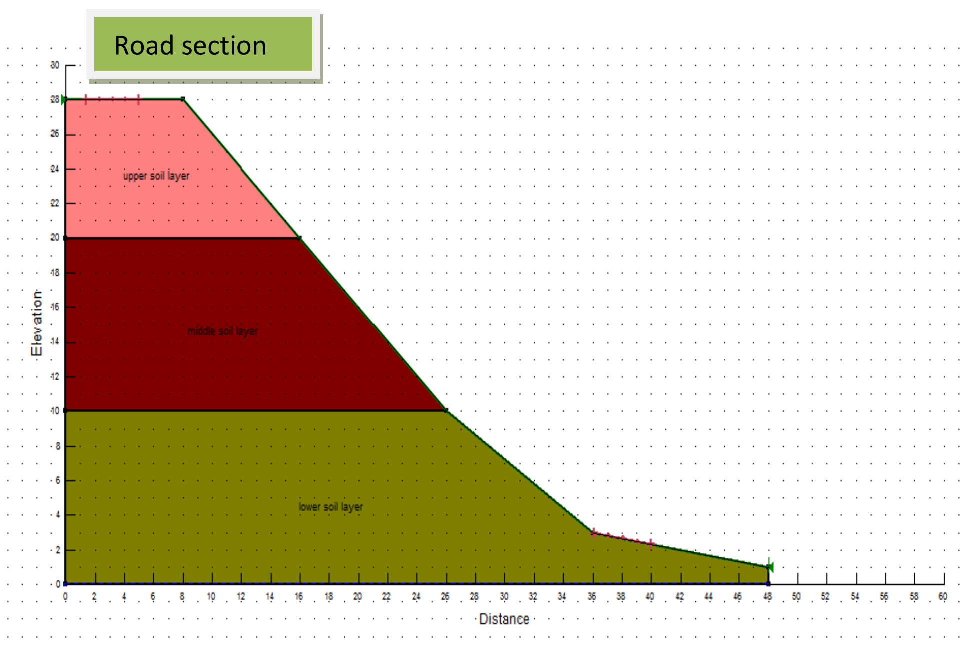

Figure4. 6 Slope Profile after Modification Condition for Site1

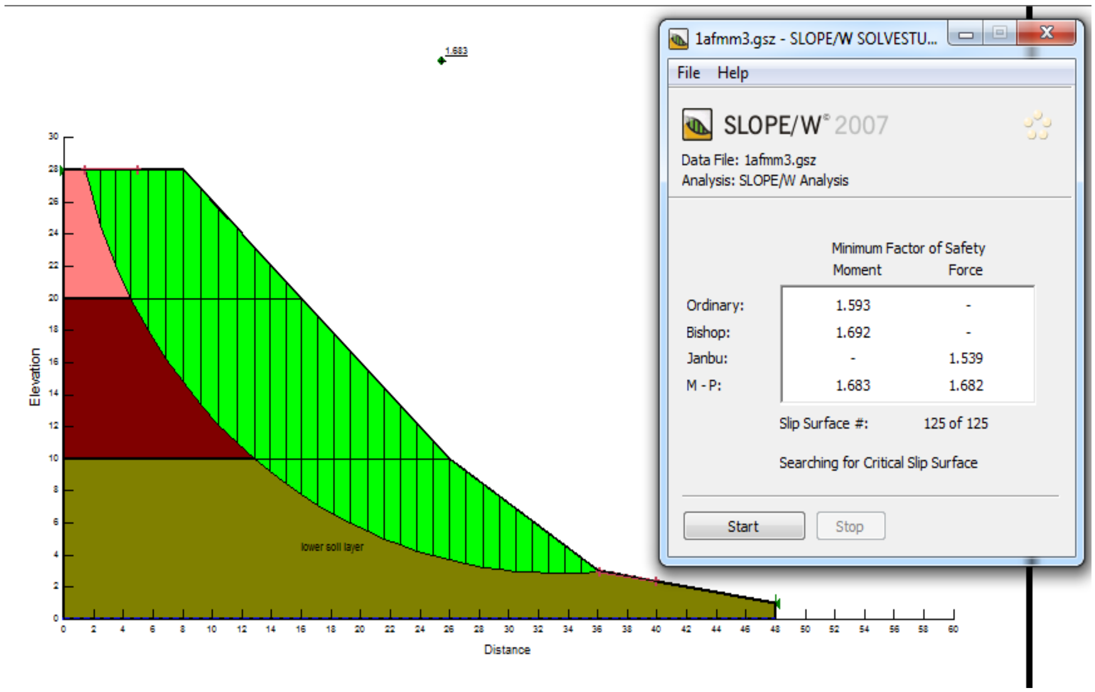

Figure4. 7 The Critical Slip Surface; CSS and FOS after Modification Slope of Site1

Table 4. 10 Input Data for Slope Stability Analysis on Site 2

Condition	Soil profile	Slope angle (^{0})	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Angle of internal friction ($^\circ$)	Ground water conditions
1	Upper soil layer	26.57	17.30	52.73	17.16	At great depth
1	Middle soil layer	63.48	21.13	60.41	27.93	At great depth
1	Lower soil layer	74.05	22.95	73.61	21.51	At great depth
2	Upper soil layer	26.58	17.30	52.73	17.16	At great depth
2	Middle soil layer	48.01	21.13	60.41	27.93	At great depth
2	Lower soil layer	41.18	22.95	73.61	21.51	At great depth

Table 4. 11 Summaries of Slide Mass and FOS for Natural Slope of Site2

Method	Total volume (m3)	Total weight (kN)	Total resisting moment (kN-m)	Total activating moment (kN)	Total resisting force (kN)	Total activating force (kN)	FOS
Morgenstern – price	164.27	3350.5	2.2789e+005	1.6615e+005	2237	1624.7	1.372
Ordinary	164.27	3350.5	2.2776e+005	1.6615e+005	-	-	1.371
Bishops	164.27	3350.5	2.2828e+005	1.6615e+005	-	-	1.374
janbu	164.27	3350.5	-	-	2244.4	1608.3	1.396

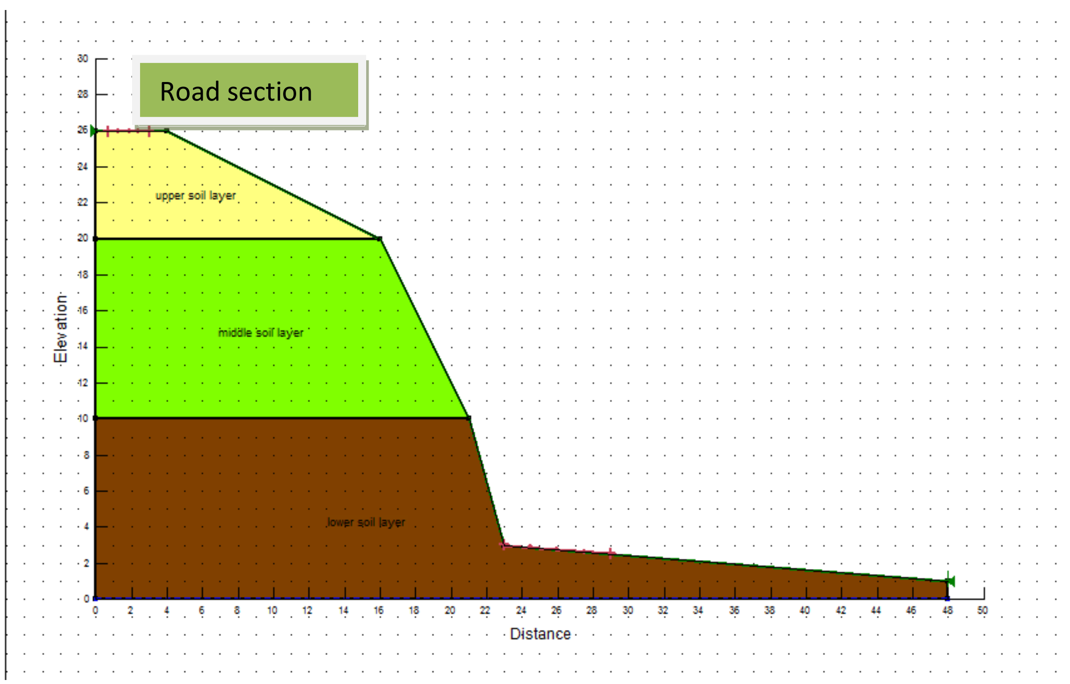

Figure4. 8 Slope profile at natural condition for site2

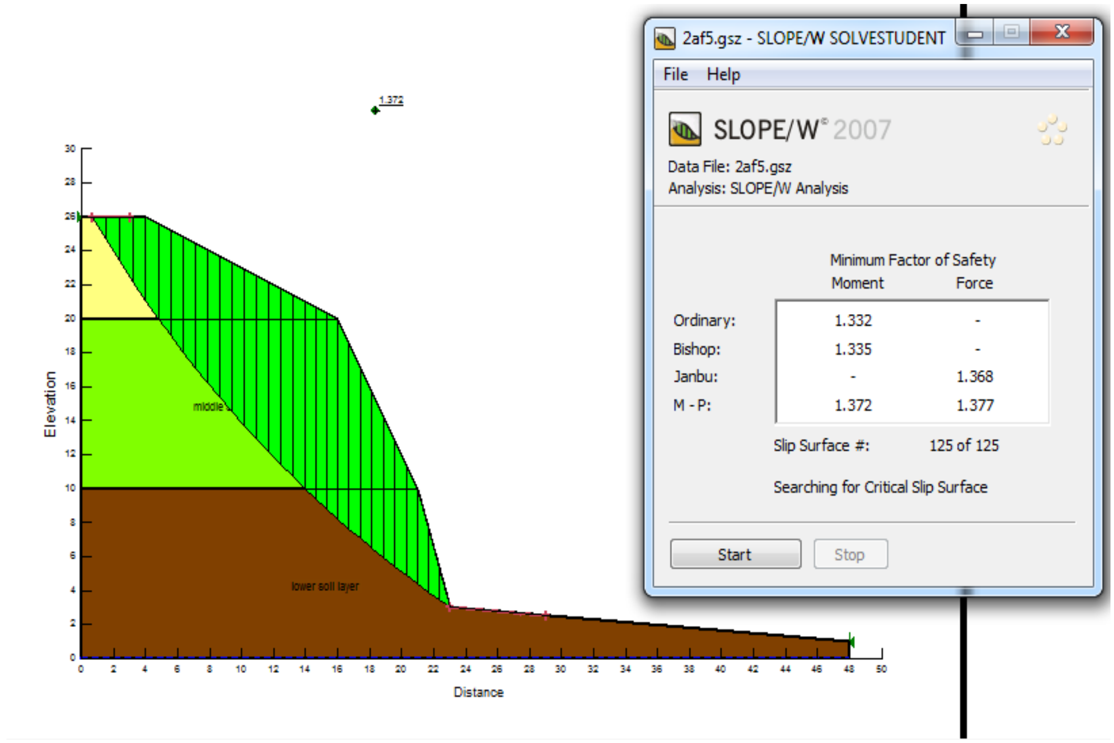

Figure4. 9 The Critical Slip Surface, CSS and FOS for Natural Slope of Site2

Table 4. 12 Summary of Slide Mass and FOS after Modification Slope of Site2

Method	Total volume (m3)	Total weight (kN)	Total resisting moment (kN-m)	Total activating moment (kN)	Total resisting force (kN)	Total activating force (kN)	FOS
Morgenstern – price	275.79	5799.7	1.5378e+005	85781	4081.5	2281	1.793
Ordinary	275.79	5799.7	1.481e+005	85781	-	-	1.726
Bishops	275.79	5799.7	1.5448e+005	85781	-	-	1.801
janbu	275.79	5799.7	-	-	4069.3	2345.3	1.735

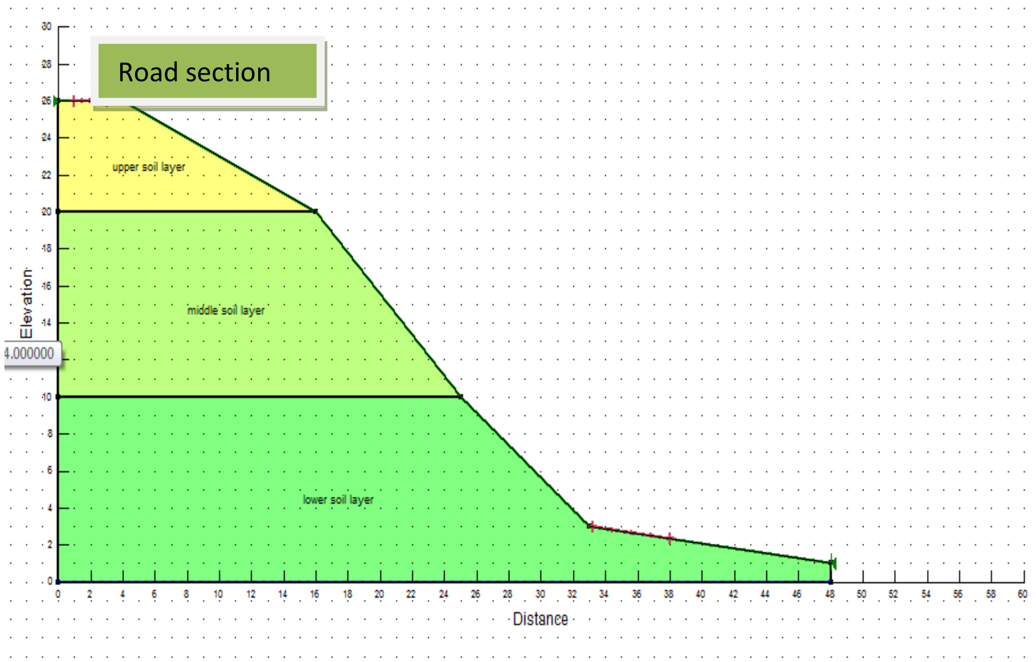

Figure4. 10 Slope Profile after Modification Condition for Site2

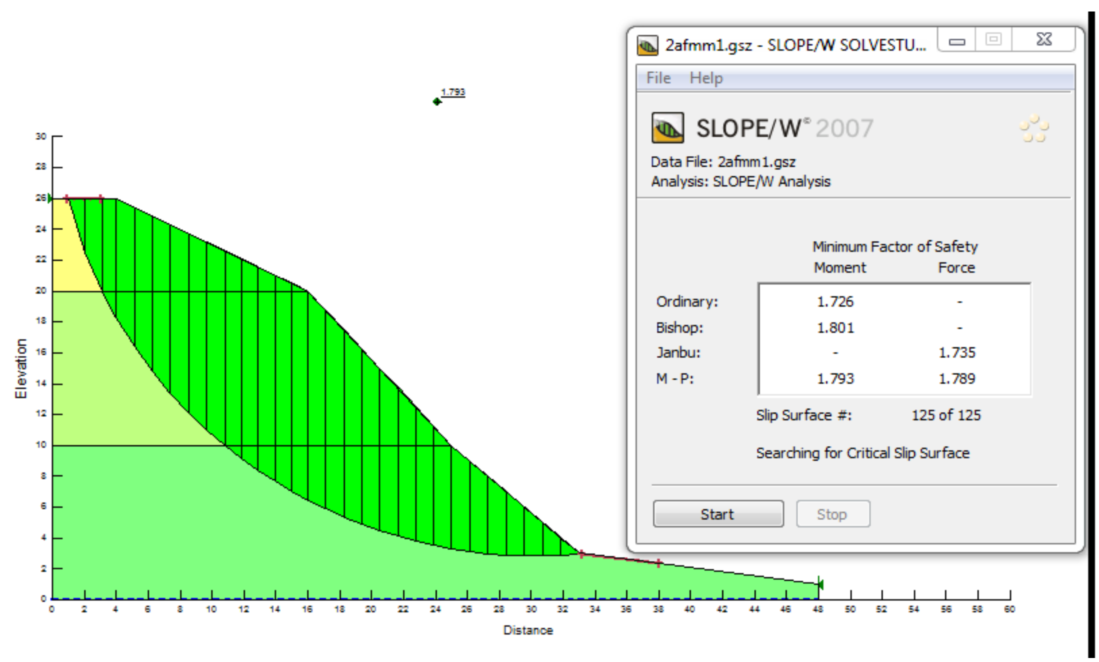

Figure4. 11 The Critical Slip Surface and FOS after Modification Slope of Site2

4.4 Causes and Triggering Factors of Landslides In Study Area

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

4.4.1 Soil Type

The result shows that the soils of the study area g*ravel* soil. This shows that very loose and loose granular material may have a great probability to initiate the landslide in Werie – Maykinatal road section. Hence, the soil type of Werie – Maykinatal road section was the cause for landslide occurred in the study area.

4.4.2 Geologic Factor

The presence of weak zone (highly to moderately weathered and fractured rock) and clay soil with small amount were the main causes for occurrence of landslide in the study area. The following figure mentioned the geological profile of the study area slope which indicates the existing material type and their thickness. The presence of erosion and gulleying at the bottom and top of the road are believed to have great influence on the stability of the slope.

Figure4. 12 Road Section General Side View

4.4.3 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 Werie – Maykinatal road section is considered as another cause for slope instability in that area.

4.5 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 through the toe of the slope as shown on Figure 4.7; hence the failure classified as toe (slope) failure which is one type of rotational failure for homogenous soil conditions; hence it was classified as rotational landslide.

4.6 Consequences of Landslide

Landslide has direct impact on the natural environment causing topographical change; land covers change (vegetation or grassland), land degradation, mass wasting (soil and rock) and socio-economic crises, damages of infrastructures and disruption of traffic flows, reduction of agricultural productivity and leads famine and poverties and suffers future life of local people. 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 ,damage of the road (asphalt), hamper traffic and rarely car accident .

4.7 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.

4.7.1 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, remove 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.

4.7.2 Providing Drainage

The study area was located almost on sloped area and no drainage provided for taking erosion during intense raifall, this makes the slope unstable against sliding. To minimize these problems providing 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 the Werie – Maykinatal road.

4.7.3 Providing Engineering Structure

Providing engineering structures such as gabion retaining wall for damaged area by landslide for Werie – Maykinatal road section Thus, constructing gabion along one 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. In addition and alternatively benching of upslope, maintenance of subsurface drainage, road side surface ditch are considered too.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Erosion and steep slope is believed to have contributed landslides that occurred in study area of Werie – Maykinatal road section. The results from gradation curve of the soil show the soils have 1.62-1.97 % clay and silt, 43.56-65.37 % sand, and 31.28-54.16 % gravel for all the samples. The results from the Atterberg limit tests show the liquid limit (LL) and plastic limit (PL) of the soils range: (a) from 38.97 to 41.15% and 27.80 to 30.61% for site1, and (b) from 26.94 to 29.68% and 19.13 to 24.33% for site2 respectively. The specific gravity value tells as the soil falls in the range specific gravity of gravel soil. The result from moisture content tests obtained for the soils range between 6.34 to 14.16 %.The density and unit weight test result shows the soil of the study area are categorized under coarse-grained soils. According to the unified soil classification system (USCS) and American Association of State Highway Transportation Officials (AASHTO) the main soil groups are gravel of low to medium plasticity with a group symbol GW and GP-GC and A-2-7 and A-2-6 respectively. The result from direct shear test obtained for the soils, the angle of internal friction (ϕ) of soils varies from 17.16^o to 27.93^o and the cohesion (C) $25.37KN/m^2$ to $73.65KN/m^2$ respectively. These parameters are used in the slope stability analysis.

Slope stability analysis revealed that the FOS values for natural slope were found to be 1.203 and 1.372 site1 and site2 respectively. From FOS result it can be understood that the slope of the study area classified as marginal stable. Marginal stable slope or$1< FOS<1.5$ obtained may be due to slope steepness, many crack, rainfall (erosion) and absence of drainage and structure. With the modified slope, the stability analyses resulted in FOS values of1.68 and1.793 for site1 and site2 respectively. 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.The landslide type of the study area is base failure which is one type of a plane slide or a rotational slide. The failure occurs as the CSS passes through the toe of the slope. The landslides at the study area have caused land degradation/mass wasting (soil and rock) and socio-economic costs (damages on infrastructures and disruption of traffic flows).

5.2 Recommendation

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

1. Marginal stable 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 angle modify 54.246 to 39.81^0 for site1 and 50.44 to 38.41^0 for site2 used as a prevention and mitigation measures of the landslide at the study area. In addition their results and designed are mentioned in appendix C.
2. According to erosion, absence of drainage and structure of the study area which causes slope instability (landslide) in the Werie – Maykinatal road section. Hence, providing a gabion retaining wall for damaged area by landslide for site1 and site2 additionally maintenance of drainage, providing surface road side ditch and benching of the upslope can minimize the defect.

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APPENDIX A: LABORATORY TESTS AND RESULTS

1. Moisture Content Test

1.1 Moisture Content Test For Site 1

Table A. 1 Data Sheet for Moisture Content Test Site 1

Observation and Calculation	1	2	3
Specimen number	TP-1	TP-2	TP-3
Mass of empty, clean can + lid,(M1)g	35	35	35
Mass of can, lid, and moist soil,(M2)g	153	111.5	118.8
Mass of can, lid, and dry soil ,(M3)g	138.8	102.9	108.4
Mass of dry soil, M5= M3-M1	103.8	67.9	73.4
MW = Mass of moisture =M2-M3	14.2	8.6	10.4
Water content, W (%)	0.137	0.127	0.142
W (%) =(M2-M3)/(M3-M1)*100	13.68	12.67	14.17
Average	13.51		
Water content, W (%)	13.51		

Result: Average moisture content, W (%) = 13.51%

1.2 Moisture Content Test for Site 2

Table A. 2 Data Sheet for Moisture Content Test Site 2

Observation and Calculation	1	2	3
Specimen number	TP-4	TP-5	TP-6
Mass of empty, clean can + lid,(M1)g	10.7	10.7	10
Mass of can, lid, and moist soil,(M2)g	144.9	157	172.6
Mass of can, lid, and dry soil ,(M3)g	135.9	147.5	162.9
Mass of dry soil, M5= M3-M1	125.2	136.8	152.9

MW = Mass of moisture =M2-M3	9	9.5	9.7
Water content, W (%)	0.07	0.07	0.06
W (%) =(M2-M3)/(M3-M1)*100	7.19	6.94	6.34
Average	6.83		
Water content, W (%)	6.83		

Result: Average moisture content, W (%) = 6.84%

2. **Specific Gravity Test**

2.1 Specific Gravity Test For Site 1

Table A. 3 Data Sheet for specific gravity Test Site 1

specific gravity	TP1-1			TP1-2			TP1-3		
Description	1	2	3	1	2	3	1	2	3
pynometer bottle No	1A	1B	1C	2A	2B	3C	3A	3B	3C
mass of empty,clean pynometer (Wp),g	34	34	34	34	34	34.0	34	34	35.0
mass of pynometer + dry soil(Wps),g	47	46	45	47	47	47.0	44	47	46.0
mass of pynometer + dry soil + water(WB),g	93	93	93	93	93	93.0	91	93	91.0
mass of pynometer +water(WA),g	85	86	86	85	85	85.0	85	85	84.00
Wo=(Wps-WP)	13	12	11	13	13	13.0	10	13	11.0
WA-WB	-8	-7	-7	-8	-8	-8.00	-6	-8	-7.0
Wo/Wo+(WA-WB)	2.6	2.4	2.75	2.6	2.6	2.60	2.5	2.6	2.750
Gs=Wo/Wo+(WA-WB)	2.6	2.4	2.75	2.6	2.6	2.60	2.5	2.6	2.750
specific gravity	**2.583**			**2.600**			**2.620**		
Average specific gravity (GS)	**2.601**								

2.2 Specific Gravity Test For Site 2

Table A. 3 Data Sheet for specific gravity Test Site 2

specific gravity	TP2-1			TP2-2			TP2-3		
Description	1	2	3	1	2	3	1	2	3
pynometer bottle No	4A	4B	4C	4A	4B	4C	6A	6B	6C
mass of empty,clean pynometer (Wp),g	34	34	35.0	34	34	34.0	34	34	34.0
mass of pynometer + dry soil(Wps),g	49	47	49.0	46	47	45.0	47	50	47.0
mass of pynometer + dry soil + water(WB),g	94	93	95.0	93	94	93.0	94	96	94.0
mass of pynometer +water(WA),g	85	85	86.0	86	86	86.0	86	86	86.0
Wo=(Wps-WP)	15	13	14.0	12	13	11.0	13	16	13.0
WA-WB	-9	-8	-9.0	-7	-8	-7.0	-8	-10	-8.0
Wo/Wo+(WA-WB)	2.5	2.6	2.8	2.4	2.6	2.75	2.6	2.6 7	2.6
Gs=Wo/Wo+(WA-WB)	2.5	2.6	2.8	2.4	2.6	2.75	2.6	2.7	2.6
specific gravity	**2.63**			**2.58**			**2.62**		
Average specific gravity (GS)	2.61								

3. Unit Weight Test

3.1 Unit Weight Test For Site 1

Table A. 5 Data Sheet for Unit Weight Test Site 1

Observation and Calculation	TP1-1	TP1-2	TP1-3
weight of wet soil from hole(Ww),g	765	1188	1181
weight of cylinder +sand (before pouring)W1,g	5447	5771	5912
weight of cylinder +sand (after pouring)W4,g	4052	3947	4242
weight of sand in hole (Wh=W1-W4-W2)g	939	1368	1214
volume of hole (Vh=Wh/rs),cm^3	651.18	948.68	841.89
Bulk density of the soil (rh=(Ww/Vh)*rs),g/cm^3	1.69	1.81	2.023
Water content, W (%)	12.67	14.169	13.78
Dry Density) of soil (rd=rh/1+w(%)),g/cm^3	1.503	1.581	1.778
Average Dry density of the soil (rs)	**1.62g/cm^3**		

Result: Average Dry unit weight (Dry Density) of the soil for site1=1.62g/cm^3

3.2 Unit Weight Test For Site 2

Table A. 4 Data Sheet for Unit Weight Test Site 2

Observation and Calculation	TP21	TP2-2	TP2-3
weight of wet soil from hole(Ww),g	1342	1463	1619
weight of cylinder +sand (before pouring)W1,g	6211	5977	5838
weight of cylinder +sand (after pouring)W4,g	4249	4174	4007
weight of sand in hole (Wh=W1-W4-W2)g	1506	1347	1375
volume of hole (Vh=Wh/rs),cm^3	1044.38	934.12	953.54
Bulk density of the soil (rh=(Ww/Vh)*rs),g/cm^3	1.853	2.26	2.45
Water content, W (%)	7.1	6.94	6.373
Dry Density) of soil (rd=rh/1+w(%)),g/cm^3	1.730	2.113	2.295
Average Dry density of the soil (rs)	2.046g/cm^3		

Result: Average Dry unit weight (Dry Density) of the soil for site2=2.046g/cm^3 or 20.46kN/m^3

4. Grain Size Analysis Test

4.1 Grain Size Analysis Test For Site 1 Pit 1(Tp1-1)

Table A. 5 Data Sheet for Grain Size Analysis Test Site 1 pit -1 (TP1-1)

Diameter (mm)	Mass of empty sieve (kg)	Mass of sieve &soil retained(kg)	Soil retained (kg)	Percent retained (%)	Cummulative % retained	Percent Passing (finer) (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.127	1.127	0.000	0.000	0.000	100.000
37.500	1.711	1.887	0.176	8.800	0.000	100.000
28.000	1.730	1.819	0.089	4.450	4.450	95.550
20.000	1.632	1.737	0.105	5.250	9.700	90.300
14.000	1.358	1.528	0.170	8.500	18.200	81.800
10.000	1.328	1.502	0.174	8.700	26.900	73.100
4.750	1.372	1.782	0.410	20.500	47.400	52.600
2.000	0.434	0.771	0.337	16.850	64.250	35.750
1.180	0.492	0.708	0.216	10.800	75.050	24.950
0.600	0.391	0.516	0.125	6.250	81.300	18.700
0.425	0.359	0.391	0.032	1.600	82.900	17.100
0.300	0.364	0.409	0.045	2.250	85.150	14.850
0.150	0.341	0.391	0.050	2.500	87.650	12.350
0.125	0.422	0.432	0.010	0.500	88.150	11.850
0.075	0.413	0.445	0.032	1.600	89.750	10.250
pan	0.418	0.437	0.019	0.950	90.700	9.300

M=2kg and Mi=1.99kg

Mass loss during sieve analysis$\frac{M-Mi}{M} * 100$ =$\frac{2-1.99}{2} * 100$=0.5% (ok if less than 2%)

From Grain Size Distribution Table:

%Gravel =35.7%

%sand-=61.25%

%fines =2.55%

Result: D10=0.08mm CU=D60/D10=75

D30=1.5mm CC= (D30)^2/(D60*D10)=4.6875

D60=6mm

4.2 Grain Size Analysis Test For Site 1 Pit 2(TP1-2)

Table A. 8 Data Sheet for Grain Size Analysis Test Site 1 pit -2 (TP1-2)

Diameter (mm)	mass of empty sieve (kg)	mass of sieve &soil retained(kg)	soil retained (kg)	percent retained (%)	commu lative % retained	percent passing(finer) (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.127	1.127	0.000	0.000	0.000	100.000
37.500	1.711	1.821	0.110	4.400	0.000	100.000
28.000	1.730	1.995	0.265	10.600	10.600	89.400
20.000	1.632	1.710	0.079	3.140	13.740	86.260
14.000	1.388	1.538	0.150	6.000	19.740	80.260
10.000	1.328	1.576	0.248	9.920	29.660	70.340
4.750	1.372	1.829	0.457	18.280	47.940	52.060
2.000	0.434	1.021	0.587	23.480	71.420	28.580
1.180	0.492	0.711	0.219	8.760	80.180	19.820
0.600	0.391	0.541	0.150	6.000	86.180	13.820
0.425	0.359	0.381	0.022	0.880	87.060	12.940
0.300	0.364	0.432	0.068	2.720	89.780	10.220
0.150	0.341	0.392	0.051	2.040	91.820	8.180
0.125	0.422	0.430	0.008	0.320	92.140	7.860
0.075	0.413	0.439	0.026	1.040	93.180	6.820
pan	0.418	0.438	0.020	0.800	93.980	6.020

M=2.5kg and Mi=2.46kg

Mass loss during sieve analysis$\frac{M-Mi}{M} * 100 = \frac{2.5-2.46}{2.5} * 100$=1.62% (ok if less than 2%)

From Grain Size Distribution Table:

%Gravel =34.06%

%sand-=62.48%

%fines =1.62%

Result: D10=0.23mm CU=D60/D10=30.87

D30=3mm CC= (D30)^2/(D60*D10)=5.51

D60=7.1mm

4.3 Grain Size Analysis Test For Site 1 Pit 3(TP1-3)

Table A. 9 Data Sheet for Grain Size Analysis Test Site 1 pit -3 (TP1-3)

Diameter (mm)	mass of empty sieve (kg)	mass of sieve &soil retained(kg)	soil retained (kg)	percent retained (%)	commulative % retained	percent passing (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.127	1.127	0.000	0.000	0.000	100.000
37.500	1.711	1.711	0.000	0.000	0.000	100.000
28.000	1.730	1.985	0.255	8.500	8.500	91.500
20.000	1.632	1.851	0.220	7.317	15.817	84.183
14.000	1.388	1.665	0.277	9.233	25.050	74.950
10.000	1.328	1.515	0.187	6.233	31.283	68.717
4.750	1.372	1.800	0.428	14.267	45.550	54.450
2.000	0.434	1.037	0.603	20.100	65.650	34.350
1.180	0.492	0.895	0.403	13.433	79.083	20.917
0.600	0.391	0.663	0.272	9.067	88.150	11.850
0.425	0.359	0.390	0.031	1.033	89.183	10.817
0.300	0.364	0.489	0.125	4.167	93.350	6.650
0.150	0.341	0.434	0.093	3.100	96.450	3.550
0.125	0.422	0.428	0.006	0.200	96.650	3.350
0.075	0.413	0.450	0.037	1.233	97.883	2.117
pan	0.418	0.440	0.022	0.733	98.617	1.383

M=3kg and Mi=2.959kg

Mass loss during sieve analysis$\frac{M-Mi}{M} * 100 = \frac{3-2.959}{3} * 100$=1.97% (ok if less than 2%)

From Grain Size Distribution Table:

%Gravel =31.28%

%sand-=65.37%

%fines =1.97%

Result: D10=0.3mm CU=D60/D10=23.33

D30=1.8mm CC= (D30)^2/(D60*D10)=1.54

D60=7.0mm

4.4 Grain Size Analysis Test For Site 2 Pit 1(TP2-1)

Table A. 10 Data Sheet for Grain Size Analysis Test Site 2 pit -1 (TP2-1)

sieve No diameter (mm)	mass of empty sieve (kg)	mass of sieve &soil retained(kg)	soil retained (kg)	percent retained (%)	commulative % retained	percent passing (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.128	1.128	0.000	0.000	0.000	100.000
37.500	1.711	1.711	0.000	0.000	0.000	100.000
28.000	1.730	1.882	0.152	6.080	6.080	93.920
20.000	1.615	2.018	0.403	16.120	22.200	77.800
14.000	1.358	1.711	0.353	14.120	36.320	63.680
10.000	1.328	1.600	0.272	10.880	47.200	52.800
5.000	1.372	1.861	0.489	19.560	66.760	33.240
2.000	0.434	0.789	0.355	14.200	80.960	19.040
1.180	0.492	0.644	0.152	6.080	87.040	12.960
0.600	0.391	0.513	0.122	4.880	91.920	8.080
0.425	0.359	0.402	0.043	1.720	93.640	6.360
0.300	0.364	0.412	0.048	1.920	95.560	4.440
0.150	0.341	0.396	0.055	2.200	97.760	2.240
pan	0.418	0.447	0.029	1.160	98.920	1.080

Result: D10=0.8mm CU=D60/D10=16.25

D30=3.5mm CC= (D30)^2/(D60*D10)=1.18

D60=13mm

4.5 Grain Size Analysis Test For Site 2 Pit 2(TP2-2)

Table A. 11 Data Sheet for Grain Size Analysis Results Site 2 pit -2 (TP2-2)

sieve No diameter (mm)	mass of empty sieve (kg)	mass of sieve &soil retained(kg)	soil retained (kg)	percent retained (%)	commulative % retained	percent passing (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.128	1.128	0.000	0.000	0.000	100.000
37.500	1.711	1.715	0.004	0.160	0.000	100.000
28.000	1.730	1.937	0.207	8.280	8.280	91.720
20.000	1.615	2.074	0.459	18.360	26.640	73.360
14.000	1.358	1.737	0.379	15.160	41.800	58.200
10.000	1.328	1.633	0.305	12.200	54.000	46.000

4.750	1.372	1.854	0.482	19.280	73.280	26.720
2.000	0.434	0.711	0.277	11.080	84.360	15.640
1.180	0.492	0.609	0.117	4.680	89.040	10.960
0.600	0.391	0.469	0.078	3.120	92.160	7.840
0.425	0.359	0.395	0.036	1.440	93.600	6.400
0.300	0.364	0.410	0.046	1.840	95.440	4.560
0.150	0.341	0.394	0.053	2.120	97.560	2.440
pan	0.418	0.456	0.038	1.520	99.080	0.920

M=2.5kg and Mi=2.481kg

Mass loss during sieve analysis$\frac{M-Mi}{M} * 100 = \frac{2.5-2.481}{2.5} * 100$=0.76% (ok if less than 2%)

From Grain Size Distribution Table:

%Gravel =54.16%

%sand-=43.56%

%fines =1.52%

Result: D10=0.7mm CU=D60/D10=15.36

D30=3.5mm CC= (D30)^2/(D60*D10)=1.63

D60=10.75mm

4.6 Grain Size Analysis Test For Site 2 Pit 3(TP2-3)

Table A. 12 Data Sheet for Grain Size Analysis Site 2 pit -3 (TP2-3)

sieve No diameter (mm)	mass of empty sieve (kg)	mass of sieve &soil retained(kg)	soil retained (kg)	percent retained (%)	commulative % retained	percent passing (%)
75.000	1.052	1.052	0.000	0.000	0.000	100.000
50.000	1.128	1.128	0.000	0.000	0.000	100.000
37.500	1.711	1.715	0.004	0.160	0.000	100.000
28.000	1.730	1.937	0.207	8.280	8.280	91.720
20.000	1.615	2.074	0.459	18.360	26.640	73.360
14.000	1.358	1.737	0.379	15.160	41.800	58.200
10.000	1.328	1.633	0.305	12.200	54.000	46.000
4.750	1.372	1.854	0.482	19.280	73.280	26.720
2.000	0.434	0.711	0.277	11.080	84.360	15.640
1.180	0.492	0.609	0.117	4.680	89.040	10.960
0.600	0.391	0.469	0.078	3.120	92.160	7.840

0.425	0.359	0.395	0.036	1.440	93.600	6.400
0.300	0.364	0.410	0.046	1.840	95.440	4.560
0.150	0.341	0.394	0.053	2.120	97.560	2.440
pan	0.418	0.456	0.038	1.520	99.080	0.920

M=2.5kg and Mi=2.481kg

Mass loss during sieve analysis$\frac{M-Mi}{M} * 100 = \frac{2.5-2.481}{2.5} * 100$=0.76% (ok if less than 2%)

From Grain Size Distribution Table:

%Gravel =54.16%

%sand-=43.56%

%fines =1.52%

Result: D10=0.9mm CU=D60/D10=12.78

D30=4.5mm CC= (D30)^2/(D60*D10)=1.96

D60=11.5mm

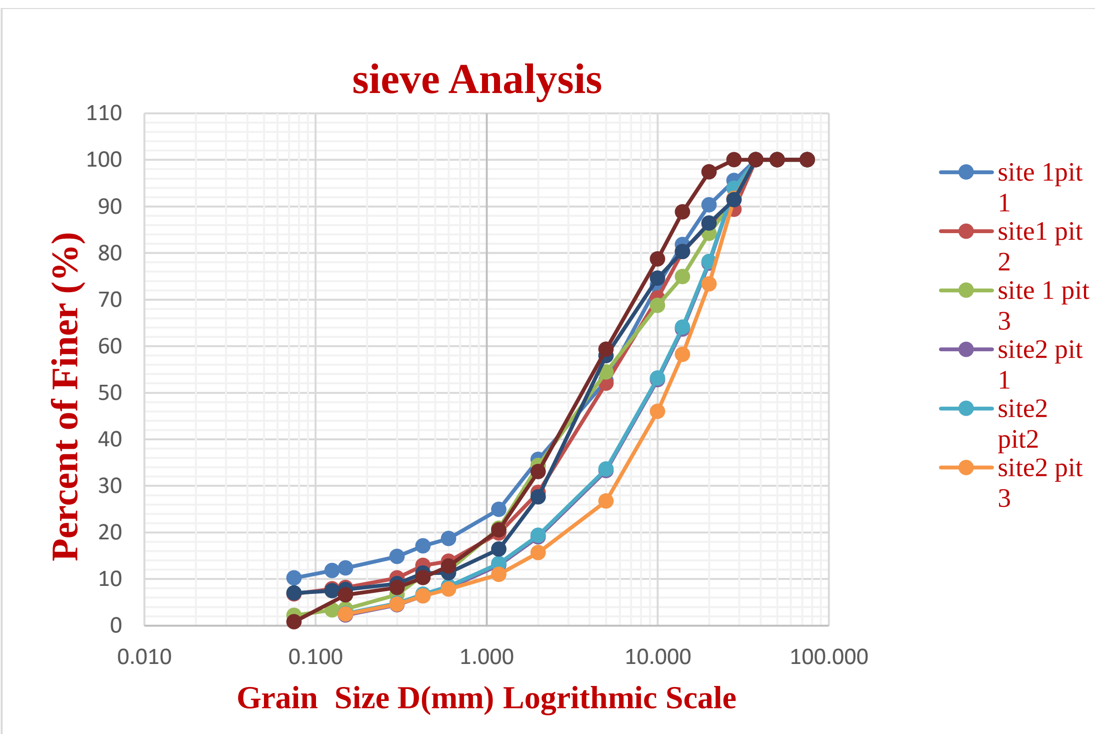

Figure A. 1 Grain Size Distribution Graph for site1 and site2

5. Atterberg Limit Test

5.1 Liquid Limit and Plastic Limit Test For Site 1 Pit 1(Tp1-1)

Table A. 13 Data Sheet for Liquid Limit Test Site 1 pit -1 (TP1-1)

`Can No	H2	KP	Y4	AF
Can Wt(gm) ,M1	18.41	18.54	11.73	13.26
Can + Wet soil (gm),M2	46.62	38.37	34.673	36.3
Can + Dry soil (gm) ,M3	38.4	32.69	28.18	29.77
M_2-M_3	8.22	5.68	6.493	6.53
M_3-M_1	19.99	14.15	16.45	16.51
No Blow ,N	18	26	37	40
Moisture Content, W (%)	41.12	40.14	39.47	39.55
From the flow curve at no blow (N)=25 ,LL	LL=40.45%			

Table A. 14 Data Sheet for Plastic Limit Test Site 1 pit -1 (TP1-1)

`Can No	A+	H*
Can Wt(gm) ,M1	108.42	57.12
Can + Wet soil (gm),M2	139.19	86.39
Can + Dry soil (gm) ,M3	132.49	80.03
M_2-M_3	6.7	6.36
M_3-M_1	24.07	22.91
Moisture Content, W (%)	27.84	27.76
Average Moisture Content, W (%)	27.80	
Plastic Limit ,PL	PL= 27.80%	

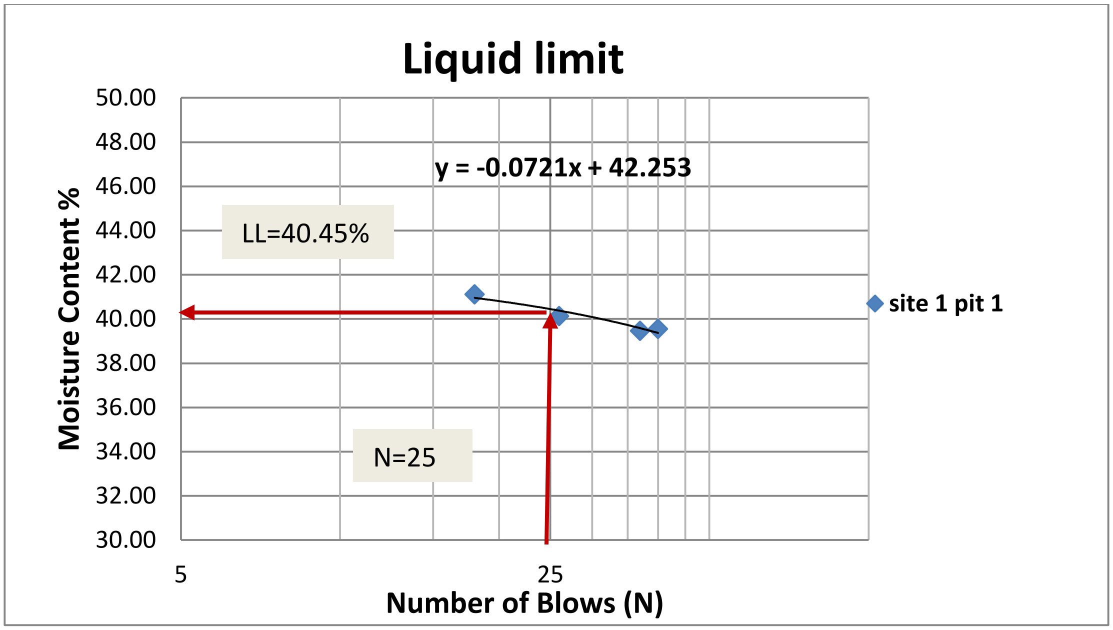

Figure A. 2 Liquid limit Graph for site1 pit 1(TP1-1)

Final Results: LL=40.45%

PL=27.80%

PI=LL-PL=12.65%

5.2 Liquid Limit and Plastic Limit Test For Site 1 Pit 2(TP1-2)

Table A. 15 Data Sheet for Liquid Limit Test Site 1 pit -2 (TP1-2)

`Can No	A	PG	M	R3
Can Wt(gm) ,M1	11.77	17.75	12.22	17.58
Can + Wet soil (gm),M2	34.38	46.5	39.98	44.05
Can + Dry soil (gm) ,M3	27.96	38.51	32.25	36.78
M_2-M_3	6.42	7.99	7.73	7.72
M_3-M_1	16.19	20.76	20.03	19.2
No Blow ,N	18	27	33	37
Moisture Content, W (%)	39.65	38.49	38.59	37.86
From the flow curve at no blow (N)=25 ,LL	LL=38.97%			

Table A. 16 Data Sheet for Plastic Limit Test Site 1 pit -2 (TP1-2)

`Can No	CC	K17
Can Wt(gm) ,M1	18.08	11.82
Can + Wet soil (gm),M2	60.63	39.98
Can + Dry soil (gm) ,M3	51.2	33.71
M_2-M_3	9.43	6.27
M_3-M_1	33.12	21.89
Moisture Content, W (%)	28.47	28.64
Average Moisture Content, W (%)	28.56	
Plastic Limit ,PL	PL=28.56%	

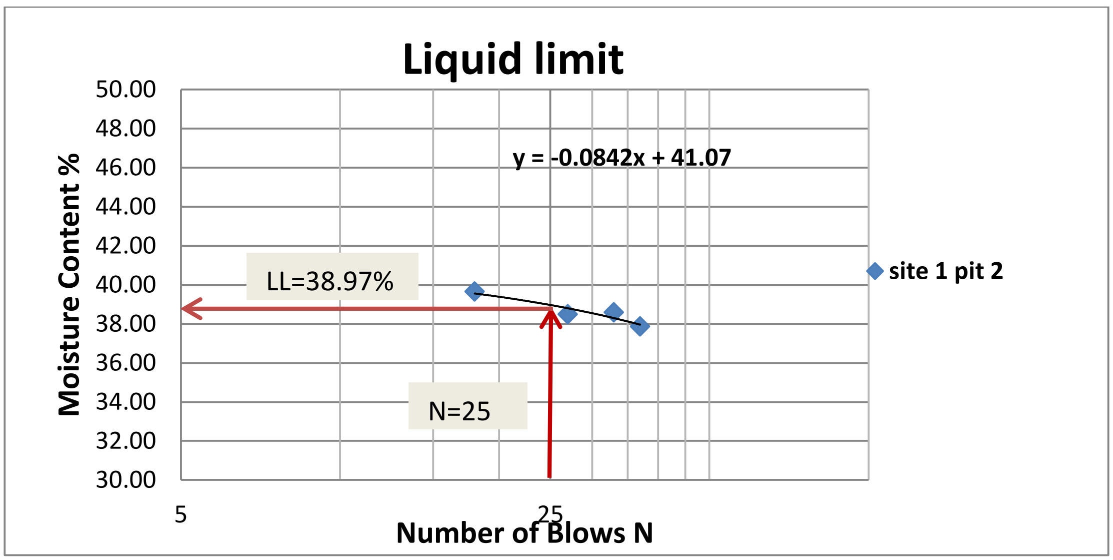

Figure A. 3 Liquid limit Graph for site1 pit 2(TP1-2)

Final Results: LL=38.97%

PL=28.97%

PI=LL-PL=10.41%

5.3 Liquid Limit and Plastic Limit Test For Site 1 Pit 3(TP1-3)

Table A. 17 Data Sheet for Liquid Limit Test Site 1 pit -3 (TP1-3)

`Can No	A17	A20	A28	A38
Can Wt(gm) ,M1	18.07	17.44	11.59	11.22
Can + Wet soil (gm),M2	43.13	55.17	40.4	43.87
Can + Dry soil (gm) ,M3	35.67	44.12	32.1	34.56
M_2-M_3	7.48	11..05	8.3	9.31
M_3-M_1	17.6	26.68	20.51	23.34
No Blow ,N	17	20	28	38
Moisture Content, W (%)	42.5	41.42	40.47	39.89
From the flow curve at no blow (N)=25 ,LL	LL=41.15%			

Table A. 18 Data Sheet for Plastic Limit Test Site 1 pit -3 (TP1-3)

`Can No	H4	A3
Can Wt(gm) ,M1	17.84	18.75
Can + Wet soil (gm),M2	41.05	41.36
Can + Dry soil (gm) ,M3	35.6	36.07
M_2-M_3	5.45	5.29
M_3-M_1	17.76	17.32
Moisture Content, W (%)	30.69	30.54
Average Moisture Content, W (%)	30.61	
Plastic Limit ,PL	PL=30.61%	

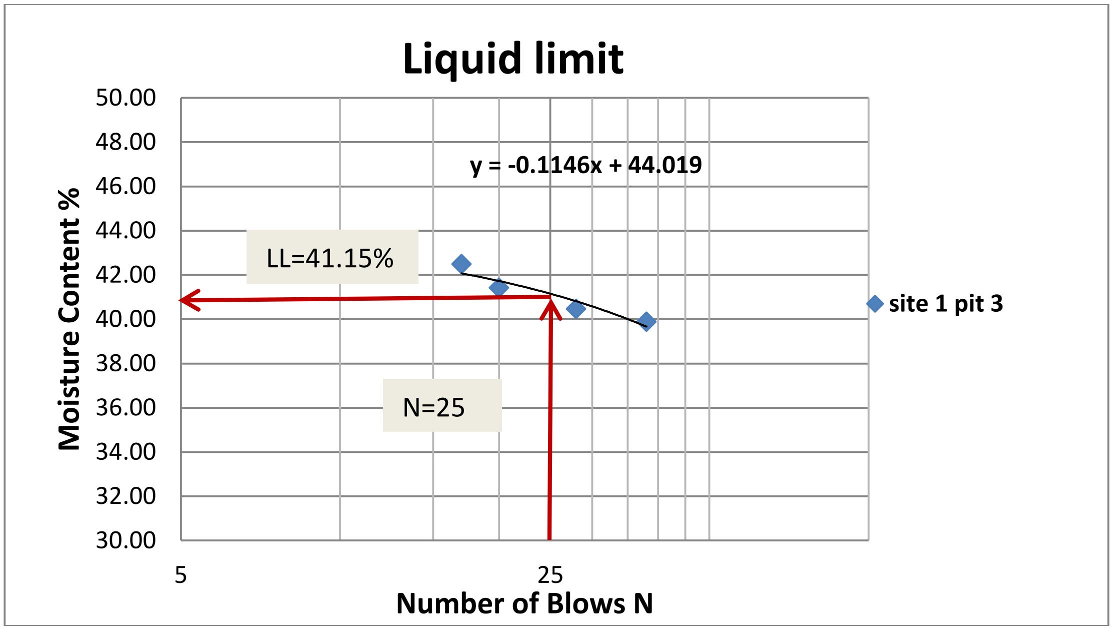

Figure A. 4 Liquid limit Graph for site1 pit 3(TP1-3)

Final Results: LL=41.15%

PL=30.61%

PI=LL-PL=10.54%

5.4 Liquid Limit And Plastic Limit Test For Site 2 Pit 1(TP2-1)

Table A. 19 Data Sheet for Liquid Limit Test Site 2 pit -1 (TP2-1)

`Can No	1C	Y	NS	A2
Can Wt(gm) ,M1	11.04	12.07	19.13	18.27
Can + Wet soil (gm),M2	38.04	40.43	44.22	48.39
Can + Dry soil (gm) ,M3	32.02	33.91	38.7	41.61
M_2-M_3	6.02	6.52	5.52	6.78
M_3-M_1	20.98	21.84	19.57	23.34
No Blow ,N	15	25	35	38
Moisture Content, W (%)	28.69	29.85	28.21	29.05
From the flow curve at no blow (N)=25 ,LL	LL=28.99%			

Table A. 20 Data Sheet for Plastic Limit Test Site 2 pit -1 (TP2-1)

`Can No	M2	Y12
Can Wt(gm) ,M1	11.76	18.06
Can + Wet soil (gm),M2	49.03	54.9
Can + Dry soil (gm) ,M3	41.09	48.36
M_2-M_3	7.94	6.54
M_3-M_1	29.33	30.3
Moisture Content, W (%)	27.07	21.58
Average Moisture Content, W (%)	24.38	
Plastic Limit ,PL	PL=24.38%	

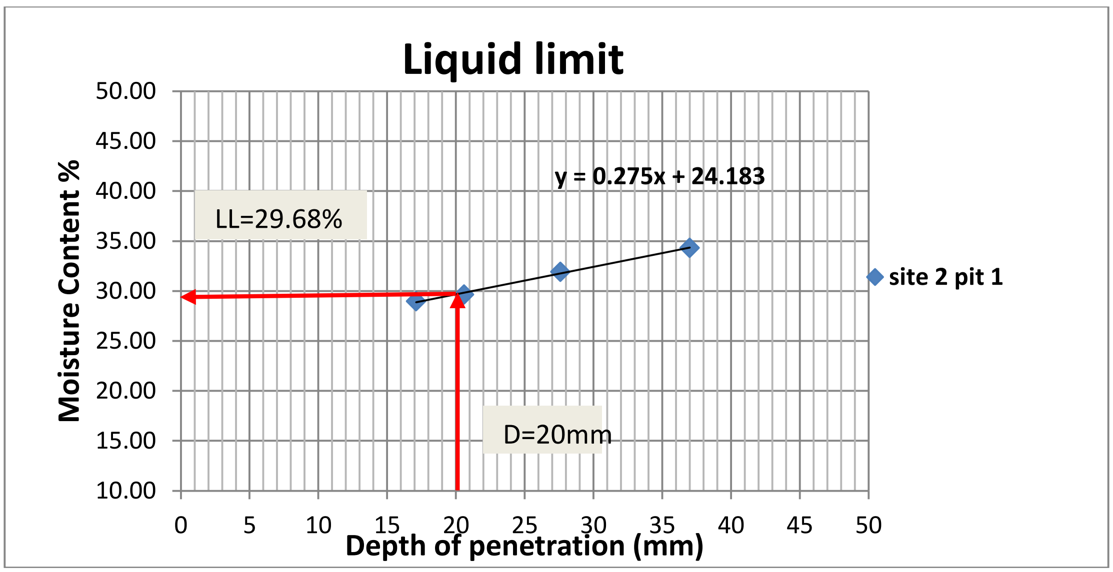

Figure A. 5 Liquid limit Graph for site2 pit 1(TP2-1)

Final Results: LL=28.99%

PL=24.23%

PI=LL-PL=4.6%

5.5 Liquid Limit and Plastic Limit Test For Site 2Pit 2(TP2-2)

Table A. 21 Data Sheet for Liquid Limit Test Site 2 pit -2 (TP2-2)

`Can No	ME	H6	AA	T1
Can Wt(gm) ,M1	12.19	17.36	17.29	18.67

Can + Wet soil (gm),M2	47.63	46.28	55.75	66.94
Can + Dry soil (gm) ,M3	39.85	39.51	46.23	54.19
M_2-M_3	7.78	6.77	9.52	12.75
M_3-M_1	27.66	22.15	28.94	35.52
Depth of penetration (mm)	19.5	23.3	30.7	35
Moisture Content, W (%)	28.13	30.56	32.9	35.9
From the flow curve Depth of penetration =20 ,LL	LL=28.54%			

Table A. 22 Data Sheet for Plastic Limit Test Site 2 pit -2 (TP2-2)

`Can No	2A	R3
Can Wt(gm) ,M1	11.64	17.55
Can + Wet soil (gm),M2	42.73	46.58
Can + Dry soil (gm) ,M3	37.41	41.67
M_2-M_3	5.32	4.91
M_3-M_1	25.77	24.12
Moisture Content, W (%)	20.64	20.36
Average Moisture Content, W (%)	20.5	
Plastic Limit ,PL	PL=20.50%	

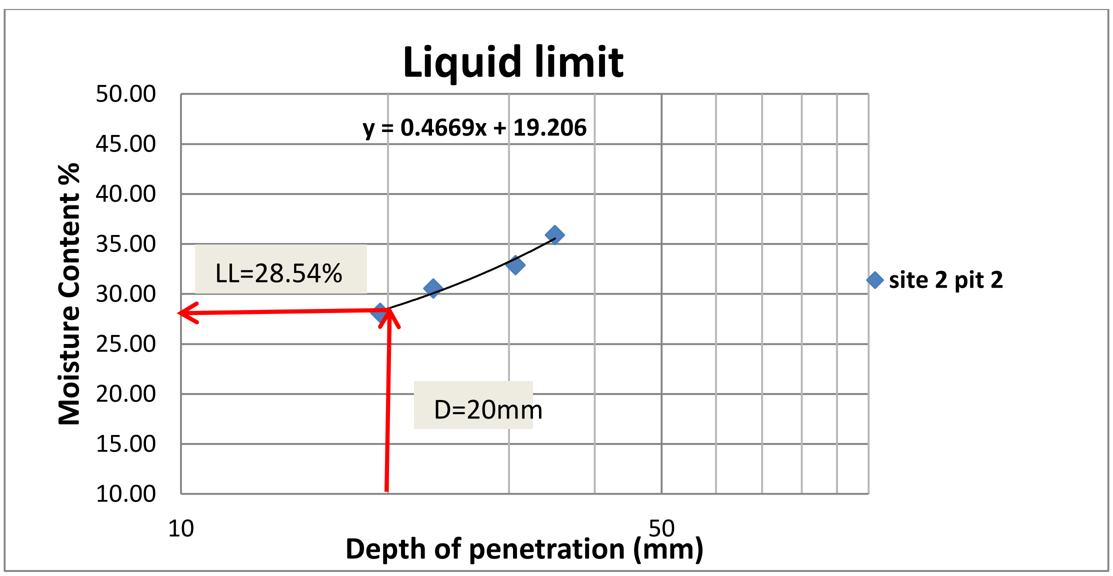

Figure A. 6 Liquid limit Graph for site2 pit 2(TP2-2

Final Results: LL=28.54%

PL=20.50%

PI=LL-PL=8.04%

5.6 Liquid Limit and Plastic Limit Test For Site 2Pit 3(TP2-3)

Table A. 23 Data Sheet for Liquid Limit Test Site 2 pit -3 (TP2-3)

`Can No	A1	MK	T27	ML
Can Wt(gm) ,M1	12.11	18.35	19.29	18.01
Can + Wet soil (gm),M2	45.12	45.3	52.8	60.19
Can + Dry soil (gm) ,M3	38.16	39.32	44.79	49.85
M_2-M_3	6.96	5.98	8.01	10.34
M_3-M_1	26.05	20.97	25.5	31.84
Depth of penetration (mm)	19.4	25.5	31.9	37.7
Moisture Content, W (%)	26.72	28.52	31.41	32.47
From the flow curve Depth of penetration =20 ,LL	LL=26.94%			

Table A. 24 Data Sheet for Plastic Limit Test Site 2 pit -3 (TP2-3)

`Can No	AF	AB
Can Wt(gm) ,M1	13.24	17.44
Can + Wet soil (gm),M2	42.05	56.4
Can + Dry soil (gm) ,M3	37.4	50.22
M_2-M_3	4.65	6.18
M_3-M_1	24.16	32.78
Moisture Content, W (%)	19.40	18.85
Average Moisture Content, W (%)	19.13	
Plastic Limit ,PL	PL=19.13%	

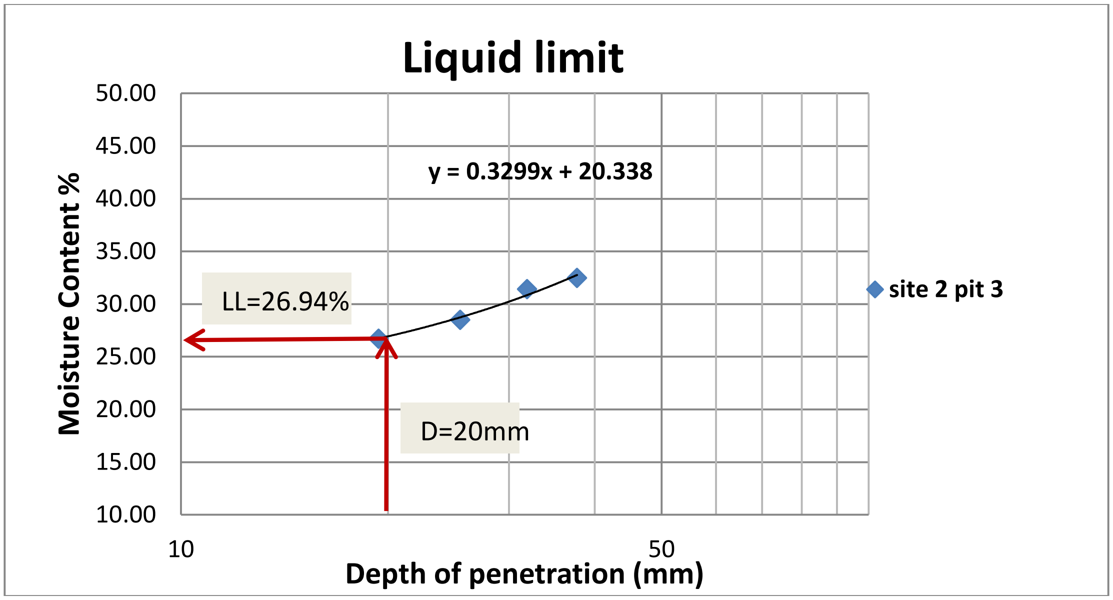

Figure A. 7 Liquid limit Graph for site2 pit 3(TP2-3)

<u>Final Results:</u> LL=26.94%

PL=19.13%

PI=LL-PL=7.81%

6. Direct Shear Test

6.1 Direct Shear Test for Site 1 Pit 1(Tp1-1)

Date Tested: 30/11/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP1-1, TP1-2 and TP1-3

Sample depth = 1.5m-2m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: Poorly-graded gravel with clay and sand (GP-GC) for TP1-1

Table A. 25 Sheet for Direct Shear Test for Site 1 pit 1 (TP1-1)

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a (Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0	0.00	0	0	0.00	0.00	0	0.000
20	0.2	0.00282	69.69	7	26.85	139.37	6	23.02	209.06	8	30.69
40	0.4	0.00280	69.99	8	30.82	139.97	9	34.67	209.96	11	42.38
60	0.6	0.00279	70.29	10	38.69	140.57	10	38.69	210.86	14	54.17
80	0.8	0.00278	70.59	12	46.63	141.18	12	46.63	211.77	15	58.29
100	1	0.00277	70.90	14	54.64	141.79	13	50.73	212.69	17	66.34
120	1.2	0.00276	71.20	15	58.79	142.41	15	58.79	213.61	18	70.55
140	1.4	0.00274	71.52	13	51.18	143.03	17	66.92	214.55	20	78.73
160	1.6	0.00273	71.83	11	43.49	143.66	18	71.17	215.49	21	83.03
180	1.8	0.00272	72.15			144.29	19	75.46	216.44	22	87.37
200	2	0.00271	72.47			144.93	20	79.78	217.40	24	95.74
220	2.2	0.00270	72.79			145.58	22	88.15	218.37	25	100.17
240	2.4	0.00268	73.11			146.23	21	84.52	219.34	26	104.64
260	2.6	0.00267	73.44			146.88	20	80.85	220.33	27	109.15
280	2.8	0.00266	73.77			147.55			221.32	28	113.71
300	3	0.00265	74.11			148.21			222.32	29	118.30
320	3.2	0.00264	74.44			148.89			223.33	30	122.94
340	3.4	0.00262	74.78			149.57			224.35	31	127.61
360	3.6	0.00261	75.13			150.25			225.38	30	124.06
380	3.8	0.00260	75.47			150.95			226.42	29	120.48
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

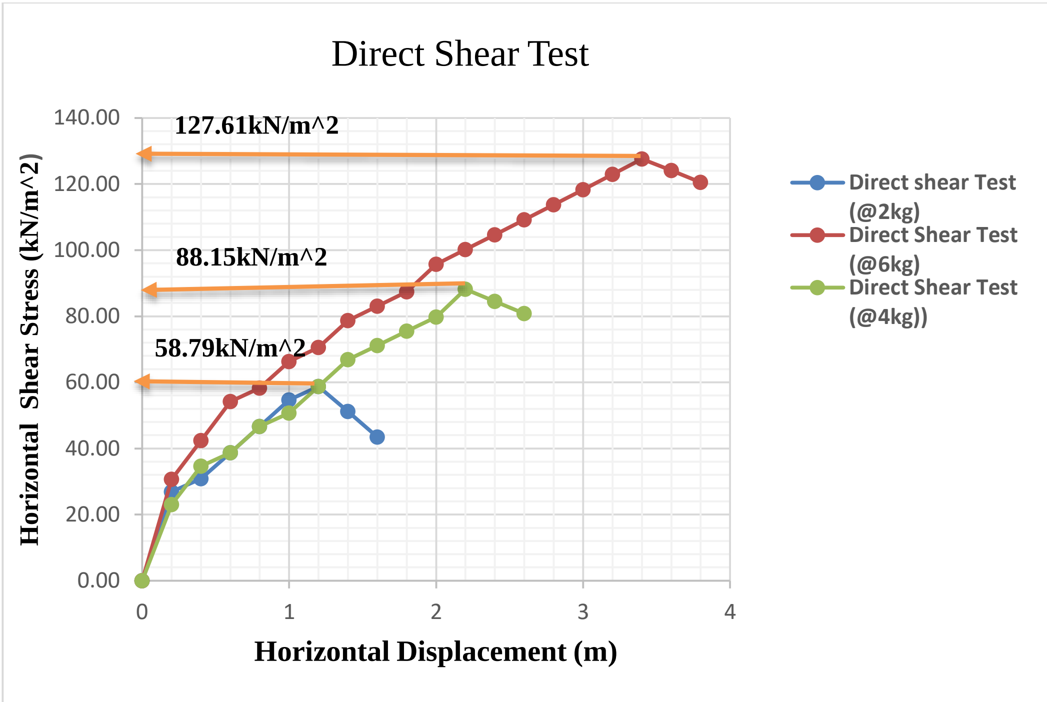

Figure A. 8 Shear Stress versus horizontal displacement graph for site1 pit 1(TP1-1)

Table A. 26 Data Sheet for shear strength for Site 1 pit 1 (TP1-1)

Normal load	Normal stress **(kN/m^2)**	Shear stress**(kN/m^2)**
2kg	71.2	58.79
4kg	145.58	88.15
6kg	224.35	127.61

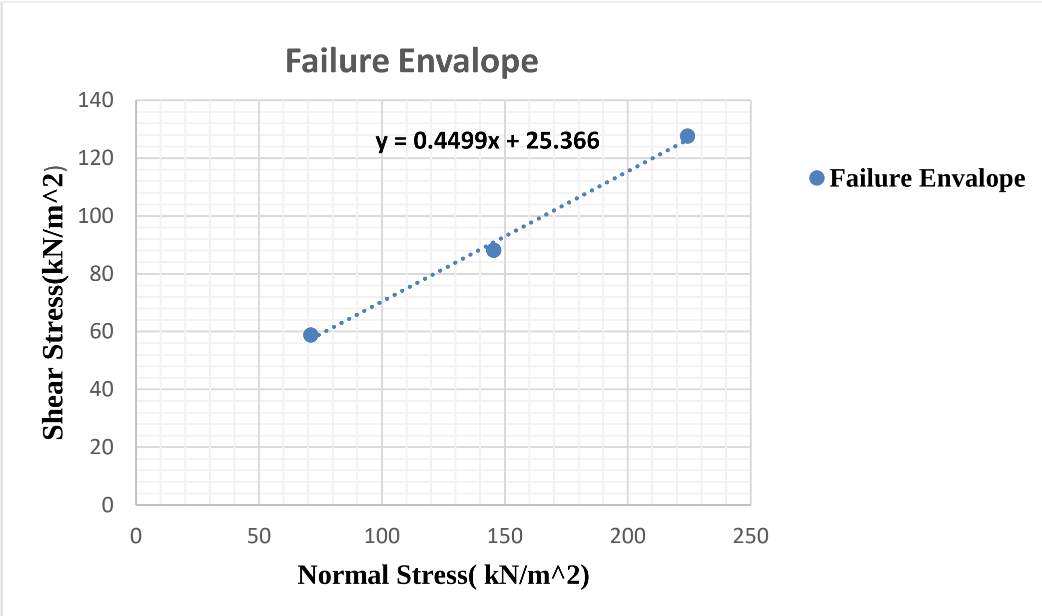

Figure A. 9 Shear Stress versus Normal stress Graph for Site1 Pit 1(TP1-1)

Final Results: C=25.37kpa

ϕ=24.23Degree

6.2 Direct Shear Test for Site 1 Pit 2(TP1-2)

Date Tested: 30/11/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP1-2

Sample depth = 1.5m-2m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: Poorly-graded gravel with clay and sand (GP-GC) for TP1-2

Table A. 27 Data Sheet for Direct Shear Test for Site 1 pit 2 (TP1-2)

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a(Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0	0.00	0	0	0.00	0.00	0	0.000
20	0.2	0.00282	69.69	5	19.18	139.37	8	30.69	209.06	10	38.36
40	0.4	0.00280	69.99	9	34.67	139.97	12	46.23	209.96	14	53.93
60	0.6	0.00279	70.29	13	50.30	140.57	17	65.77	210.86	20	77.38
80	0.8	0.00278	70.59	16	62.17	141.18	20	77.71	211.77	24	93.26
100	1	0.00277	70.90	18	70.25	141.79	23	89.76	212.69	27	105.37
120	1.2	0.00276	71.20	19	74.47	142.41	27	105.83	213.61	30	117.59
140	1.4	0.00274	71.52	19	74.80	143.03	29	114.16	214.55	33	129.91
160	1.6	0.00273	71.83	18	71.17	143.66	30	118.62	215.49	34	134.43
180	1.8	0.00272	72.15		67.51	144.29	30	119.14	216.44	35	139.00
200	2	0.00271	72.47		63.82	144.93	29	115.68	217.40	36	143.60
220	2.2	0.00270	72.79			145.58	28	112.19	218.37	35	140.24
240	2.4	0.00268	73.11			146.23		0.00	219.34	34	136.84
260	2.6	0.00267	73.44			146.88		0.00	220.33		0.00
280	2.8	0.00266	73.77			147.55			221.32		0.00
300	3	0.00265	74.11			148.21			222.32		0.00
320	3.2	0.00264	74.44			148.89			223.33		0.00
340	3.4	0.00262	74.78			149.57			224.35		0.00
360	3.6	0.00261	75.13			150.25			225.38		0.00
380	3.8	0.00260	75.47			150.95			226.42		0.00
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

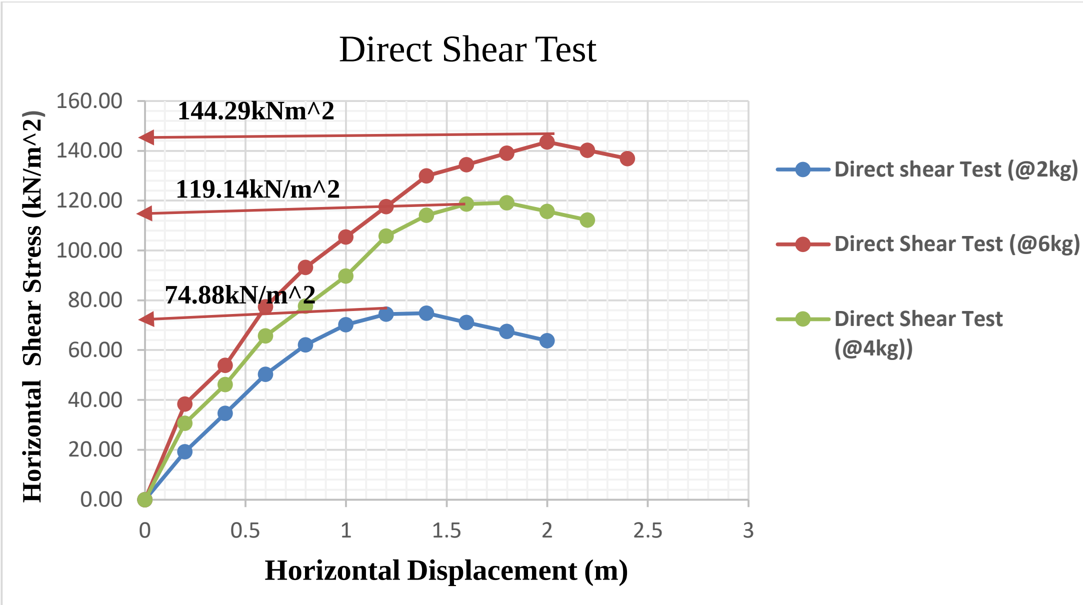

Figure A. 10 Shear Stress versus horizontal displacement graph for Site1 Pit 2(TP1-2)

Table A. 28 Data Sheet for shear strength for Site 1 pit 2 (TP1-2)

Normal load	**Normal stress (kN/m^2)**	**Shear stress(kN/m^2)**
2kg	71.52	74.8
4kg	144.29	119.14
6kg	217.4	143.6

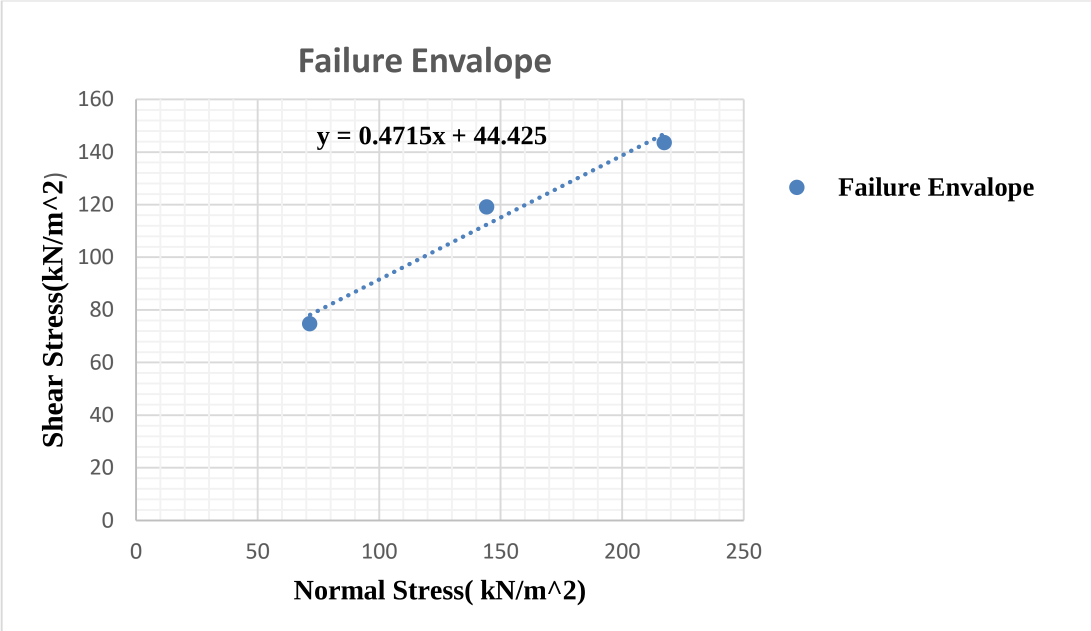

Figure A. 11 Shear Stress versus Normal stress Graph for Site1 Pit 2(TP1-2)

Final Results**:** C=44.425kpa

ϕ=25.244Degree

6.3 Direct Shear Test for Site 1 Pit 2(TP1-2)

Date Tested: 30/11/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP1-3

Sample depth = 1.5m-2m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: well-graded gravel with sand (GW) for TP2-1, well-graded gravel (GW) for TP2-2 and well-graded gravel with sand (GW) for TP2-3

Table A. 29 Data Sheet for Direct Shear Test for Site 1 pit 3 (TP1-3)

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a(Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20	0.2	0.00282	69.69	7	26.85	139.37	8	30.69	209.06	17	65.21
40	0.4	0.00280	69.99	12	46.23	139.97	15	57.79	209.96	21	80.90
60	0.6	0.00279	70.29	16	61.90	140.57	20	77.38	210.86	27	104.46
80	0.8	0.00278	70.59	19	73.83	141.18	25	97.14	211.77	31	120.46
100	1	0.00277	70.90	20	78.05	141.79	28	109.27	212.69	34	132.69
120	1.2	0.00276	71.20	21	82.31	142.41	30	117.59	213.61	37	145.02
140	1.4	0.00274	71.52	22	86.61	143.03	30	118.10	214.55	38	149.59
160	1.6	0.00273	71.83	20	79.08	143.66	28	110.71	215.49	39	154.20
180	1.8	0.00272	72.15		0.00	144.29	26	103.26	216.44	40	158.86
200	2	0.00271	72.47		0.00	144.93	25	99.72	217.40	39	155.57
220	2.2	0.00270	72.79			145.58		0.00	218.37	38	152.26
240	2.4	0.00268	73.11			146.23		0.00	219.34	35	140.86
260	2.6	0.00267	73.44			146.88		0.00	220.33		0.00
280	2.8	0.00266	73.77			147.55			221.32		0.00
300	3	0.00265	74.11			148.21			222.32		0.00
320	3.2	0.00264	74.44			148.89			223.33		0.00
340	3.4	0.00262	74.78			149.57			224.35		0.00
360	3.6	0.00261	75.13			150.25			225.38		0.00
380	3.8	0.00260	75.47			150.95			226.42		0.00
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

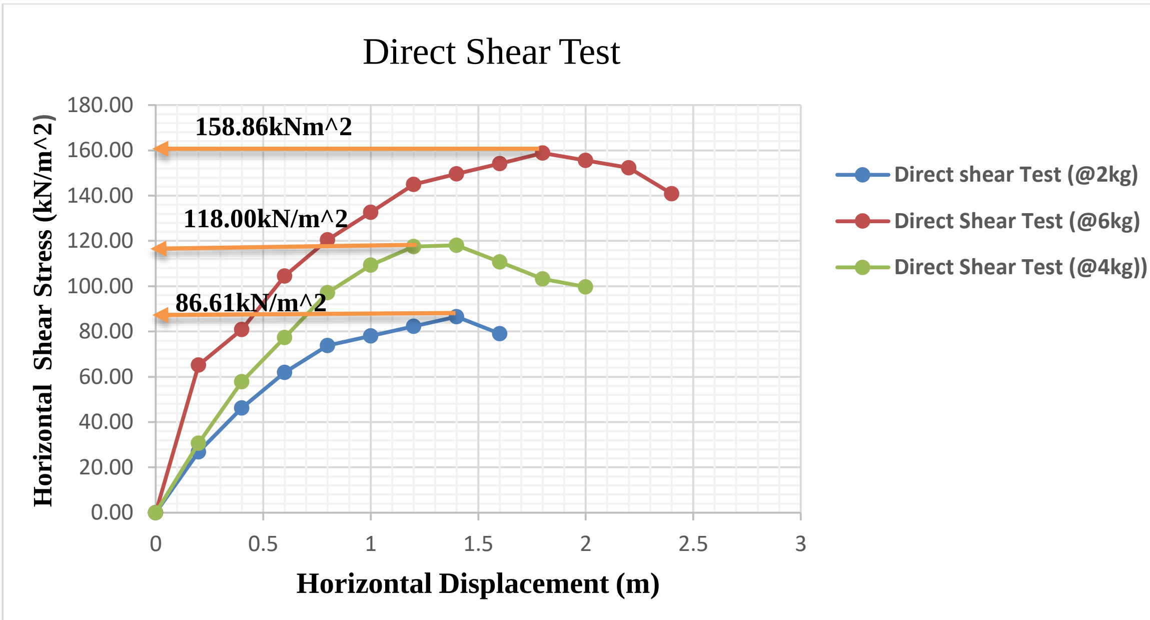

Figure A. 12 Shear Stress versus horizontal displacement graph for site1 pit 3(TP1-3)

Table A. 30 Data Sheet for shear strength for Site 1 pit 3 (TP1-3)

Normal load	Normal stress (kN/m^2)	Shear stress(kN/m^2)
2kg	71.52	86.6
4kg	143.03	118
6kg	216.4	158.86

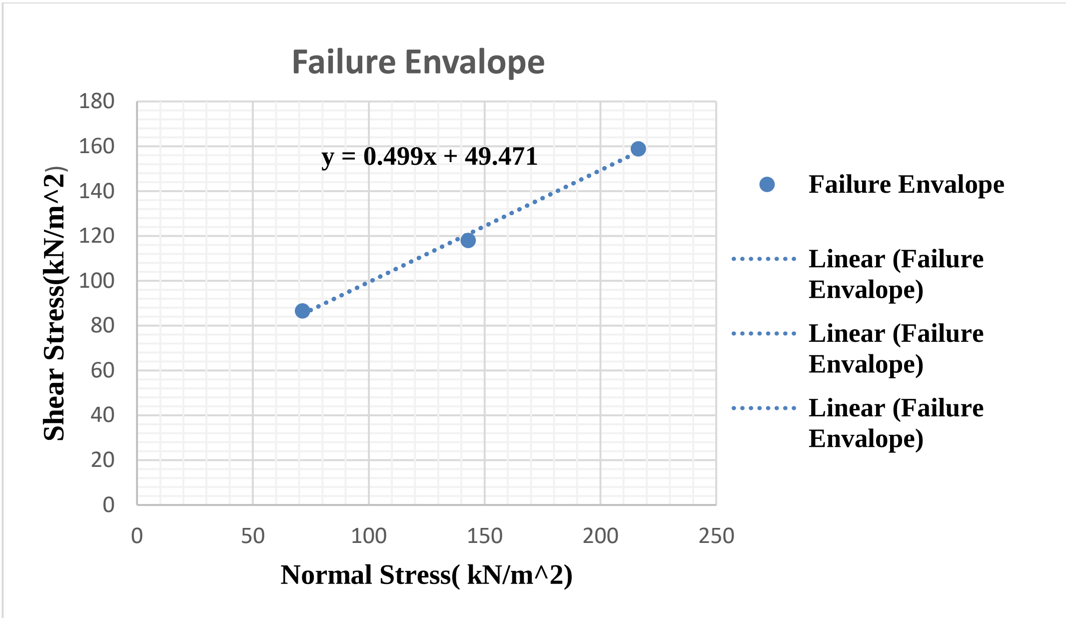

Figure A. 13 Shear Stress versus Normal stress Graph for Site1 Pit 2(TP1-3)

Final Results**:** C=49.471kpa

ϕ=26.519Degree

6.4 Direct Shear Test for Site 2 Pit 1(TP2-1)

Date Tested: 30/11/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP2-1

Sample depth = 2m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: well-graded gravel with sand (GW) for TP2-1

Table A. 31 Data Sheet for Direct Shear Test for Site 2 pit 1 (TP2-1)

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a(Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0	0.00	0	0	0.00	0.00	0	0.000
20	0.2	0.00282	69.69	9	34.52	139.37	8	30.69	209.06	10	38.36
40	0.4	0.00280	69.99	13	50.08	139.97	15	57.79	209.96	17	65.49
60	0.6	0.00279	70.29	18	69.64	140.57	20	77.38	210.86	21	81.25
80	0.8	0.00278	70.59	20	77.71	141.18	22	85.49	211.77	24	93.26
100	1	0.00277	70.90	20	78.05	141.79	23	89.76	212.69	27	105.37
120	1.2	0.00276	71.20	19	74.47	142.41	20	78.39	213.61	29	113.67
140	1.4	0.00274	71.52	18	70.86	143.03	19	74.80	214.55	30	118.10
160	1.6	0.00273	71.83		0.00	143.66	18	71.17	215.49	31	122.57
180	1.8	0.00272	72.15		0.00	144.29		0.00	216.44	30	119.14
200	2	0.00271	72.47		0.00	144.93		0.00	217.40	29	115.68
220	2.2	0.00270	72.79			145.58		0.00	218.37	28	112.19
240	2.4	0.00268	73.11			146.23		0.00	219.34		0.00
260	2.6	0.00267	73.44			146.88		0.00	220.33		0.00
280	2.8	0.00266	73.77			147.55			221.32		0.00
300	3	0.00265	74.11			148.21			222.32		0.00
320	3.2	0.00264	74.44			148.89			223.33		0.00
340	3.4	0.00262	74.78			149.57			224.35		0.00
360	3.6	0.00261	75.13			150.25			225.38		0.00
380	3.8	0.00260	75.47			150.95			226.42		0.00
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

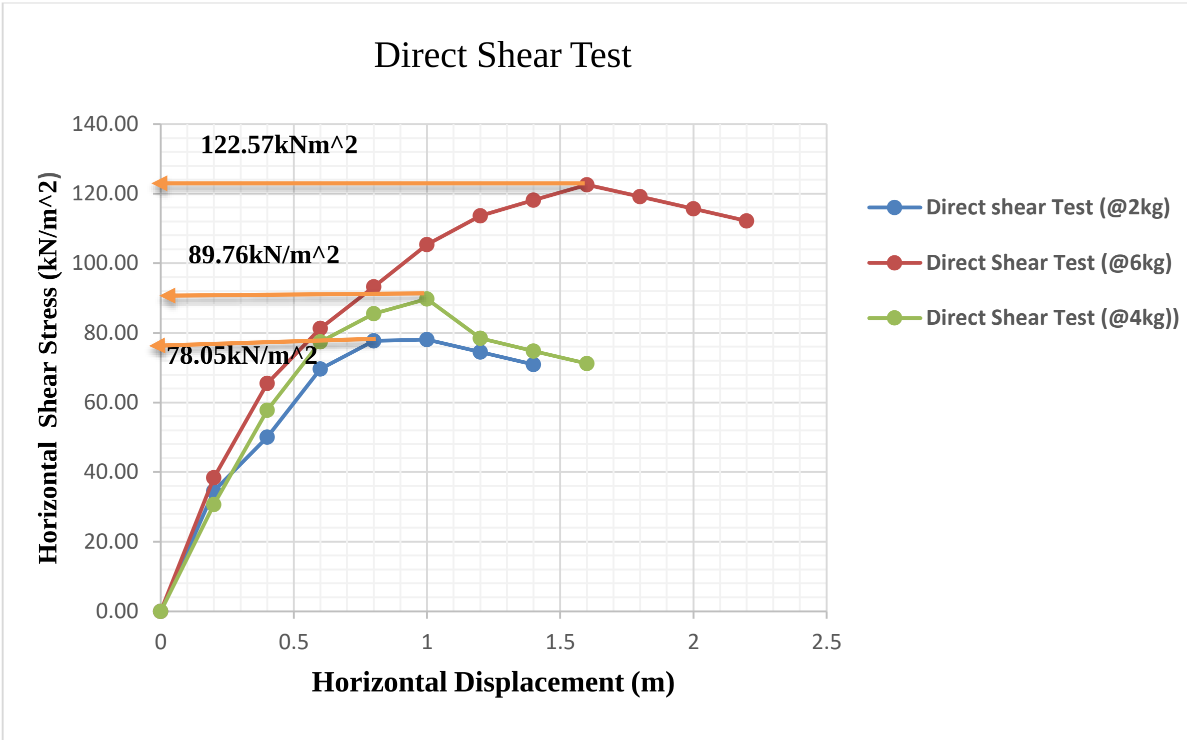

Figure A. 14 Shear Stress versus normal stress graph for site2 pit 1(TP2-1)

Table A. 32 Data Sheet for shear strength for Site 2 pit 2 (TP2-1)

Normal load	**Normal stress (kN/m^2)**	**Shear stress (kN/m^2)**
2kg	70.9	78.05
4kg	141.79	89.78
6kg	215.49	122.57

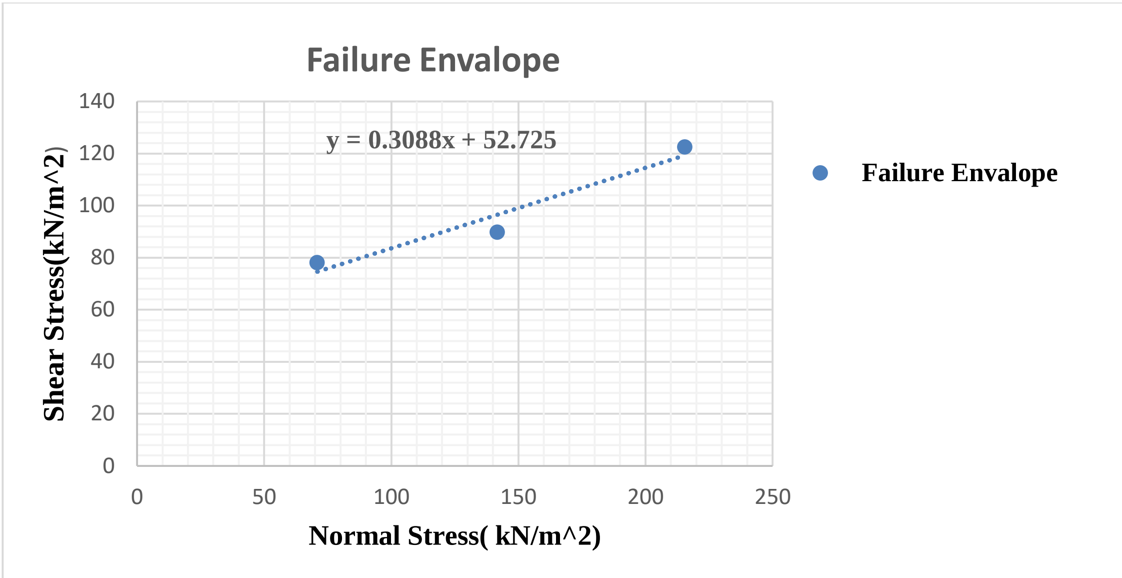

Figure A. 15 Shear Stress versus Normal stress Graph for Site2 Pit 1(TP2-1)

Final Results**:** C=52.73kpa

ϕ=17.16Degree

6.5 Direct Shear Test for Site 2 Pit 2(TP2-2)

Date Tested: 1/12/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP2-1

Sample depth = 2m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: well-graded gravel (GW) for TP2-2

Table A. 33 Data Sheet for Direct Shear Test for Site 2 pit 2 (TP2-2

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a(Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0	0.00	0	0	0.00	0.00	0	0.000
20	0.2	0.00282	69.69	10	38.36	139.37	10	38.36	209.06	10	38.36
40	0.4	0.00280	69.99	17	65.49	139.97	18	69.34	209.96	18	69.34
60	0.6	0.00279	70.29	23	88.99	140.57	24	92.86	210.86	28	108.33
80	0.8	0.00278	70.59	26	101.03	141.18	28	108.80	211.77	34	132.11
100	1	0.00277	70.90	27	105.37	141.79	30	117.08	212.69	40	156.10
120	1.2	0.00276	71.20	25	97.99	142.41	31	121.51	213.61	43	168.54
140	1.4	0.00274	71.52	24	94.48	143.03	29	114.16	214.55	45	177.15
160	1.6	0.00273	71.83	23	90.94	143.66	27	106.76	215.49	46	181.88
180	1.8	0.00272	72.15		0.00	144.29	26	103.26	216.44	45	178.71
200	2	0.00271	72.47		0.00	144.93		0.00	217.40	44	175.52
220	2.2	0.00270	72.79			145.58		0.00	218.37	42	168.28
240	2.4	0.00268	73.11			146.23		0.00	219.34	41	165.01
260	2.6	0.00267	73.44			146.88		0.00	220.33		0.00
280	2.8	0.00266	73.77			147.55			221.32		0.00
300	3	0.00265	74.11			148.21			222.32		0.00
320	3.2	0.00264	74.44			148.89			223.33		0.00
340	3.4	0.00262	74.78			149.57			224.35		0.00
360	3.6	0.00261	75.13			150.25			225.38		0.00
380	3.8	0.00260	75.47			150.95			226.42		0.00
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

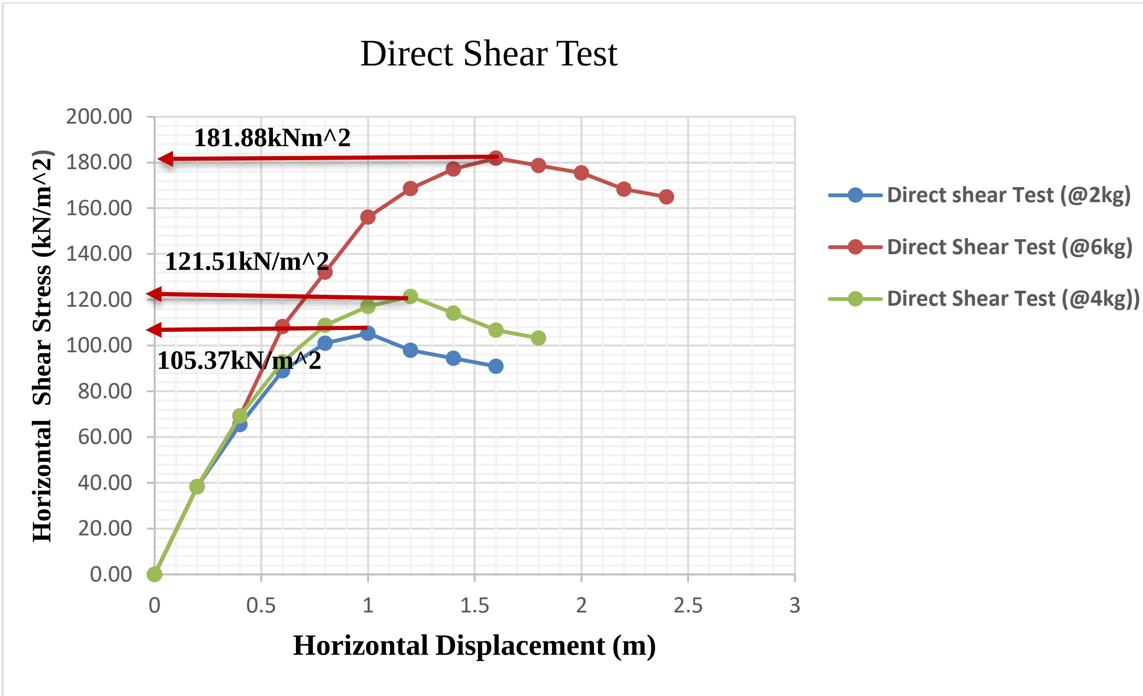

Figure A. 16 Shear Stress versus Normal stress Graph for Site2 Pit 2(TP2-2)

Table A. 34 Data Sheet for shear strength for Site 2 pit 2 (TP2-2)

Normal load	Normal stress (kN/m^2)	Shear stress (kN/m^2)
2kg	70.9	105.37
4kg	142.41	121.51
6kg	215.49	181.88

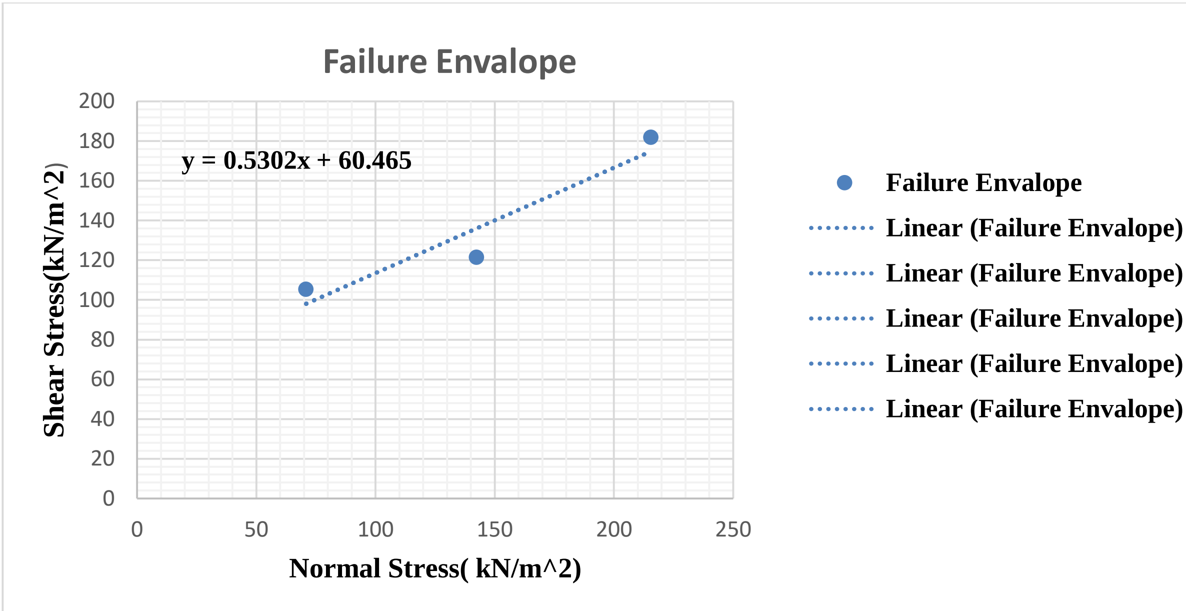

Figure A. 17 Shear Stress versus Normal stress Graph for Site2 Pit 2(TP2-2)

Final Results**:** C=60.405kpa

ϕ=27.933Degree

6.1 Direct Shear Test for Site 2 Pit 3(TP2-3)

Date Tested: 1/12/2011

Project Name: Werie – Maykinatal road

Sample Number (Pit No): TP2-3

Sample depth = 1.5m

Sample Condition: disturbed sample

Ring Calibration Factor=10.81N/mm,

Sample Description: well-graded gravel with sand (GW) for TP2-3

Table A. 35 Data Sheet for Direct Shear Test for Site 2 pit 3 (TP2-3)

Normal load			2kg			4kg			6kg		
Horizontal Dial Reading	Horizontal Displacement (0.01m)	Area a(Ao) m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2	Normal Stress kN/m^2	Proving Dial Reading	Shear Stress kN/m^2
0	0	0.00283	0.00	0	0.00	0	0	0.00	0.00	0	0.000
20	0.2	0.00282	69.69	10	38.36	139.37	9	34.52	209.06	10	38.36
40	0.4	0.00280	69.99	15	57.79	139.97	15	57.79	209.96	15	57.79
60	0.6	0.00279	70.29	20	77.38	140.57	20	77.38	210.86	22	85.12
80	0.8	0.00278	70.59	24	93.26	141.18	25	97.14	211.77	30	116.57
100	1	0.00277	70.90	25	97.56	141.79	30	117.08	212.69	35	136.59
120	1.2	0.00276	71.20	23	90.15	142.41	33	129.34	213.61	38	148.94
140	1.4	0.00274	71.52	22	86.61	143.03	34	133.85	214.55	39	153.53
160	1.6	0.00273	71.83	20	79.08	143.66	35	138.39	215.49	39	154.20
180	1.8	0.00272	72.15		0.00	144.29	34	135.03	216.44	38	150.91
200	2	0.00271	72.47		0.00	144.93	32	127.65	217.40	37	147.59
220	2.2	0.00270	72.79			145.58	30	120.20	218.37	36	144.24
240	2.4	0.00268	73.11			146.23		0.00	219.34		0.00
260	2.6	0.00267	73.44			146.88		0.00	220.33		0.00
280	2.8	0.00266	73.77			147.55			221.32		0.00
300	3	0.00265	74.11			148.21			222.32		0.00
320	3.2	0.00264	74.44			148.89			223.33		0.00
340	3.4	0.00262	74.78			149.57			224.35		0.00
360	3.6	0.00261	75.13			150.25			225.38		0.00
380	3.8	0.00260	75.47			150.95			226.42		0.00
400	4	0.00259	75.82			151.65					
420	4.2	0.00258	76.18			152.35					
440	4.4	0.00256	76.53			153.06					
460	4.6	0.00255	76.89			153.78					
480	4.8	0.00254	77.25			154.51					
500	5	0.00253	77.62			155.24					

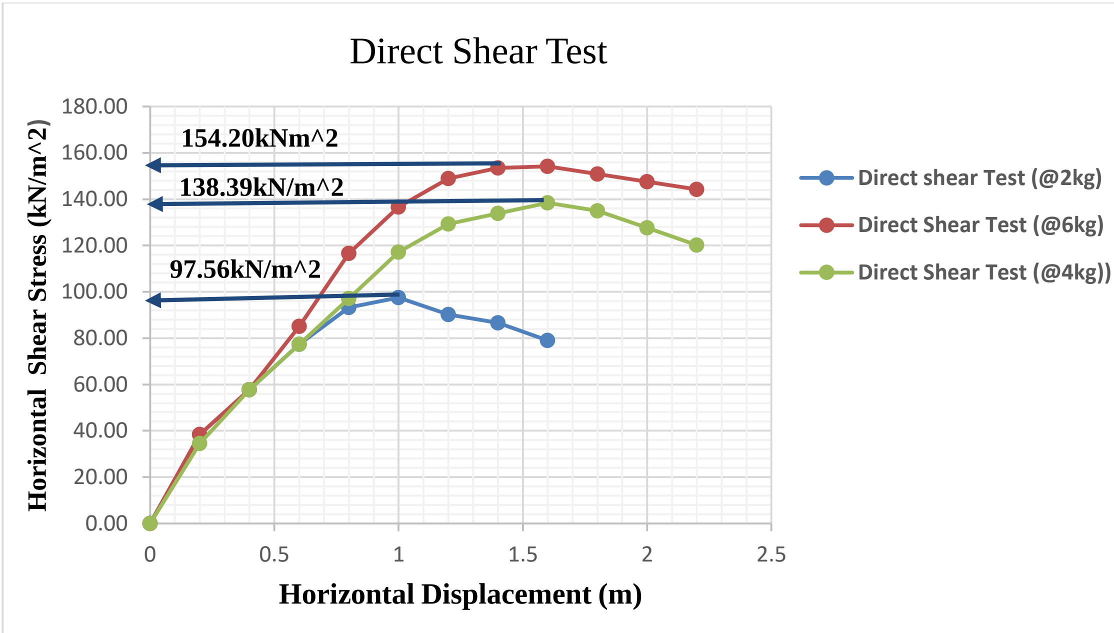

Figure A. 18 Shear Stress versus Normal stress Graph for Site2 Pit 3(TP2-3)

Table A. 36 Data Sheet for shear strength for Site 2 pit 3 (TP2-3)

Normal load	Normal stress (kN/m^2)	Shear stress(kN/m^2)
2kg	70.9	97.56
4kg	143.66	138.39
6kg	215.49	154.49

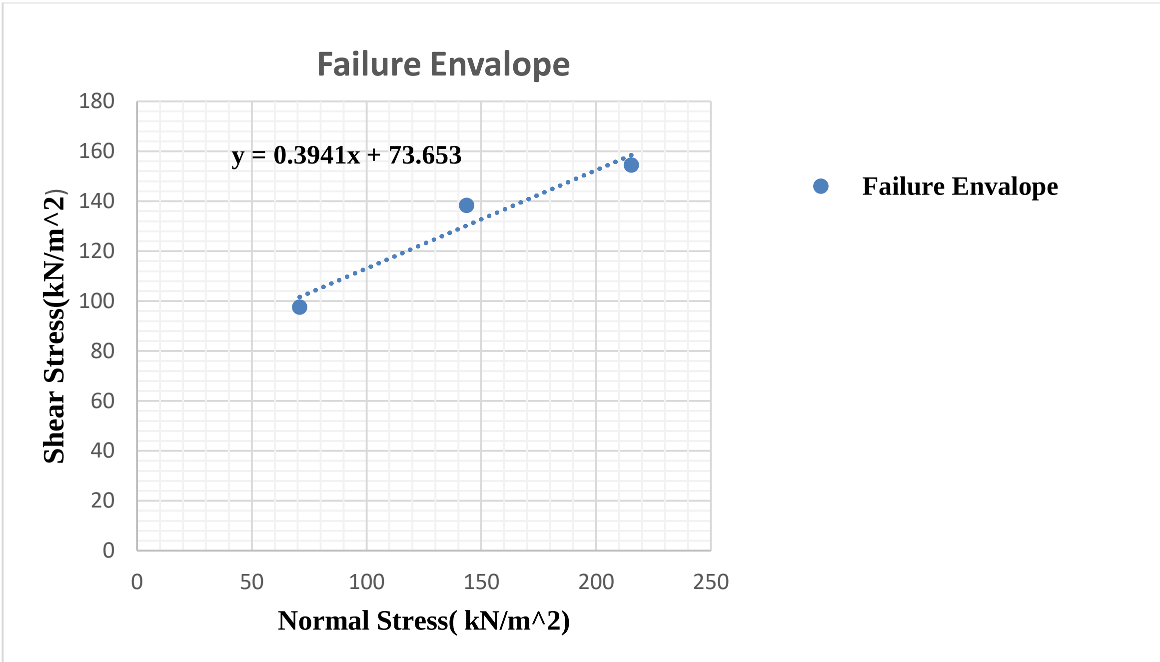

Figure A. 19 Shear Stress versus Normal stress Graph for Site2 Pit 3(TP2-3)

<u>Final Results</u>**:** C=73.653kpa

ϕ=21.509Degree

APPENDIX B: SLOPE ANALYSIS AND RESULTS

1. Geo-Studio 2007, SLOPE/W Analysis Results for Natural Slope of Site1

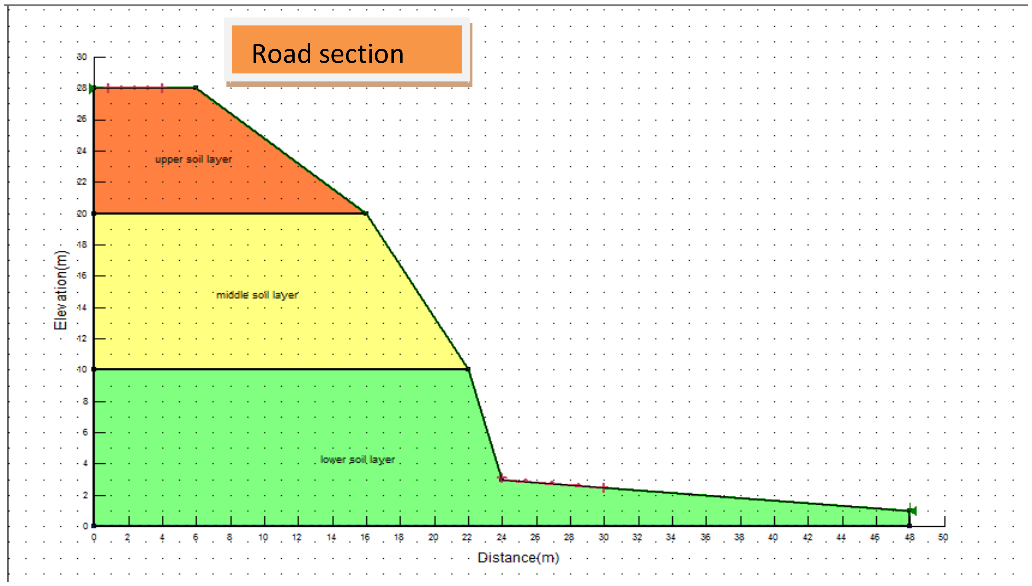

Figure B. 1 Slope profile for natural slope (site-1)

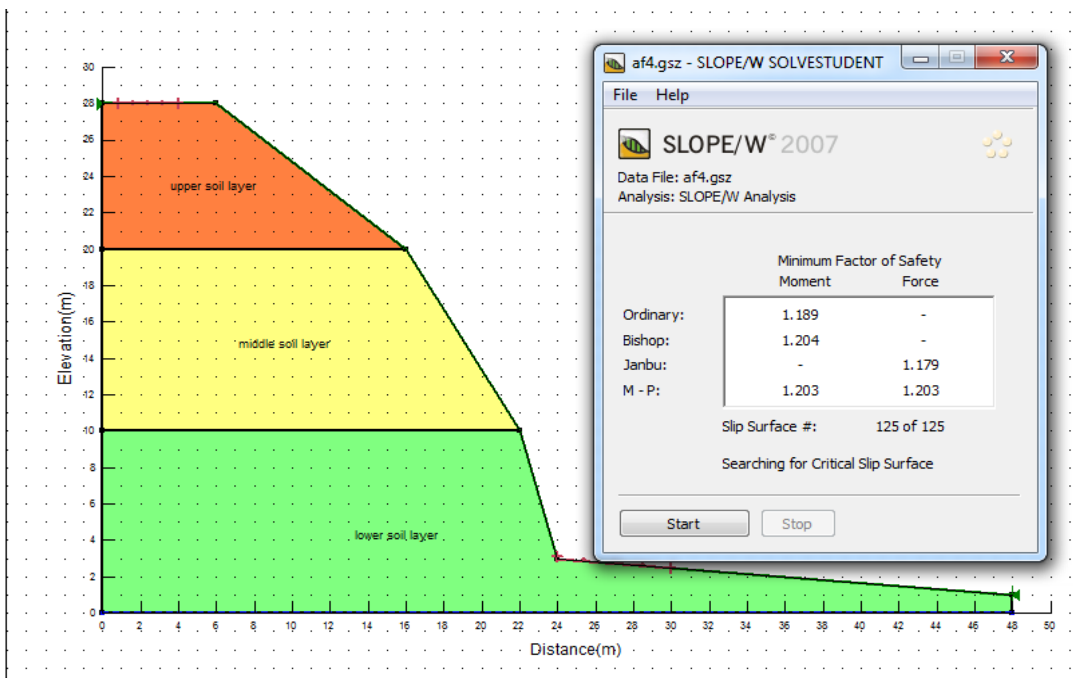

Figure B. 2 Resulting factor of safety for the SLOPE/W analysis (site 1)

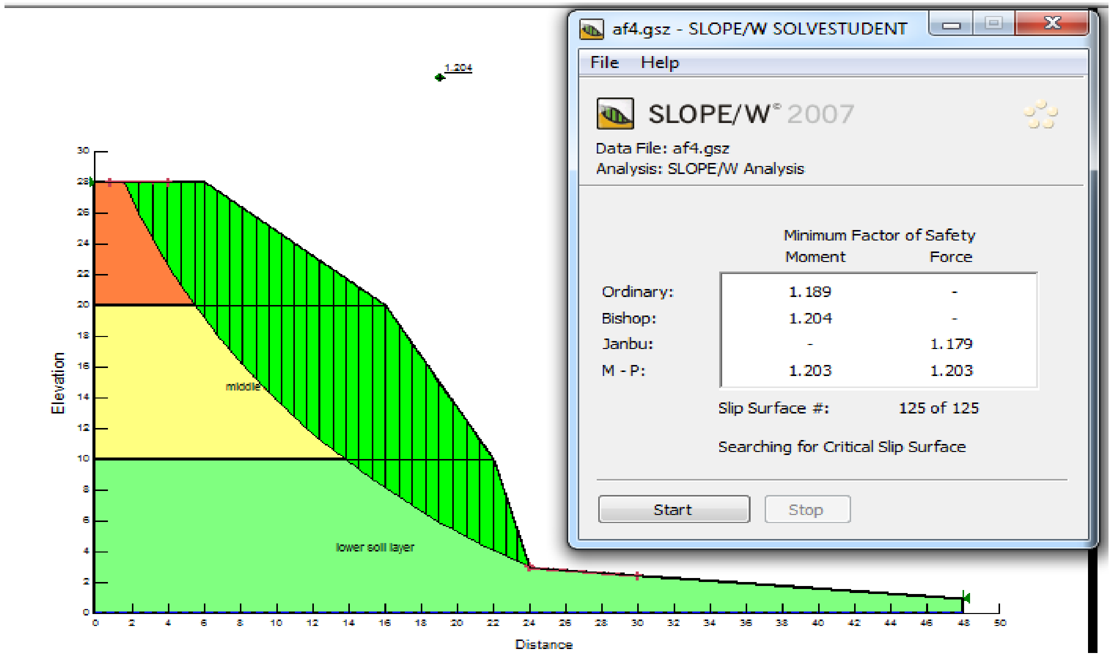

Figure B. 3 Factor of Safety and slip surface at great depth water level (site-1)

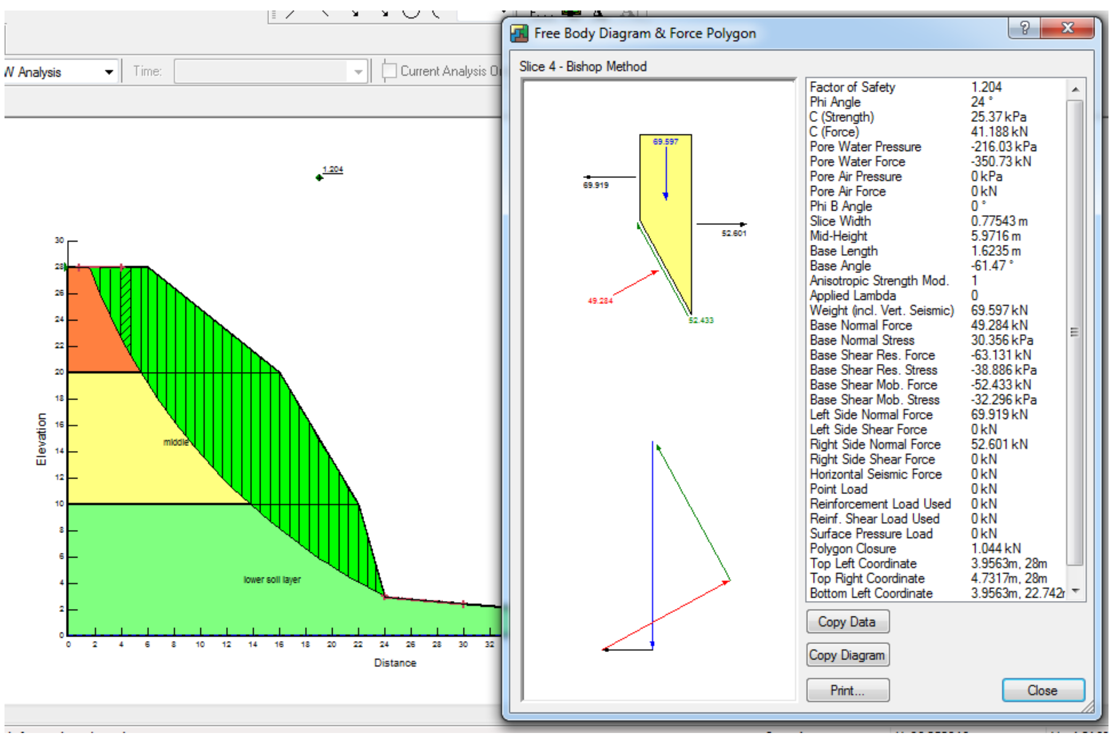

Figure B. 4 Slice information of slope profile at great depth water level (site1)

1. **Slope/W Analysis Repot For Natural Slope of Site1**

Report generated using GeoStudio 2007, version 7.10. Copyright © 1991-2008 GEO-SLOPE International Ltd.

File Information

Revision Number: 2
Date: 1/1/2020
Time: 11:36:33 PM
File Name: af4.gsz
Directory: C:\Users\ash8\Documents\slope site 2\final result site 2\
Last Solved Date: 1/1/2020
Last Solved Time: 11:36:38 PM

Critical Slip Surfaces

	Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
1	28	1.203	(44.13,43.66)	45.295	(1.63, 28)	(23.97,3.098)

Slices of Slip Surface: 28

	Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	28	2.017	27.018	-264.97	-17.496	-7.789	25.37
2	28	2.793	25.170	-246.845	1.62115	0.722	25.37
3	28	3.568	23.522	-230.684	16.7866	7.474	25.37
4	28	4.343	22.028	-216.031	29.3930	13.087	25.37
5	28	5.119	20.657	-202.58	40.2787	17.933	25.37
6	28	5.7535	19.606	-192.279	39.46544	18.605	44.42
7	28	6.3555	18.675	-183.154	46.5883	21.963	44.42
8	28	7.0666	17.632	-172.913	52.82928	24.905	44.42
9	28	7.7776	16.647	-163.259	58.6391	27.6435	44.42
10	28	8.4887	15.715	-154.115	64.17443	30.253	44.42
11	28	9.1998	14.830	-145.448	69.5536	32.789	44.42

12	28	9.9109	13.989	-137.198	74.86738	35.294	44.42
13	28	10.621	13.188	-129.34	80.17722	37.797	44.42
14	28	11.333	12.424	-121.851	85.5186	40.355	44.42
15	28	12.044	11.694	-114.683	90.9072	42.855	44.42
16	28	12.755	10.995	-107.838	96.33958	45.416	44.42
17	28	13.466	10.327	-101.277	101.7961	47.988	44.42
18	28	14.184	9.6800	-94.9324	107.0981	53.444	49.47
19	28	14.910	9.0535	-88.7881	113.281	56.529	49.47
20	28	15.636	8.4531	-82.8996	119.2723	59.5189	49.47
21	28	16.375	7.8685	-77.1662	121.6179	60.689	49.47
22	28	17.125	7.2991	-71.5829	119.8275	59.796	49.47
23	28	17.875	6.7537	-66.2346	116.9856	58.378	49.47
24	28	18.625	6.2314	-61.111	112.9230	56.351	49.47
25	28	19.375	5.7310	-56.2046	107.525	53.657	49.47
26	28	20.125	5.2518	-51.5057	100.6666	50.234	49.47
27	28	20.875	4.7930	-47.0055	92.27890	46.048	49.47
28	28	21.625	4.3539	-42.6989	82.3455	41.09	49.47
29	28	22.328	3.9587	-38.8227	62.21034	31.044	49.47
30	28	22.986	3.6046	-35.3511	31.6663	15.802	49.47
31	28	23.64	3.2644	-32.0140	-0.44486	-0.222	49.47

2. Geo-Studio 2007, SLOPE/W Analysis Results For Natural Slope of Site 2

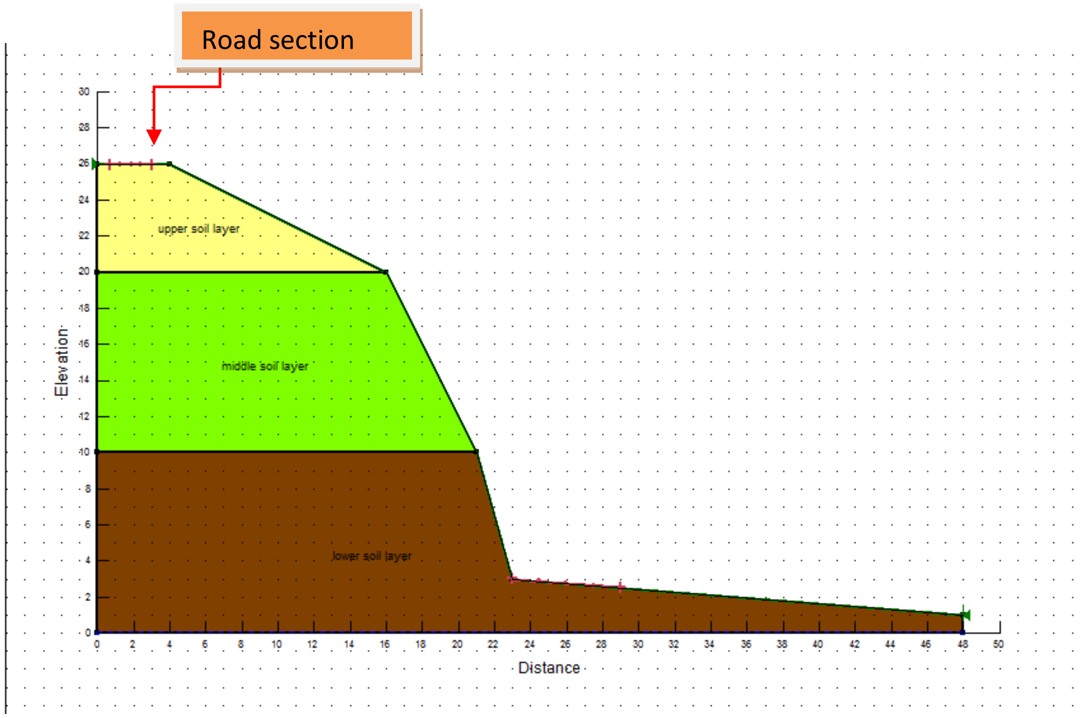

Figure B. 5 Slope profile for natural slope (site-2)

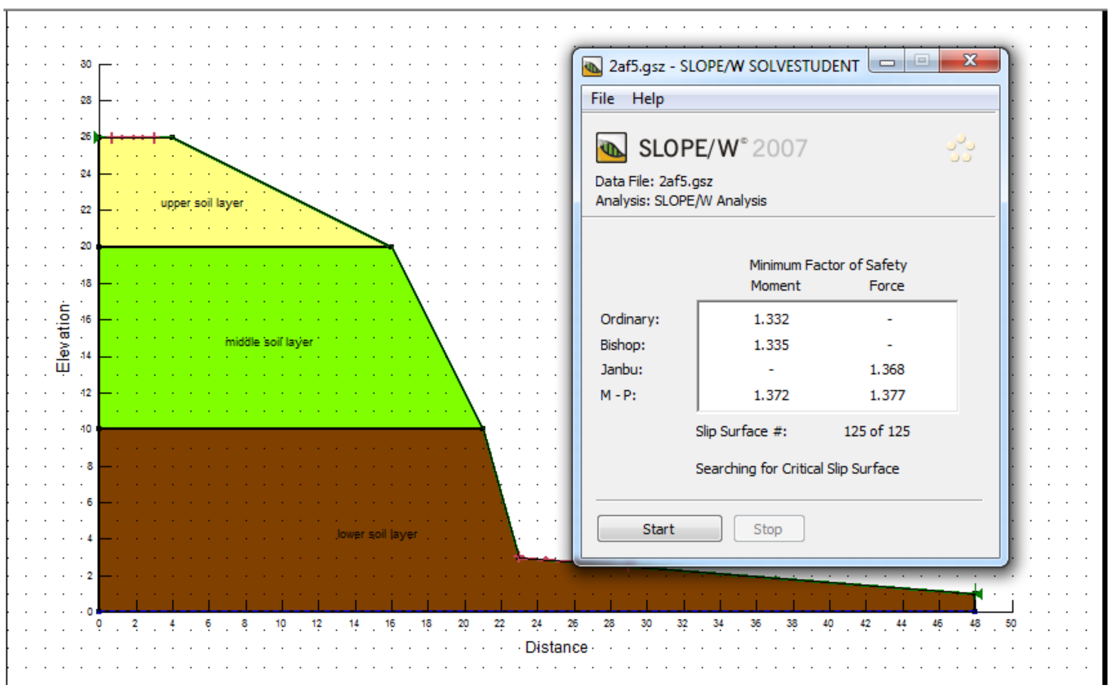

Figure B. 6 Resulting factor of safety for the SLOPE/W analysis (site 2)

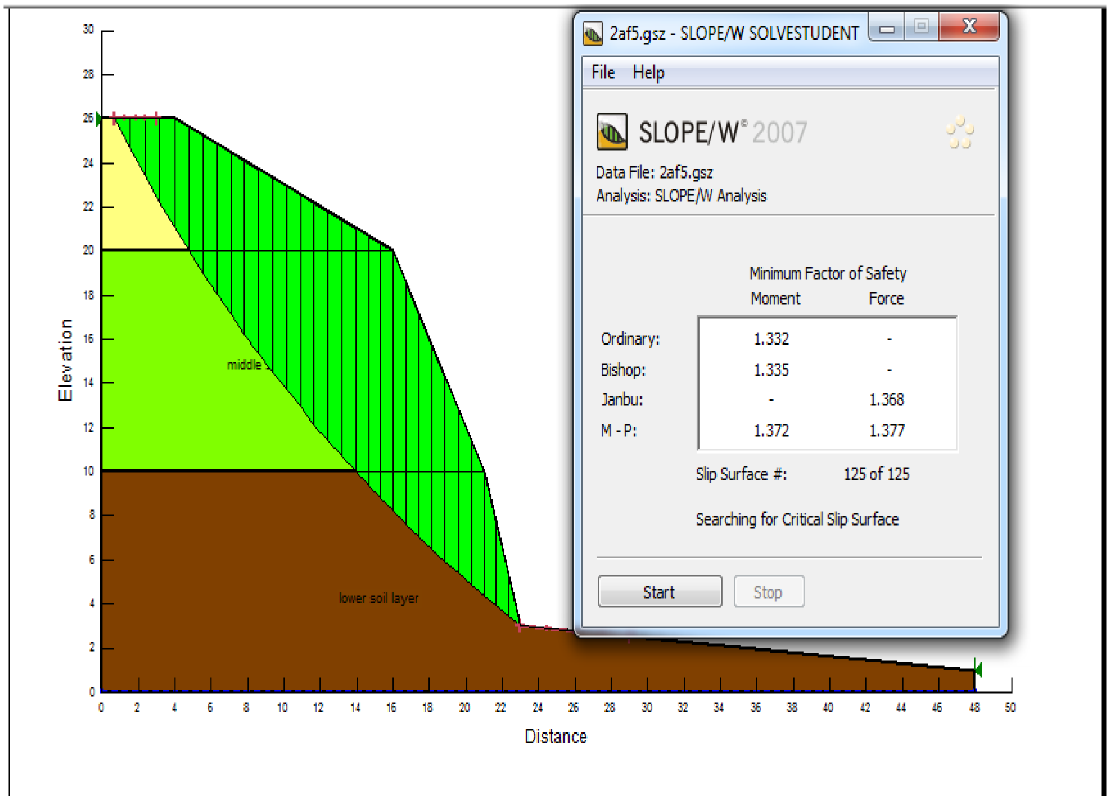

Figure B. 7 Factor of Safety and slip surface at great depth water level (site-2)

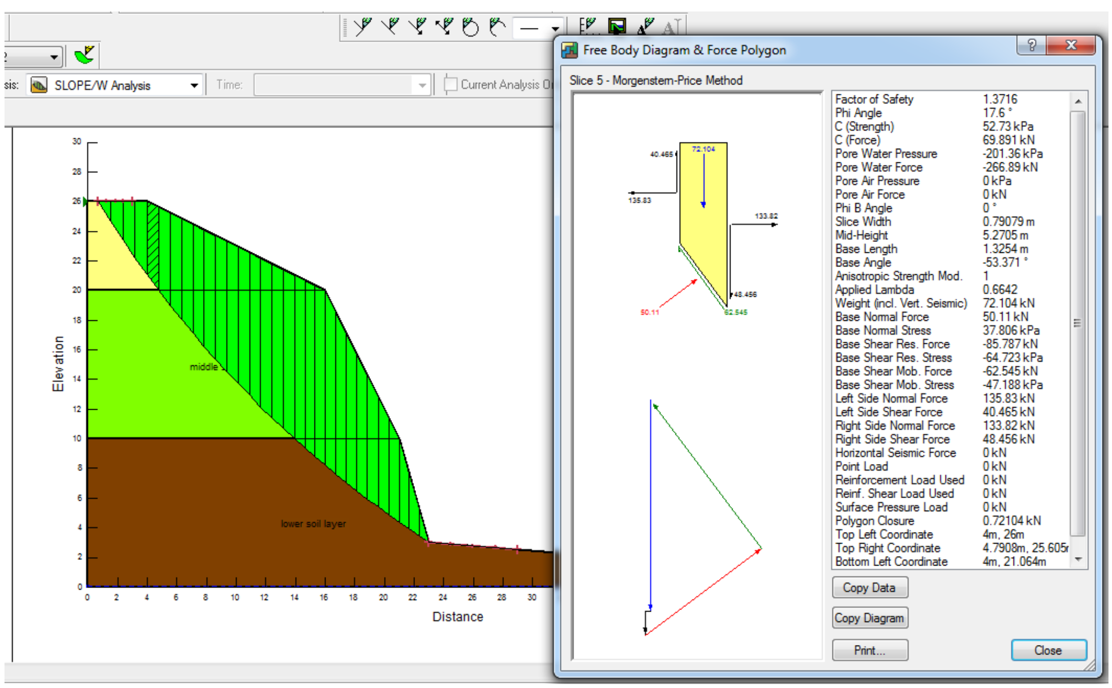

Figure B. 8 Slice information of slope profile at great depth water level (site-2)

3. Slope/W Analysis Repot For Natural Slope of Site2

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File Information

Revision Number: 4

Date: 1/2/2020

Time: 1:46:23 AM

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Directory: C:\Users\ash8\Documents\slope site 2\final result site 2\

Last Solved Date: 1/2/2020

Last Solved Time: 1:48:06 AM

Critical Slip Surfaces

Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
2	1.372	62.68, 63.87	72.63	(0.7, 26)	22.99,3.04

Slices of Slip Surface: 2

	Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	2	1.12	25.34	-248.52	-31.94	-10.13	52.73
2	2	1.94	24.05	-235.8	-8.31	-2.63	52.73
3	2	2.76	22.82	-223.8	10.74	3.40	52.73
4	2	3.59	21.64	-212.2	26.55	8.42	52.73
5	2	4.39	20.53	-201.3	37.81	11.99	52.73
6	2	5.17	19.504	-191.2	40.27	21.35	60.40
7	2	5.94	18.53	-181.7	49.48	26.24	60.40

8	2	6.700	17.591	-172.5	57.96	30.73	60.40
9	2	7.46	16.68	-163.6	65.88	34.92	60.40
10	2	8.23	15.80	-154.9	73.39	38.91	60.40
11	2	8.99	14.95	-146.6	80.65	42.75	60.40
12	2	9.76	14.13	-138.5	87.77	46.53	60.40
13	2	10.519	13.32	-130.6	94.86	50.288	60.40
14	2	11.283	12.54776	-123.0	102.0	54.08	60.40
15	2	12.047	11.79	-115.7	109.28	57.94	60.40
16	2	12.811	11.06	-108.5	116.75	61.89	60.40
17	2	13.575	10.35	-101.5	124.45	65.97	60.40
18	2	14.297	9.69	-95.09	132.74	52.31	73.63
19	2	14.978	9.095	-89.19	140.98	55.56	73.63
20	2	15.659	8.51	-83.45	149.55	58.94	73.63
21	2	16.357	7.92	-77.72	150.76	59.42	73.63
22	2	17.071	7.341	-71.99	143.85	56.69	73.63
23	2	17.785	6.77	-66.41	136.02	53.61	73.63
24	2	18.5	6.22	-60.97	127.08	50.09	73.63
25	2	19.214	5.67	-55.66	116.87	46.06	73.63
26	2	19.928	5.15	-50.50	105.23	41.47	73.63
27	2	20.642	4.64	-45.46	91.989	36.25	73.63
28	2	21.33	4.15	-40.72	67.52	26.61	73.63
29	2	21.99	3.69	-36.68	30.92	12.18	73.63
30	2	22.66	3.25	-31.9	-8.697	-3.43	73.63

APPENDIX C: STABILITY ANALYSIS

4. Geo-Studio 2007, Slope/W Analysis Results For Modified Slope of Site 1

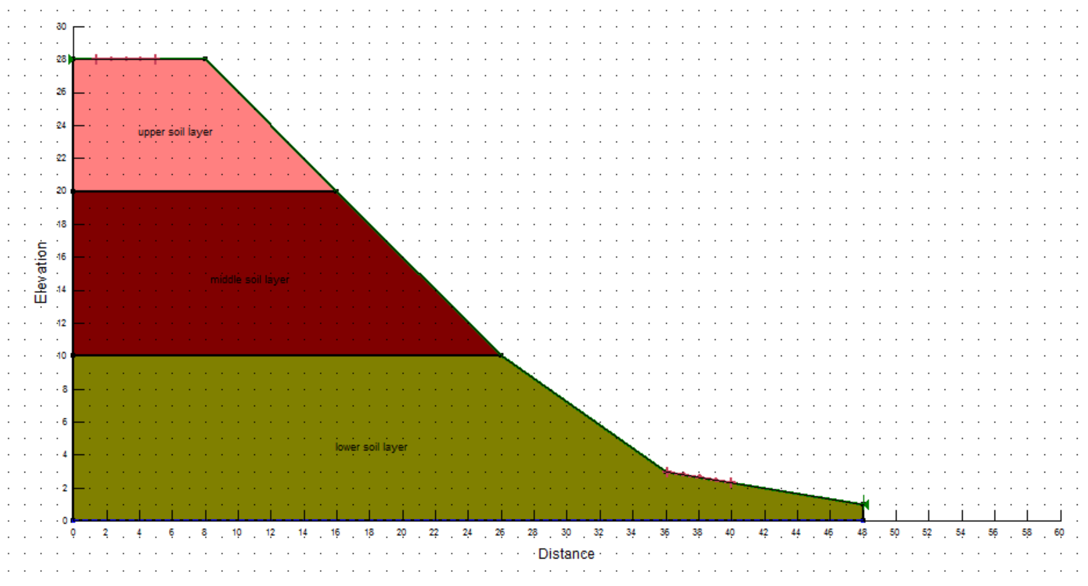

Figure D. 1 Slope profile for modified slope (site-1)

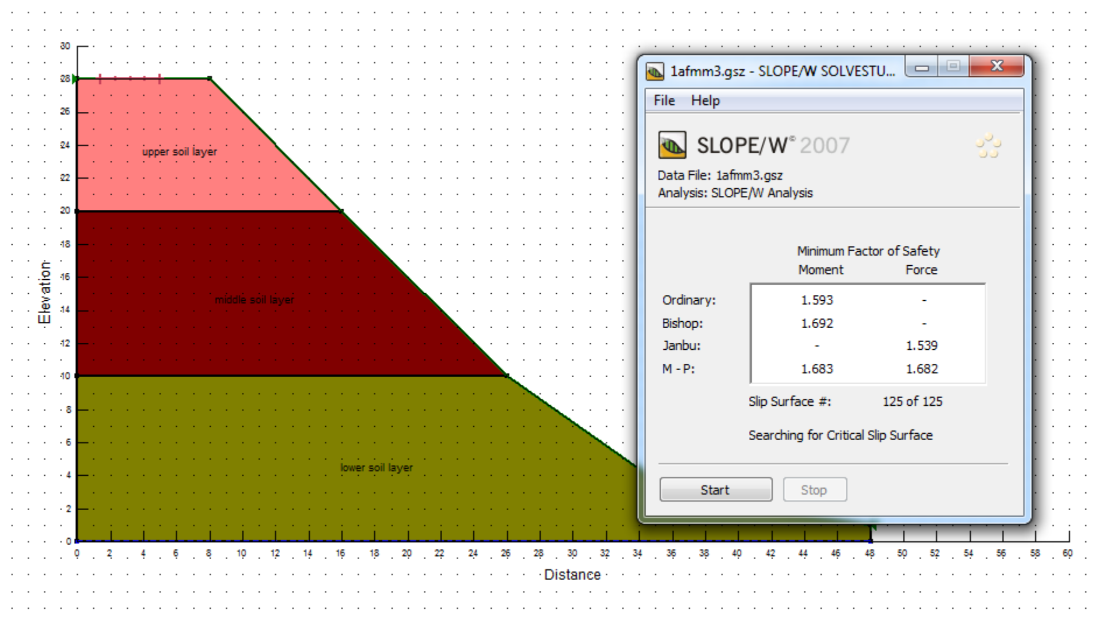

Figure D. 2 Resulting factor of safety for the SLOPE/W analysis (site 1)

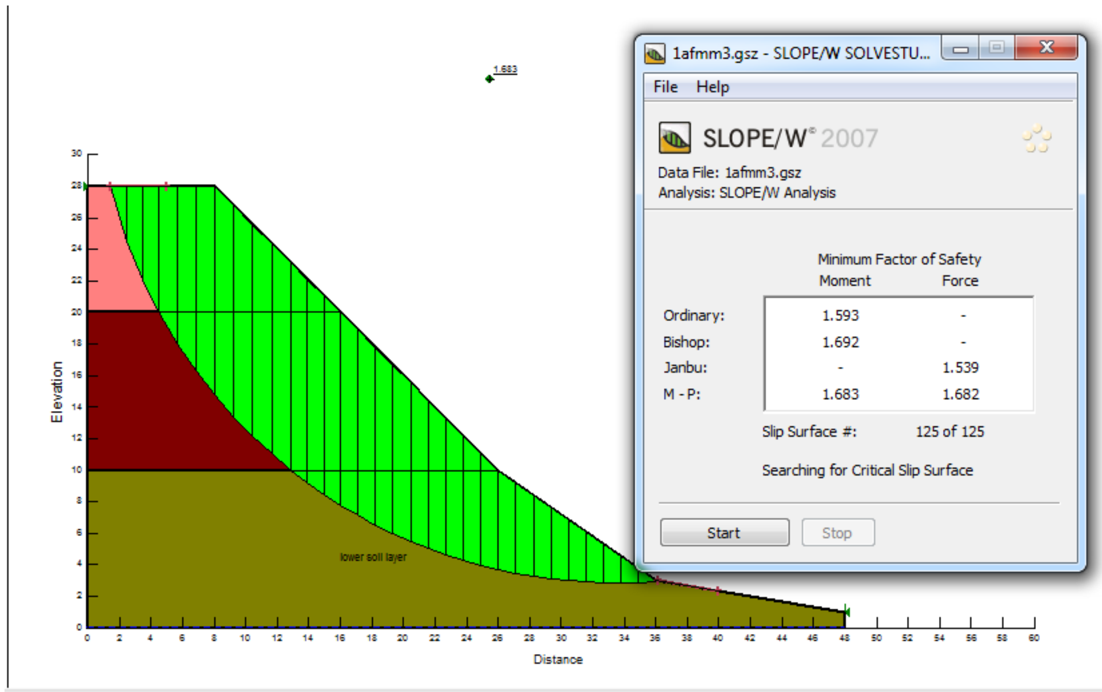

Figure D. 3 Factor of Safety and slip surface at great depth water level (site-1)

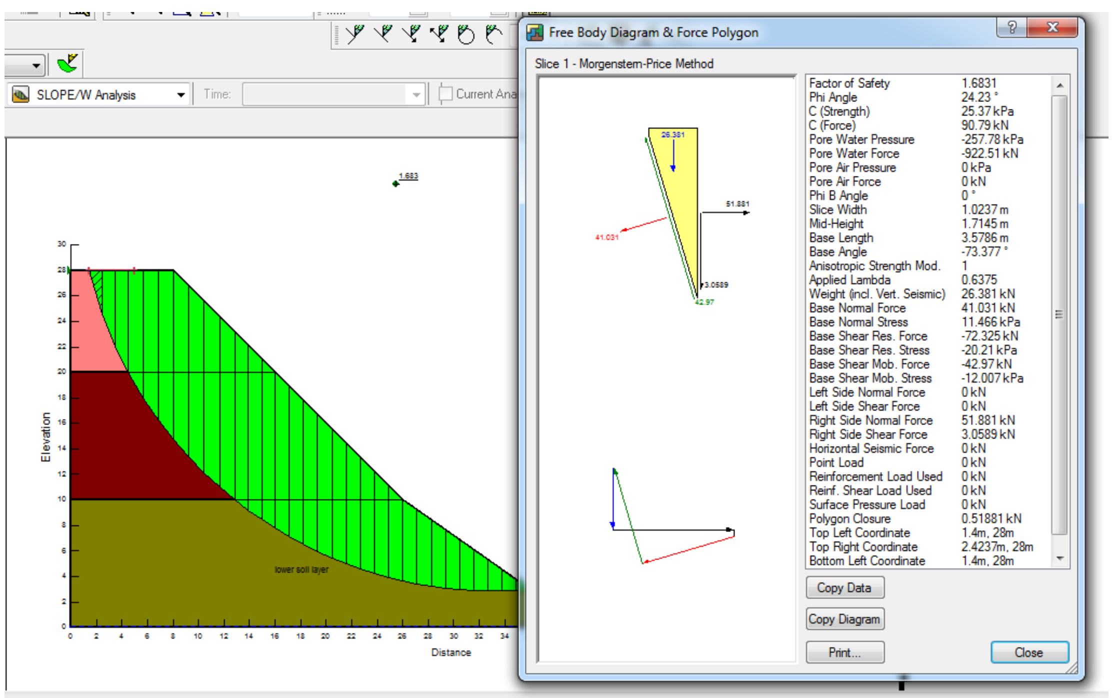

Figure D. 4 Slice information of slope profile at great depth water level (site-1)

5. Slope/W Analysis Report For Modify Slope of Site2

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File Information

Revision Number: 2

Date: 1/3/2020

Time: 10:14:23 PM

File Name: 1afmm3.gsz

Directory: C:\Users\ash8\Documents\slope site 2\final result site 2\

Critical Slip Surfaces

Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
4	1.683	33.29, 35.65	32.798	(1.4, 28)	(36.127, 2.979

Slices of Slip Surface: 4

	Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	4	1.91	26.28	-257.78	-11.46	-5.160	25.37
2	4	2.94	23.32	-228.61	19.92714	8.96816	25.37
3	4	3.96	21.02	-206.20	42.71	19.2212	25.37
4	4	5.06	19.00	-186.36	52.76	24.87	44.42
5	4	6.235	17.145	-168.15	73.67	34.731	44.42
6	4	7.41	15.529	-152.30	92.12	43.43	44.42
7	4	8.60	14.078	-138.07	104.01	49.03	44.42
8	4	9.82	12.76	-125.15	109.81	51.76	44.42
9	4	11.03	11.57	-113.52	115.03	54.23	44.42

10	4	12.25	10.51	-103.04	119.90	56.52	44.42
11	4	13.38	9.599	-94.145	124.108	61.93	49.47
12	4	14.42	8.831	-86.60	129.30	64.525	49.47
13	4	15.47	8.124	-79.671	134.229	66.98	49.47
14	4	16.55	7.4553	-73.114	138.629	69.178	49.47
15	4	17.67	6.82	-66.92	142.34	71.033	49.47
16	4	18.78	6.25	-61.28	145.37	72.547	49.47
17	4	19.88	5.726	-56.15	147.51	73.61	49.47
18	4	21	5.252	-51.51	148.50	74.105371	49.47
19	4	22.11	4.83	-47.334	148.08	73.89	49.47
20	4	23.22	4.45	-43.60	145.95	72.83174	49.47
21	4	24.33	4.108	-40.29	141.82	70.775	49.47
22	4	25.44	3.8140	-37.40	135.473	67.6032	49.47
23	4	26.55	3.5605	-34.91	128.509	64.128	49.47
24	4	27.66	3.3471	-32.82	120.958	60.360	49.47
25	4	28.77	3.1732	-31.12	110.983	55.382	49.47
26	4	29.88	3.0380	-29.79	98.671	49.238	49.47
27	4	31	2.9411	-28.83	84.22881	42.031	49.47
28	4	32.11	2.8821	-28.26	67.9859	33.926	49.47
29	4	33.22	2.8609	-28.05	50.3684	25.1346	49.47
30	4	34.33	2.8774	-28.219	31.85921	15.898	49.47
31	4	35.44	2.9316	-28.750	12.95630	6.4654227	49.47
32	4	36.06	2.9735	-29.161	3.074604	1.5342816	49.47

6. Geo-Studio 2007, Slope/W Analysis Results For Modified Slope of Site 2

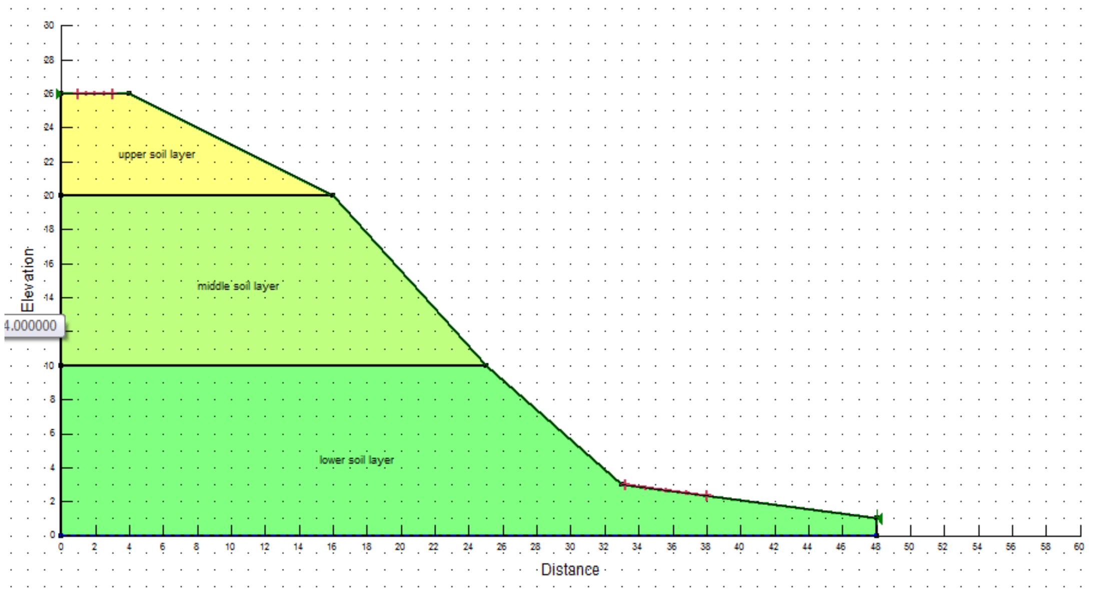

Figure D. 5 Slope profile for modify slope (site-2)

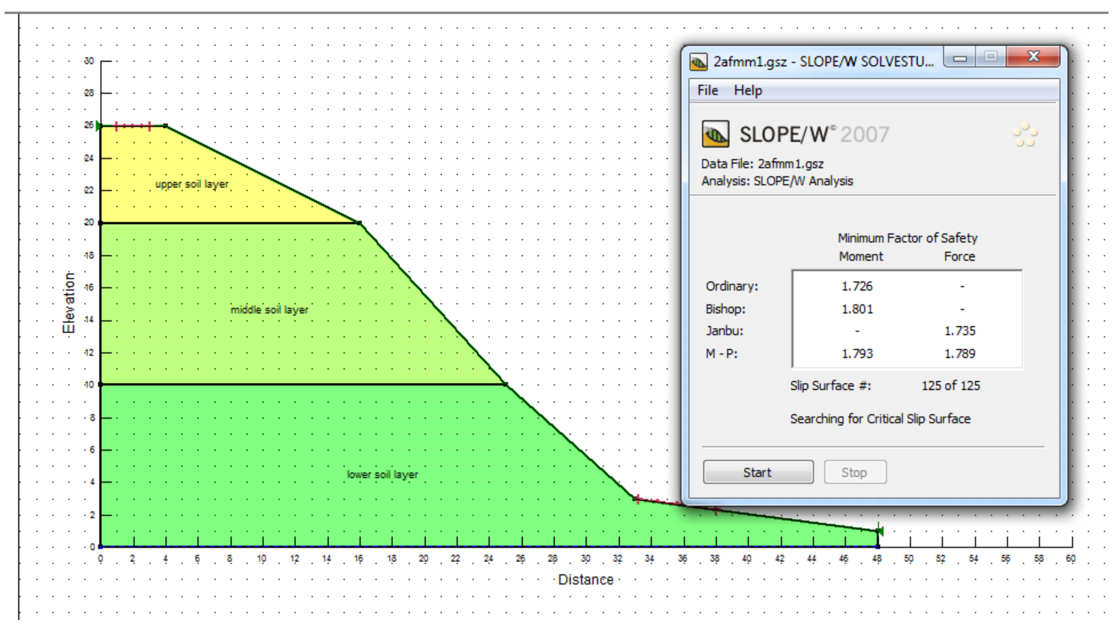

Figure D. 6 Resulting factor of safety for the SLOPE/W analysis (site 2)

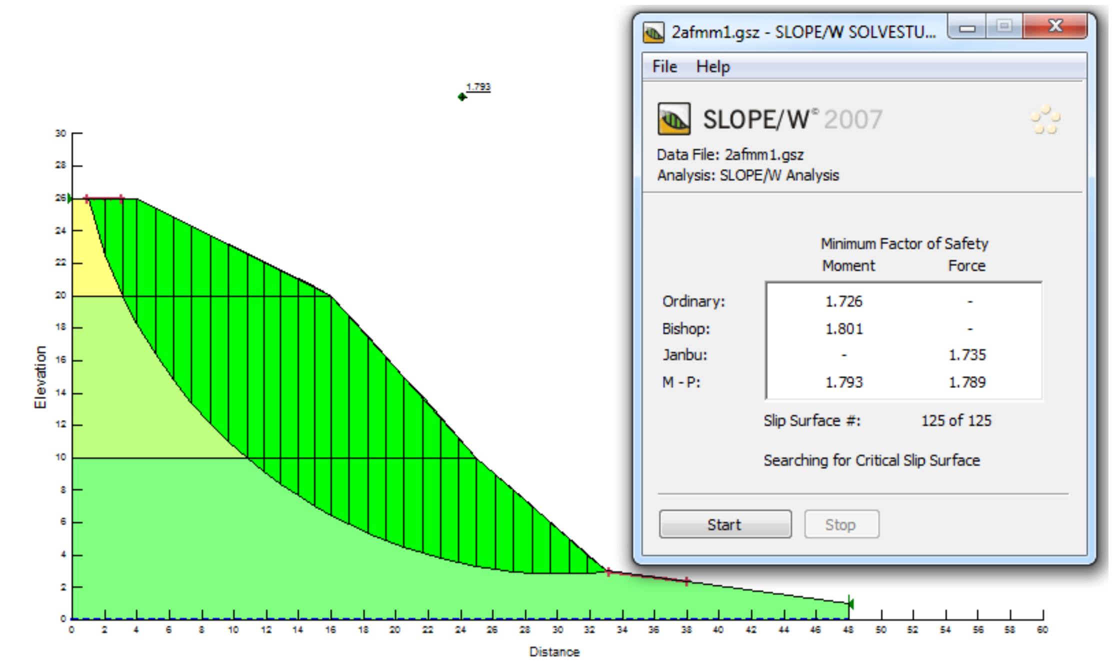

Figure D. 7 Factor of Safety and slip surface at great depth water level (site-2)

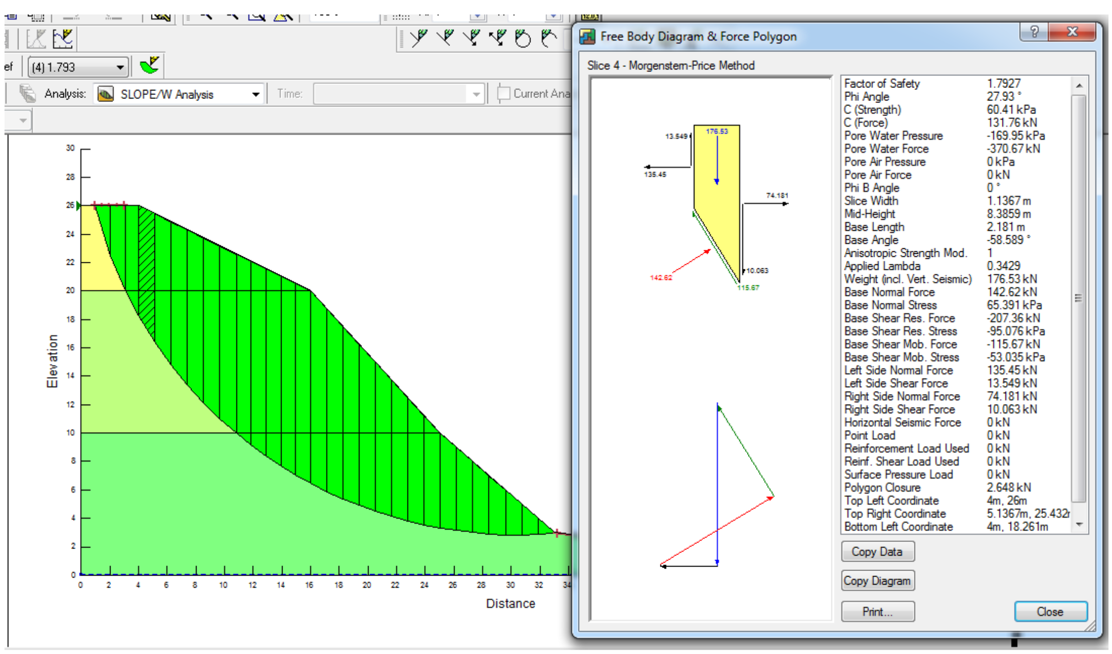

Figure D. 8 Slice information of slope profile at great depth water level (site-2)

7. Slope/W Analysis Report For Modified Slope of Site2

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File Information

Revision Number: 3

Date: 1/3/2020

Time: 9:47:13 PM

File Name: 2afmm1.gsz

Directory: C:\Users\ash8\Documents\slope site 2\final result site 2\

Last Solved Date: 1/3/2020

Last Solved Time: 10:26:36 PM

Critical Slip Surfaces

Number	FOS	Center (m)	Radius (m)	Entry (m)	Exit (m)
4	1.793	30.37, 33.08	30.249	(0.96, 26)	(33.22, 2.97)

Slices of Slip Surface: 4

	Slip Surface	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
1	4	1.49	24.255	-237.8	-38.80	-11.98	52.73
2	4	2.56	21.25	-208.4	13.03	4.02	52.73
3	4	3.55	19.13	-187.6	38.13	20.21	60.41
4	4	4.59	17.33	-169.9	65.39	34.67	60.41
5	4	5.71	15.60	-152.9	88.73	47.039	60.41
6	4	6.81	14.09	-138.1	108.68	57.62	60.41
7	4	7.98	12.77	-125.1	126.35	66.98	60.41
8	4	9.12	11.57	-113.5	142.41	75.49	60.41

9	4	10.25	10.51	-103.0	157.35	83.42	60.41
10	4	11.34	9.58	-93.95	173.57	68.41	73.63
11	4	12.37	8.78	-86.08	186.95	73.68	73.63
12	4	13.41	8.04	-78.88	199.80	78.74	73.63
13	4	14.45	7.37	-72.29	212.19	83.62	73.63
14	4	15.48	6.76	-66.28	224.12	88.33	73.63
15	4	16.56	6.177	-60.57	229.14	90.30	73.63
16	4	17.68	5.62	-55.19	226.59	89.30	73.63
17	4	18.81	5.14	-50.37	222.45	87.672	73.63
18	4	19.94	4.69	-46.03	216.51	85.33	73.63
19	4	21.06	4.31	-42.25	208.5	82.17	73.63
20	4	22.18	3.96	-38.91	198.20	78.11	73.63
21	4	23.31	3.6754	-36.04	185.44	73.084	73.63
22	4	24.43	3.428	-33.61	170.0733	67.028	73.63
23	4	25.57	3.2238	-31.61	153.7876	60.6095	73.63
24	4	26.71	3.062	-30.03	136.655	53.85642	73.63
25	4	27.85	2.94536	-28.88	117.0284	46.12233	73.63
26	4	29	2.872	-28.16	95.1288	37.4914	73.63
27	4	30.14	2.8417	-27.93	71.2370	28.07537	73.63
28	4	31.28	2.85	-27.99	45.74	18.02	73.63
29	4	32.42	2.91	-28.55	19.04	7.50	73.63
30	4	33.11	2.96	-29.03	4.7199198	1.86	73.63

APPENDIX D: DIFFERENT TYPEOF CHARTS

Coarse-Grained Soils

% passing #200	% of C.F. passing #4	% passing #200			USCS Symbol	USCS Name
<50%	>50%	0-5%	$c_u>6$ and $1<c_c<3$?	yes	SW	Well-graded sand
				no	SP	Poorly-graded sand
		5-12%	Dual classification		SP-SM	Poorly-graded sand with silt
					SP-SC	Poorly-graded sand with clay
					SW-SM	Well-graded sand with silt
					SW-SC	Well-graded sand with clay
		12-50%	PI>0.73(LL-20)%?	yes	SC	Clayey sand
				no	SM	Silty sand
	<50%	0-5%	$c_u>4$ and $1<c_c<3$?	yes	GW	Well-graded gravel
				no	GP	Poorly-graded gravel
		5-12%	Dual classification		GP-GM	Poorly-graded gravel with silt
					GP-GC	Poorly-graded gravel with clay
					GW-GM	Well-graded gravel with silt
					GW-GC	Well-graded gravel with clay
		12-50%	PI>0.73(LL-20)%?	yes	GC	Clayey gravel
				no	GM	Silty gravel

Table C. 1 AASHTO Classification chart for Coarse-Grained Soils (Based on the fraction passing No. 40 sieve)

Soil group (1)		Grain size: Passing No. 10 sieve (2)	Grain size: Passing No. 40 sieve (3)	Grain size: Passing No. 200 sieve (4)	Liquid limit* (5)	Plasticity index* (6)	Material type (7)	Subgrade rating (8)
A-1	*A-1-a*	50 max.	30 max.	15 max.		6 max	Stone fragments, gravel and sand	Excellent to good
	A-1-b		50 max.	25 max.		6 max.		
A-3			51 min.	10 max.		Nonplastic	Fine sand	
A-2	*A-2-4*			35 max.	40 max.	10 max.	Silty and clayey gravel and sand	
	A-2-5			35 max.	41 min.	10 max.		
	A-2-6			35 max.	40 max.	11 min.		
	A-2-7			35 max.	41 min.	11 min.		

Table C. 2 Typical values of unit weight for soils

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	16–22	14–21

Table C. 3 Descriptions Based on Relative Density

D_r (%)	Description
0–15	Very loose
15–35	Loose
35–65	Medium dense
65–85	Dense
85–100	Very dense

Table C. 4 Hydraulic conductivity of some soils (after Casagrande, 1939)

K (cm/sec)	soil type	drainage conditions
10^1 to 10^2	clean gravels	good
10^1	clean sand	Good
10^{-1} to 10^{-4}	clean sand and gravel mixtures	good
10^{-5}	very fine sand	Poor
10^{-6}	silt	Poor
10^{-7} to 10^{-9}	clay soils	practically impervious