Slope Instability Assessments And Mitigation Measures On Road Failure: In Case Of Addisu Gebeya To Sululta Road

Rihanna Ansuar Adem

A Thesis Submitted to the Department of Civil Engineering
School of Civil Engineering and Architecture

Presented In Partial Fulfillment of The Requirement for The Degree of Masters in Civil Engineering (Geotechnical Engineering)

Office of Graduate Studies
Adama Science and Technology University

June,2023
Adama, Ethiopia

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Declaration

I hereby declare that this Master Thesis entitled **"Slope Instability Assessments and Mitigation Measures on Road Failure: In Case of Addisu Gebeya to Sululta Road"** is my original work. That is, it has not been submitted for academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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Name of student	Signature	Date

Recommendation of the Advisor

I, the advisor of the thesis entitled **"Slope Instability Assessments and Mitigation Measures on Road Failure in Case of Addisu Gebeya to Sululta Road"** by Rihanna Ansuar, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Srikanth Vadlamudi (PhD)		
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Co-Advisor/ Co- Supervisor	Signature	Date

Approval of the Advisor

I, the major advisor of this thesis entitled **"Slope Instability Assessments and Mitigation Measures on Road Failure: In Case of Addisu Gebeya to Sululta Road"** and developed by Rihanna Ansuar, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Srikanth Vadlamudi (PhD)		
Major Advisor	Signature	Date

Approval of the Board of examiners

We, the undersigned, members of the Board of examiners of the thesis by Rihanna Ansuar have read and evaluated the thesis entitled **"Slope Instability Assessments and Mitigation Measures on Road Failure: In Case of Addisu Gebeya to Sululta Road"** and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Geotechnical Engineering.

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LIST OF ACRONYMS AND ABBREVIATIONS

ASTM	American Society for Testing and Materials
FEM	Finite Element Method
FOS	Factor of Safety
GPS	Global Positioning System
LEM	Limit Equilibrium Method
LL	Liquid Limit
MDD	Maximum Dry Density
MSMR	Modified Slope Mass Rating
MC	Mohr-Coulomb
MP	Morgenstern Price
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
SMR	Slope Mass Rating
SSPC	Slope Stability Probability Classification
TP	Test Pit
c	cohesion
ϒ	Unit Weight
τ	Shear Stress
ϕ	Frictional Angle
σ	Normal Stress

ABSTRACT

The aim of this research study was to investigate the geotechnical properties of various types of soil, assess the slopes' stability, and offering suggestions for mitigation measures to deal with the slope stability issues. Fieldwork, laboratory tests, and slope stability studies are some of the approaches/methods employed. This research was conducted on the road from Entoto check point to Dilber area since this road is used since it is the Addis Ababa - Bahirdar connecting route and has traffic flow every day. Research was done by selecting three critical slope sections and conducting laboratory tests and slope stability analysis using both limit equilibrium method (Slope/W) and finite element method (Plaxis) in order to get factor of safety and observe the difference between the two methods. The laboratory tests were conducted using standards and the conducted tests are natural moisture content, .Specific gravity, Atterberg limit, Grain size distribution, Direct shear test, Free swell test and Compaction in order to characterize the soil material gathered. The software analysis by Slope/W resulted a factor of safety of 1.281, 1.415, and 1.352 respectively for the three critical sections and the software Plaxis resulted a factor of safety 1.237, 1.361 and 1.14 respectively. The possible causes for future instability can be due to the presence of the clay soil that is being subjected to intense rainfall for long period throughout the year. Mitigation measures were also suggested in this research which are geometric modification, providing proper drainage, providing retaining structures such as Gabion walls, remove unstable sections and increase vegetation coverage in order to avoid future problem.

Keywords: Geotechnical, Limit Equilibrium Method, Finite Element Method, Factor of Safety, Gabion, Slope Stability.

CHAPTER ONE

1. INTRODUCTION

1.1. Background of the study

In geotechnical engineering, slope stability is one of the main issues where disasters that results human casualties and damage to supporting infrastructure (Dai & Lee, 2002; Kanungo et al., 2006; Lee et al., 2008). Mass movements that are driven by gravitational forces are constantly changing the earth's surface. Hill slopes' gradients may be decreased to stable angles as a result of the large-scale movements known as landslides. Although there are many natural causes of slope instabilities, in many situations, human activity such as road construction or other disturbance of hillsides directly causes slope instability (Gorsevski et al., 2006).

Slope instability can result from a combination of internal and external factors that result in an immediate or nearly immediate slope failure, either by lessening the material's shear strength or raising the shear stress on the slope (Gebremicheal, 2017).

It is challenging to completely avoid slope failure because both natural and human activities are to blame. However, by evaluating the stability condition and taking various preventive steps on it, the degree of damage can be considerably reduced. Different corrective actions are taken to lessen the effects of slope failures, and these actions can be divided into four classes: slope reinforcement, drainage control, geometric alteration, and retaining structure (Popescu, 2001).It should be noted that understanding the cause of failure, mode of failure, and methods of stability analysis approaches are required in order to design safe and affordable slopes, evaluate the stability of existing slopes, and implement appropriate remedial measures on it.

Ethiopia has been engaged in extensive road construction over the past twenty years because it recognizes the value of the transportation industry. The nation has been constructing additional roads to connect local administrative regions (Tabias/Keble). (Woldearegay, 2013).

A well-planned and executed road project has many advantages, one of which is fostering growth through various development endeavors of a country, such as: promoting market access opportunities for agricultural products, access to services (health, schools, etc.), and contributing to the socioeconomic development of a region (Wu & Abdel-Latif, 2000).

In an average year, landslides kill about 5-7 people in Norway, 18 in Italy, 25–50 in the United States, 186 in Nepal, 170 in Japan, and 140–150 in China. This figure accounts for both the minor but common and the huge and rare catastrophic landslides (Sidle & Ochiai, 2013). Although landslides can result in fatalities, they can also cause property damage, infrastructure loss, loss of productivity in the affected area, unpredictable changes to the local watercourse, and a reduction in usable or inhabited land Schuster, (1996) as quoted in Lee et al., (2008).

The development of soil and rock mechanics as a whole has closely paralleled the development of slope stability analysis in geotechnical engineering. The integrity and future performance of slopes linked with planned engineering development must be taken into consideration when designing new slopes as well as when determining potentially unstable sections in existing slopes. An understanding of analytical techniques, investigative tools, and stabilizing techniques is necessary for engineering solutions to slope instability concerns (Abramson et al., 2002) .

In civil engineering, assessing the stability of slopes in soil is a crucial, intriguing, and difficult task. The earth's geomorphology is naturally shaped by a geodynamic process called slope instability. The safety of persons and property is a serious issue, though, if those unstable slopes were to affect them. Rain-induced landslides of all forms and sizes commonly impact Ethiopia's highland landscape, which is steep and mountainous. Debris or earth flows and medium- to large-scale rock slides are the two main forms of landslides that have been observed to be caused by intense rainfall.

The results of numerous slope stability studies conducted globally have improved knowledge of the causes, mechanisms, analytical techniques, and potential corrective actions. Similar investigations have been conducted in Ethiopia at various sites over the years. However, the damages due to slope failure are increasing from year to year and still the major difficulty for the development of infrastructural constructions. A review of previous slope stability assessments (Assefa et al., 2016; Samuel, 2017 and Woldearegay, 2013) indicates that slope failures are the main constraint for road and railway construction in Ethiopia.

The road on the study area is a road found on northern part of the city of Addis Ababa which is located on high altitude and have visual slope stability problems and since the road have high traffic flow any cautions and studies must be undertaken in order to avoid catastrophic problems on the location, on the road and on the people nearby therefore a study is needed on the area.

1.2. Statement of the problem

Researchers and experts in the field of soil and rock slope engineering are primarily concerned about slope instability. Although it is the most frequent geohazard, slope failure is the second most devastating natural hazard after earthquake (Li et al., 1999) . The need for applying and creating more trustworthy methods besides the conventional slope stability analysis model to examine the stability of those sensitive structures has increased due to the deaths of residents of hilly areas and the falling of blocks into roads. Slope failures are always catastrophic due to their large affected areas and great energy, generated by the collapsed soil or rocks with rapid and long run-out movement. This study has very important scientific significance and practical worth. The main triggering factors for the instabilities of the areas are the different environmental factors (e.g., intensive rainy summer), variable geological and structural elements (weak rocks, slide debris weak soils, shear zones, and faults) difficult road characteristics (narrow roads with tight horizontal and vertical curvature. This study will forward the general remedial measures which may be adopted to minimize or to eliminate the possible hazard along the road. This will be accomplished by using different methodologies and techniques based on slope stability result and site conditions.

In our country, the stability of the slope is currently a problem because landslide activities seriously harm infrastructures and human life, particularly when building roads. The road cut slopes of the study areas are prone to challenges because roads and highways were built in high altitude regions, particularly during periods of heavy rainfall infiltration. Therefore, rather than just considering the present circumstances, road cut slopes must also be built to withstand the worst possible future conditions. Failure of these slopes can be attributed to improperly designed embankments, cut slopes, and support structures that failed to account for all potential undermining factors. One factor contributing to this issue is the lack of knowledge about advanced computational techniques in the construction sector.

1.3. Objectives

1.3.1. General Objective

The general objective of this research is to assess the slope instability leading to road failure in case of Addisu Gebeya to Sululta road, and to identify and evaluate effective mitigation measures for preventing or minimizing such failures.

1.3.2. Specific Objective

1. To identify the geotechnical characteristics of soils present in the study area by conducting various laboratory test
2. To identify the causes, triggers, and effects of the landslide in the study area.
3. To perform a software-based study of slope stability to determine factor of safeties using limit equilibrium method and finite element method.
4. To recommend a mitigation measures in order to overcome future problems.

1.4. Significance of the study

The results of this study will be used to identify the type, characteristics, and role of the soils in the occurrence of slope instability, as well as the strength and safety factor of the soil, the consequences of slope instability, the causes of downward soil mass movement, and potential preventative and corrective measures. The results of this study will be used by academic institutions, governmental agencies, the commercial sector, and other interested parties to assess erosion and slope stability hazards and carry out remedial activities around the study region. The study's findings will be extremely important for the rising number of highway and railroad project involving cut and embankment slopes. The effectiveness and appropriateness of the various slope profiles described in this study will help with the selection and cost-effective design of cut slopes with related geologic conditions.

Additionally, such research work may inspire additional scholars to work on a thorough slope stability analysis in other regions of the nation in order to lessen the potential harm that could be caused by the landslide threats there.

1.5. Scope of the study

In this research, the finite element method and limit equilibrium were both used to analyze slope. Three locations were chosen to illustrate various soil mass situations on road, and was examined in this research. Suitable data for the analysis was collected through field investigation and laboratory tests. And software analysis was done by SLOPE/W(FEM) and PLAXIS(FEM)

1.6. Organization of thesis

This work is reported in 5 chapters.

Chapter 1 provides a general introduction of slope stability, statement of the problem, and the research objectives including its scope and significance.

Chapter 2 presents a literature review about slope failure and its stability analysis. Various types and causes of failure, shear strength criteria, and different stability analysis approaches are discussed. Further slope stability studies, stabilization methods, and software applications also included in this chapter.

Chapter 3 presents methods and methodology which contains description of the study area, location, population, climate and topography. The methodology was also be discussed under this section which covers the methods implied for the research such as desk study, field investigation, sampling, laboratory tests to be conducted and at last computer soft wares to be used for analyses.

Chapter 4 presents mathematical modelling of softwares used for analysis.

Chapter 5 presents the result of the laboratory tests mentioned in methodology and the software analysis results using the methods limit equilibrium method(Slope/W) and finite element method (Plaxis) after determination of factor of safety from analysis a remedial measure is suggested in order to prevent further failure of slope in the area.

Chapter 6 presents conclusion and recommendation for the research conducted according to the results withdrawn through the research period.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. General

Failure of slopes is a recurring problem that affects infrastructures, human life, and natural resources all over the world. The major objective of slope stability studies is to assist in the safe and economical design of new slopes and stability evaluation of existing slopes. Engineering solutions to slope stability problems require a full understanding of the causes and mechanisms of slope failure, analytical approaches, investigative tools, and stabilization measures.

Slope failures can occur suddenly or slowly, with or without apparent cause, and in practically every situation imaginable. Excavation, undercutting the base of an existing slope, a gradual breakdown of the soil's structure, an increase in the pore water pressure in a few particularly porous strata, or a shock that causes the soil to liquefy are the common reasons. Issues with slope collapses, both man-made and natural, frequently cause tremendous challenges in geotechnical engineering.(Nelson, 2013)

Therefore, a review of the causes and mechanisms of failure, shear strength criteria, analysis techniques, past stability evaluations, stabilization techniques, and software applications was done in this part to have a knowledge about these failures.

2.2. Types of slopes

There are generally two main types of slopes in which geotechnical engineers are interested in which are natural slopes and man-made slopes. Whereas the former is formed due to geological and geomorphological processes in nature, the latter is formed by human activity usually during the process of construction or waste disposal

Slopes can also be separated into three classes as stated by Msilimba, (2002):

1. **Stable slopes** are those whose margin of stability is high enough to withstand all destabilizing forces,
2. **Marginally stable slopes** are those that will eventually fail when the activity of the destabilizing forces reaches a critical level, and

3. **Active unstable slopes** are those where the destabilizing forces cause continuous or intermittent movement.

Types of slopes can also be based on the basis of type of soil as:

1. **Cohesive soil slope:** Having purely cohesive soil as its content.
2. **Frictional soil slope:** Slopes having frictional soil as its contents.
3. **Cohesive frictional soil:** Slopes made up of soil which has both frictional as well as cohesive properties.

Another type of slope is according to extent are:

Infinite Slopes

A type of slope that continues infinitely or simply goes so far as to have no defined borders. The soil parameters at all similar depths below the surface are identical for this sort of slope. There is no contribution to the creation of natural slopes.

Finite Slopes

The slope is of a limited extent. These slopes can be dealt by engineers. A constant slope with an unlimited extent is referred to as having an infinite slope. This kind of slope has an unlimited extent, whereas finite slopes have a long slope like the mountain's face. Examples of finite slopes are the slopes of earth dams and embankments. The height of the dam determines the slope's length.

2.3. Factors influencing slope stability

Numerous variables influence the stability of slopes. Any one of these variables, alone or in combination, can change the slope's steady state condition, reducing its stability and resulting in slope failure. When the slope is in a critical stability state, a comparatively sudden triggering event of natural (like an earthquake, soil saturation) and human events can cause destabilization. The following are some explanations of the key elements influencing slope stability. (Duncan et al., 2014).

2.3.1. Force of gravity

The primary factor influencing shear stress is the gravitational force. The stability of the slope is influenced by the gradient of the inclination. The gravitational forces can be split into two components a component acting perpendicular to the slope and a component acting tangential to the slope. On steeper slopes, shear stress, sometimes referred to as the tangential component of gravity, rises while the perpendicular component of gravity decreases. Therefore, the ability of a material to flow downhill is affected by steep slope angles that increase the shear stress and decrease shear strength. Shear stress in a material can be increased by undercutting, mining action, tectonic tilting, and elimination of lateral support. Rock or regolith's intrinsic properties, such as the angle of internal friction, cohesion, and the binding force of plant roots between particles, determine the shear strength of the material.

2.3.2. Hydrologic factor

Both the soil mass and the solid rock contain a great deal of water. Rocks and sediment can slide down slopes more easily as a result of gravity when water is present since it can weaken the shear force. Water reduces the shear strength of earth elements by applying positive pressure within their pore spaces. Water that seeps into slope materials can deeply saturate the soil granules by filling the pore spaces. The soil particles are separated by the water pressure created by the weight of the water on top(Sidle, 1985).

Slope failures commonly occur after intense, persistent rain (Boylan et al., 2008). This is a triggering factor that is often taken into account for dynamic slope failure models. Slopes are also eroded by streams' erosive activity in addition to rainfall. Streams cut into the lower valley's slope, raising its grade and causing certain areas of instability.

2.3.3. Seismicity

Slope failure can be brought on by explosions, earthquakes, or volcanic eruptions that raise shear stress. These situations evolved naturally, but human impact can make them worse. The water pressure produced by vigorous shaking in the pore areas of sediments might cause them to liquefy. Slopes that were previously stable can collapse because of the increased vertical acceleration brought on by earthquake vibration. According to Muthu & Petrou, (2007) the likelihood that an earthquake may cause a landslide event depends more on the shaking of the ground than the earthquake's actual magnitude. The likelihood of an earthquake triggering a landslide event depends on the magnitude of the earthquake itself.

2.3.4. Land cover change

According to Cruden & Varnes, (1996)one of the most significant variables influencing the occurrence of slope failures brought on by rainfall is land cover change. With deforestation, the risk of unstable hillside increases. People in developing countries have removed land of trees and other vegetation to build their homes. The slope is kept solid by the roots of the vegetation, which hold the soil together and protect it from heavy rainfall. But if the vegetation is removed the slope is left open to the possibility of failure from erosion. Many slope failures occur in regions where there has been significant deforestation.(Ayenew & Barbieri, 2005)

2.3.5. Anthropogenic (human) factors

Anthropogenic factors are associated with human activity in response to variations in slope. Human activity can affect a landscape's slope, although this ultimately fails. A summary of human behaviors that can trigger landslides can be found below.

- ✓ Undercutting and slope modification techniques used in the construction of freeways and roads produce an artificial slope that is steeper than the angle of repose. As a result, the average slope gradients increase. Therefore, slope failures are more likely as a result.
- ✓ Slopes are overloaded when mining and quarrying activities are taking place. Changes in hydrology may have a substantial impact on slope stability, and this additional weight may increase the chance that the slope will fail. (Boylan et al., 2008).
- ✓ Deforestation brought on by human activity promotes the failure of soil slopes because tree roots are no longer able to keep the soil in place and can no longer shield it from heavy rain. It also reduces evapo-transpiration and raises water levels. (Cruden & Varnes, 1996).
- ✓ Vibrations brought on by man-made events like machine activity and underground blasts can seriously flood large reservoirs and cause sediment failure.

2.3.6. Geologic factors

Geology is one of the key factors considered in a slope stability analysis (Cruden & Varnes, 1996). Depending on the type of regolith, there is a substantial relationship between the geology of the material and slope instability in a particular area. Weathering changes the mechanical, mineral, and hydraulic properties of the regolith, which in many situations plays a substantial role in slope instability. Geological formations and type of lithology have a big impact on slope stability.

2.3.7. Geomorphic factors

Slope instability initiation is significantly influenced by geomorphic variables. These characteristics, such as the slope's gradient, aspect, and shape, are directly linked to the topography of the region.

2.3.8. Slope aspect and shape

The slope's aspect is the way that it is facing. It significantly influences hydrological processes through differential evaporation, which in turn affects weathering and root development, especially in drier areas (Sidle & Ochiai, 2013). Particularly in metamorphic rocks, the relationship between aspect and bedrock structure is strong. Rock failures are common on slope aspects that are parallel to the foliation and lineation lines (Vieira & Fernandes, 2004).

By concentrating or dispersing surface and mainly subsurface water in the landscape, the slope's shape has a significant impact on the stability of steep terrain(Sidle & Ochiai, 2013). a) Divergent, b) planar, and c) convergent slope forms all have varying degrees of water infiltration, making them three hydro-geomorphic slope units that are significant in determining terrain stability.

2.3.9. Engineering properties of soils

An evaluation of the soil shear strength that is quantitative is necessary for analyses of slope stability that use limiting equilibrium approaches. This clarifies important technical properties of slope stability and their connections to other aspects of soil properties (Wu & Sangrey, 1978). A fundamental characteristic known as soil shear strength controls the stability of both natural and man-made slopes. Three shear strength parameters—normal stress on the slip surface, cohesiveness, and internal angle of friction—are basically utilized to assess the shear strength of soil materials. According to Baver et al., (1972), soil cohesiveness and strength decline as soil moisture levels rise. Due to the higher surface contact area of clay particles compared to other soil particles, cohesion of the substance often increases correspondingly to clay concentration. The internal angle of friction is significant in soils with low cohesiveness.

2.4. Modes of Slope Failure

The stability of rock slopes is greatly influenced by joint sets and their orientation, joint properties, seepage pressure, the depth and steepness of the excavated slope face, and the orientation of the slope face with respect to the joint set. The following types of rock slope collapse occur frequently (Spang & Sonser, 1995):

(A) Circular/rotational or quasi-rotational failure: - Failure of this kind is a heavily jointed, crushed, and weathered rock mass is anticipated to slide along a curved surface and this usually happens at a larger scale. Frequently, the sliding surface is approximated to a circle arc and referred to as circular in order to make the analysis simpler (Spang & Sonser, 1995).

(B) Plane slide: -When a rock block rests on an inclined weakness plane that "daylights" into the free space, this type of failure develops under gravity alone. Failure plane inclination larger than plane friction angle (Spang & Sonser, 1995).

(C) Wedge failure/slide: -When two weakening planes meet to form a tetrahedral block, wedge failure results. This type of failure can happen if the junction line of the two discontinuities daylights into the excavation and the intersection line of the two planes has a low friction angle (Spang & Sonser, 1995).

(D) Toppling failure:- The rock strata are turned over in this kind of failure: A block will topple and rotate around its lowest contact edge if the vector corresponding to its weight descends outside of the base (Spang & Sonser, 1995).

(E) Buckling failure:- Depending on the depth of the excavation, these layers may buckle and fracture near the toe and sliding of the top sections of the layers may result in a buckling type of failure where the excavation is carried out with its face parallel to the thin weakly bonded and steeply dipping layers (Spang & Sonser, 1995).

2.5. Types of slope failure

The following are the most commonly observed slope failures all around the world according to The Landslide Handbook— A Guide to Understanding Landslides (Highland & Bobrowsky, 2008) .

1. **SLIDES:** Although the general term "landslide" refers to a variety of mass movements, the more specific use of the term only refers to mass movements in which the slide material is clearly separated from more stable underlying material by a zone of weakness. Rotational slides and translational slides are the two main categories of slides.

   Rotational slide**:** This slide has a surface of rupture that is concavely curved upward, and it moves roughly in a rotation around an axis that is parallel to the ground and extends transverse to the slide. (Fig 2.1A)

   Translational slide**:** In this type of slide, the landslide mass moves along a roughly planar surface with little rotation or backward tilting. (Fig 2.1B)

   A *block slide:* is a translational slide in which the moving mass consists of a single unit or a few closely related units that move down slope as a relatively coherent mass. (Fig 2.1C)

2. **FALLS:** Falls are sudden movements of geologic material masses, such as rocks and boulders, that separate from cliffs or steep slopes. Movement involves falling freely, bouncing, and rolling, and separation happens along discontinuities like fractures, joints, and bedding planes. Gravity, mechanical weathering, and the presence of interstitial water all have a significant impact on falls. (fig2.1D)
3. **TOPPLES:** Toppling failures are distinguished by the forward rotation of a unit or units about some pivotal point, below or low in the unit, under the actions of gravity and forces exerted by adjacent units or by fluids in cracks. (Fig2.1E)
4. **FLOWS:** There are five basic categories of flows that differ from one another in fundamental ways.

   a.Debris flow: When loose soil, rock, organic matter, air, and water mobilize to produce a slurry that flows down slope, it is referred to as a debris flow. About 50% of debris flows are fines. Debris flows are frequently created by high surface-water flow that erodes and mobilizes loose soil or rock on steep slopes as a result of heavy precipitation or quick snow melt. Other forms of landslides that happen on steep slopes, are almost saturated, and contain a significant amount of silt- and sand-sized material frequently mobilize debris flows as well. Steep gullies are frequently found near debris-flow source locations, and the presence of debris fans at gully mouths typically indicates the existence of debris-flow deposits.(fig2.1F)

b. Debris avalanche: This is a variety of very rapid to extremely rapid debris flow. (Fig 2.1G)

c. Earth flow: The "hourglass" shape of earth flows is a unique feature. A bowl-shaped dip or run-out from the slope material creates the slope's head. It typically happens in fine-grained materials or rocks that contain clay, on moderate slopes, and in saturated conditions. The flow itself is elongate. However, it is also possible for granular material to flow dry.(Fig 2.1H)

d. Mud-flow: A mud-flow is an earth flow consisting of material that is wet enough to flow rapidly and that contains at least 50 percent sand-, silt-, and clay-sized particles.

e. Creep: The gradual, steady descent of slope-forming soil or rock is known as creep. Shear stress that is large enough to create permanent deformation but not enough to cause shear collapse is what causes movement. In general, there are three different kinds of creep: (1) seasonal, where movement occurs within the depth of the soil and is influenced by seasonal variations in soil moisture and soil temperature; (2) continuous, where shear stress continuously exceeds the material's strength; and (3) progressive, where slopes are approaching the point of failure along with other types of mass movements. Curved tree trunks, twisted fences or retaining walls, tilted poles or fences, and minor soil ripples or ridges are all signs of creep. (Fig 2.1I)

5. **LATERAL SPREADS:** Lateral spreads are distinctive because they usually occur on very gentle slopes or flat terrain. With shear or tensile fractures, lateral extension is the predominant mechanism of movement. Liquefaction, the process by which saturated, loose, cohesion less sediments (often sands and silts) are changed from a solid to a liquefied condition, is what led to the failure. Failure can be purposely generated but is typically brought on by sudden ground motion, such as that encountered during an earthquake. The top units may fracture and extend, subside, translate, rotate, disintegrate, or liquefy and flow when cohesive material, such as bedrock or soil, rests on materials that liquefy. Lateral spreading in fine-grained materials on shallow slopes is usually progressive. A small region experiences an abrupt start to the collapse, which spreads quickly. Although a slump is frequently the first failure, certain materials shift

for no apparent reason. A complicated landslide is a combination of two or more of the aforementioned types. (Fig 2.1J)

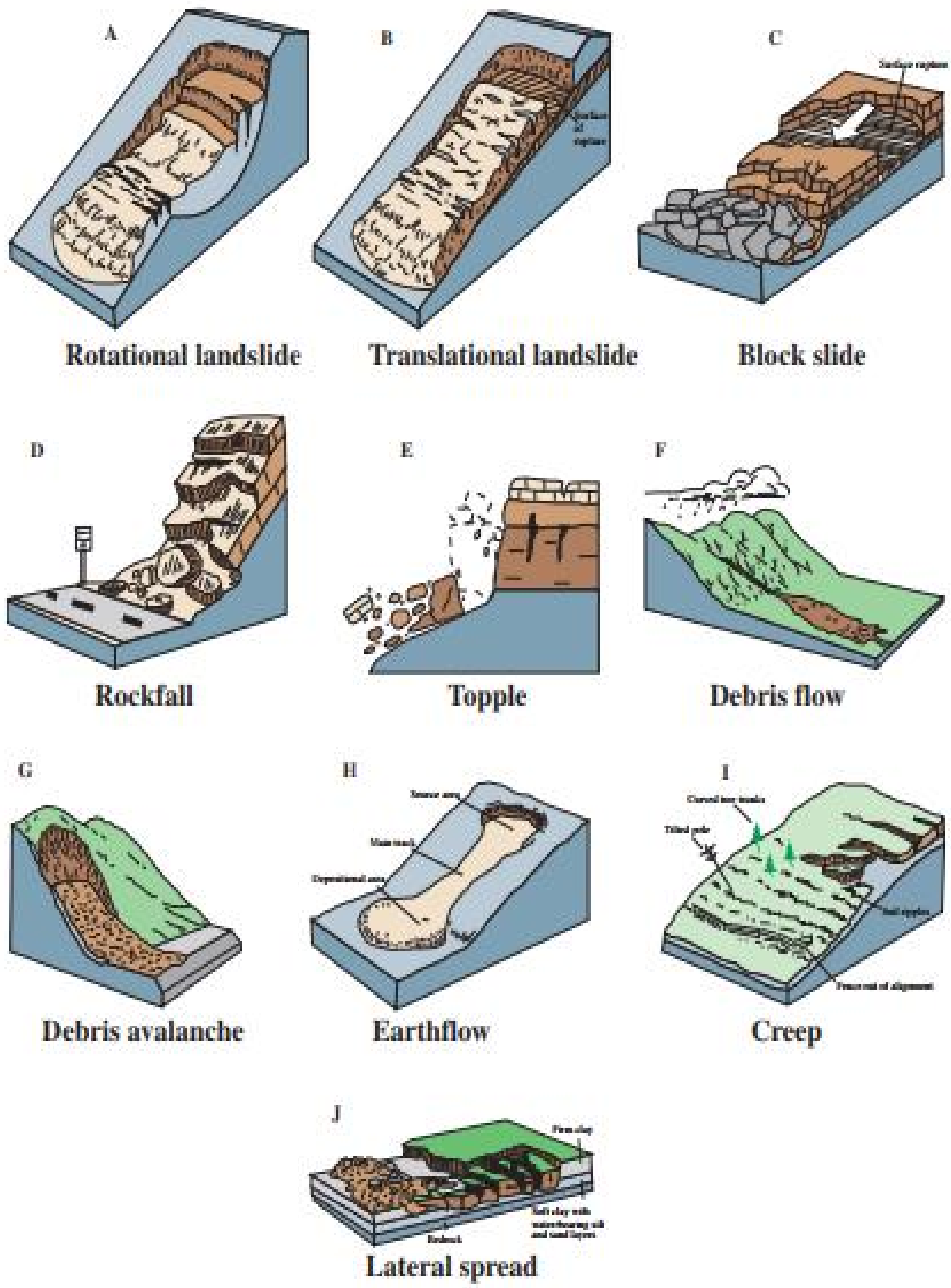


Figure 2.1: Schematic illustration of the major types of landmass movement(Highland & Bobrowsky, 2008) .

Table 2.1: Types of landmass movement. Abbreviated version of Varnes’ classification of slope movements (Varnes, 1978)

TYPE OF MOVEMENT		TYPE OF MATERIAL		
		BEDROCK	ENGINEERING SOILS	
			Predominantly coarse	Predominantly fine
FALLS		Rock fall	Debris fall	Earth fall
TOPPLES		Rock topple	Debris topple	Earth topple
SLIDES	ROTATIONAL	Rock slide	Debris slide	Earth slide
	TRANSLATIONAL			
LATERAL SPREADS		Rock spread	Debris spread	Earth spread
FLOWS		Rock flow (deep creep)	Debris flow	Earth flow
			(soil creep)	
COMPLEX		Combination of two or more principal types of movement		

The table 2.1 is based on the classification of Varnes,(1978) The chief criteria used in the classification presented here are, as in 1958, type of movement primarily and type of material secondarily. Types of movement (defined above) are divided into five main groups: falls, topples, slides, spreads, and flows. A sixth group, complex slope movements, includes combinations of two or more of the other five types.

2.6. Shear strength parameter of the slope material

Cohesion and the angle of internal friction are the two shear strength variables that are most important for defining the failure of slope materials. Any reduction in these shear strength parameters will therefore have a significant effect on the slope stability.

1. Cohesion

Cohesion is a measurement of a rock's or soil's resistance when there is no intermediate contact. Soil aggregation is the outcome of an enormous and infinite number of resistances, each with a unique set of resilience values. A rock's or soil's quality can be determined by how effectively it resists breaking or being harmed by forces like gravity. Granite and soil with weak connections to the slope tend to be less stable.

The following elements increase the cohesive power of soil and rock: a) Stickiness or friction between the particles may hold the soil granules together. b) man-made reinforcements can stop certain material mobility. c) The cementation of grains through the deposition of calcite or silica can consolidate soil components into strong rocks.

The following list of substances can weaken the cohesive power of soil and rock materials: a) A high water content can make collaboration challenging since too much water will absorb soil and make a mass of soil heavier. b) Water expands and contracts soil as it dries, decreasing the cohesiveness just like with expansion during freezing and constriction during thawing. c) Undercutting at the base of the slopes. d) Earthquake, sonic boom, and blasting-related vibrations disrupt cohesiveness and cause mass movement.

2. The angle of internal friction

The interior friction angle is one of the indicators of a soil's or rock's shear resistance. This characteristic refers to a rock or soil's capacity to withstand stress. It describes the organized soil's resistance to friction shear under normal effective stress and is derived from the Mohr-Coulomb failure criterion. The slope of the Mohr-Coulomb shear resistance line with respect to the horizontal axis of the stress plane of effective/normal shear stress is the angle of internal friction of the soil. The value of the angle of internal friction depends on the size and roundness of the particles. A lesser degree of roundness or a higher median particle size result in a bigger friction angle.

Shear strength testing on soil can be done in a number of ways: The vane shear test, the bore hole shear test, the direct shear test, the triaxial test and the unconfined compression (UCC) test.

The shear strength formula outlines the Mohr–Coulomb envelope, which states that the shear strength is equal to the sum of cohesion (c) and a frictional component ($\sigma'\tan\varphi$). The shear strength formula of granular soils is:

$$\boldsymbol{\tau = c + \sigma' \tan\varphi} \text{………………….Equation 1}$$

where τ = soil shear strength (kPa) c = cohesion (kPa)

σ' = normal effective stress (kPa)

φ = the frictional angle

2.6.1. Shear Strength Criteria of Soil

1. Coulomb failure criteria

According to Coulomb's failure criteria, a slip surface's impending frictional sliding causes a failure. In this soil failure criteria, the earth is regarded as a rigid, frictional material. The most suitable soils to use Coulomb failure criteria on are those that are layered, fissured, over consolidated, or have a pre failure plane exists. Direct shear is used to interpret the test results.

2. Mohr-Coulomb Failure Criterion for Soil

The Mohr-Coulomb measure is the one used in geotechnical engineering most frequently. The use of this strength model is necessary for many geotechnical research techniques and programs. A linear relationship between cohesion, normal, and shear stresses (or maximum and lowest principal stresses) at failure is described by the Mohr-Coulomb criterion.

Total stress $\boldsymbol{\tau = c + \sigma tan\phi}$..............................Equation 2

Effective stress $\boldsymbol{\tau f = c' + \sigma' tan\phi'}$ ....Equation 3

Where *c* is the cohesion, ϕ is the angle of internal friction of the soil and σ is the applied normal stress.

The MC failure criterion has the advantages of being mathematically straightforward, having a clear physical meaning for the material parameters, and being widely accepted. The numerical application of a failure criterion with corners in the p-plane as opposed to a smooth function, such as the Drucker & Prager, (1952) failure criterion, has several limitations. Deformation analysis requires a flow rule, a relationship between strain increments and stress, so that the flow rule determines the orientation of the strain increment vector with respect to the yield condition, for example, normal for an associative flow rule. As a result, the strain-increment vectors have a unique orientation along the sides of the MC pyramid. However, there is some flexibility in the orientation at the pyramid's edges (corners in the p-plane). (Drescher, 1991).

Mohr-coulomb uses an important combination of normal and shear stress to provide a soil substance failure criterion (Braja, 2010) . The Mohr-Coulomb criterion links the cohesion, normal tension, and angle of internal friction of the material to its shear strength on the failure plane. As shown in Figure below any combination of normal and shear stress along with the MC failure envelop (A) is secure and no combination of normal, (B) leads slope failure, any combination of normal and shear stress below the MC envelope and (C) shear stress can be existing above the failure envelope.

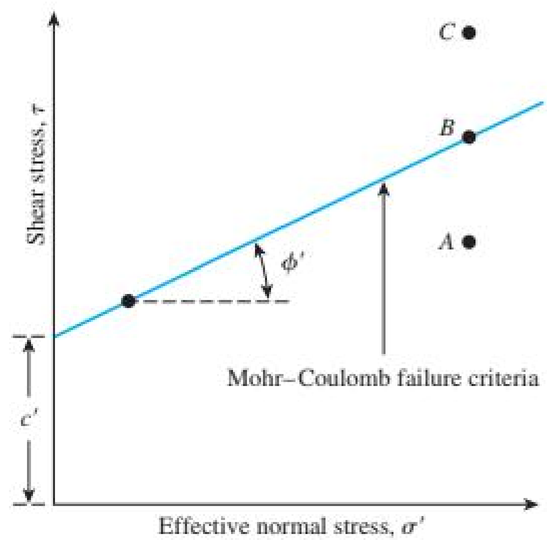


Figure 2.2: Mohr-Coulomb failure criterion (Das, 2010)

3. Tresca Failure Criteria

In this failure criteria failure occurs when one half the maximum principal stress difference is achieved. The Tresca failure criteria views earth as a homogeneous solid. The short-term (undrained state) strength of fine-grained soils is best measured using the Tresca failure criteria. Triaxial testing is used to analyze test results.

4. Taylor Failure Criteria

Failure is caused by soil particle interlocking and sliding (frictional strength). Taylor's failing criteria considers soil to be a frictional, deformable solid. It works best for the short- and long-term power of uniform soil. The direct simple shear test is used to evaluate test results.

2.7. Slope stability analysis approaches

As there is a growing need for a better understanding of the failure mechanism of both man-made and natural slopes, analysis of slope stability is a frequent task in geotechnical engineering. The main categories of stability analysis approaches are i) conventional methods which includes kinematic, empirical, deterministic/limit equilibrium, and probability methods (Baba et al., 2012; Karaman et al., 2013) and ii) numerical methods which includes continuum, discontinuum, and hybrid modeling (Eberhardt, 2003; Stead et al., 2006).

The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.(Abramson et al., 2002; Eberhardt, 2003).

2.7.1. Conventional approach

2.7.1.1.Kinematic approach

The kinematics principle is the foundation of kinematic approaches, which investigates the geometrical conditions necessary for the movement of the rock block over the discontinuity plane without taking into consideration any forces causing the sliding.(Kulatilake et al., 2011; Zainalabideen & Helal, 2016). The stereographic projections are used to determine the possible mode of rock slope failure (planer, wedge, toppling) by considering the strike angle, dip angle, and dip direction of slope and discontinuity surface in the kinematic analysis (Gadaawin, 2019).Kinematic analyses concentrate on the feasibility of translational failures due to the formation of day lighting wedges or planes of sliding. As such, these analyses rely on the detailed evaluation of rock mass structure and the geometry of existing discontinuity sets that may contribute to block instability (Zainalabideen & Helal, 2016).

2.7.1.2.Empirical methods

In past several empirical methods based on rock mass classification systems have been developed. The important slope classification systems are; classification proposed by Selby(1980), Slope Mass Rating (SMR) (Romana, 1985) ,Modified Slope Mass Rating (MSMR) (Anbalagan et al., 1992), Slope Stability Probability Classification (SSPC) (Hack, 1998) and rock mass classification system for slopes proposed by Liu & Chen,(2007). SMR classification, proposed by Romana, (1985) can be used to assess the stability condition of a rock slope. Anbalagan et al., (1992) modified SMR by considering wedge mode of failure as a separate case. For stability analysis of slope, having plane mode of failure, both SMR and MSMR classifications can be utilized. Hack, (1996) proposed SSPC to classify the rock mass and to define its in situ stability condition with probability of failure to occur.

2.7.1.3.Probabilistic methods

The probabilistic methods facilitate to incorporate parameters, which show uncertainty, in a systematic way and define the stability condition of the slope in probabilistic terms (Alzo’ubi, 2016; Chowdhury et al., 2009). For probabilistic analysis of a slope, having plane mode of failure, the parameters to be used are first defined as fixed dimension parameters and as random variables (Hoek, 2007). Fixed dimension parameters are mainly the geometric parameters which can be obtained directly from the geometry of the slope such as; slope height, slope inclination, upper slope inclination and dip of the potential failure plane. The random variables are those which show uncertainty in their values and may vary considerably such as; cohesion (c) and angle of friction (φ), ratio of depth of water in tension crack to the depth of the tension crack etc. (Hoek, 2007).

2.7.1.4.Limit equilibrium methods

Limit equilibrium methods are relatively simple in its application, as compared to the numerical methods (Tang et al., 2017; Yang & Zou, 2006). For plane mode of failure analysis, the limit equilibrium methods were proposed by many researchers, such as; Tang et al.,(2017), Ahmadi & Eslami, (2011), Shukla et al.,(2009), Price, (2009), Hoek, (2007), Zheng et al., (2005), Sharma et al., (1999), Ling & Cheng, (1997), Sharma et al., (1995), Hoek & Bray, (1981) etc. In limit equilibrium method various forces responsible for driving of the rock mass and the resisting forces are evaluated and the ratio of resisting forces to driving forces at equilibrium, defines the factor of safety (FOS) (Hoek & Bray, 1981; Price, 2009; Sharma et al., 1995). If FOS value is more than ‘1’ it suggest stable slope, if less than ‘1’ unstable and if FOS is equal to ‘1’ the slope is in a critical state of equilibrium (Hossain, 2011). The FOS will be affected by various governing factors such as; geometry of the slope, failure plane characteristics, water forces and external triggering factors. The FOS is inversely proportional to the height of the slope. Increase in height (h) of the slope will increase the shear stresses, thus FOS will decrease (Anbalagan et al., 1992; Hack, 2002; Hoek & Bray, 1981; Raghuvanshi et al., 2014; Raghuvanshi & Solomon, 2005).

2.7.2. Numerical methods

Stability condition of complex slopes cannot be evaluated by following conventional techniques (Eberhardt, 2003; Karaman et al., 2013). In order to simulate such complexities, numerical methods are required. In general, numerical methods; include continuum, discontinuum and hybrid methods (Eberhardt, 2003; Stead et al., 2006; Tang et al., 2017).

2.7.2.1.Continuum modeling

The continuum modeling is based on finite element, finite difference and boundary element methods. The basic input data that is required for continuum modeling approach are; constitutive model, in-situ stress, shear strength parameters for surfaces, groundwater etc. For the slope stability analysis through continuum modeling finite difference and finite element models are commonly used. These methods can suitably be applied for weak rock masses in which failure results due to intact rock deformation or through the discrete stratified discontinuities (Stead et al., 2006). The major limitation with continuum techniques is in defining rock mass as continuum, for which simplifications are needed. Thus, the final results are highly influenced by such simplifications (Hack, 2002).

2.7.2.2.Discontinuum modeling

The Discontinuum modeling is suitable for discontinuous rock mass which contains discontinuities and the failure mechanism is controlled by pre-existing discontinuities. In Dis continuum analysis movement of intact rock blocks bounded within discontinuities and the deformation of intact rock can be addressed for both, static and the dynamic conditions (Stead et al., 2006). The input data required for modeling includes data on slope geometry, discontinuity characteristic, discontinuity shear strength and stiffness, in-situ stress and groundwater data. For Discontinuum modeling, distinct element method and discontinuous deformation analysis are widely used. The Discontinuum method is quite effective to simulate the discontinuity behavior and to assess the rock slope stability for given conditions (Stead et al., 2006).

2.7.2.3.Hybrid modeling

Hybrid modeling approaches utilizes the combined capabilities of both continuum and Dis continuum methods. The strength of the rock mass is a combined effect of strength along the rock joints and the strength of the intact rock between the joints (Alzo'ubi, 2016). Complex slope and discontinuity geometry can be considered in hybrid modeling approach. Step-path geometries can be simulated by hybrid finite-discrete element method with fracture propagation approach (Stead et al., 2006).

Each of the methods available for plane failure analysis has certain advantage and limitations (Abderrahmane & Abdelmadjid, 2016; Salunkhe, 2017; Stead et al., 2006). Thus, selection of these methods will depend on governing parameters involved in the analysis, complexity of the geological conditions, hydrologic and geometric parameters, purpose for which the slope stability has to be assessed, computational capacity and the capability of an evaluator.

Table 2.2: Advantages and limitations of conventional analysis methods (Raghuvanshi, 2019).

Conventional approach		
Method	Advantage	Limitation
Kinematic method	They are simple in their application.	They suggest the potential for failure and do not provide slope stability condition in quantitative terms.
Empirical methods	Applied over large area to investigate slope stability condition simple cases such as uniform planar discontinuities can be solved directly.	For complex cases, involving variable slope geometric and geologic conditions, these empirical methods cannot be applied.
Limit equilibrium methods	Simple and the data required for analysis can easily be collected from the field.	Assumptions made in limit equilibrium methods may possibly lead to over simplification and thus, the results may not be realistic.
Probabilistic methods	More realistic results on slope stability condition help to recognize and assess uncertainties among the governing parameters in a systematic manner	This method requires collection of detailed parameter data well distributed over the slope.

Table 2.3: Advantages and limitations of numerical modeling methods (Raghuvanshi, 2019).

Numerical approach		
Method	Advantage	Limitation
Continuum methods	Applied for weak rock masses in which failure results due to intact rock deformation or through the discrete stratified discontinuities.	Not reliable in slopes whose failure mechanism is governed by discontinuity
Discontinuum method	Suitable for rock mass which contains discontinuities and the failure mechanism is controlled by the pre-existing discontinuities	On-availability of discontinuity data at block level which generally leads to incorporate more simplified discontinuity data
Hybrid method	Able to simulate complex slope and discontinuity geometry	it requires high memory and high computational capacity of the system

Finally, the method to be adopted for the slope failure analysis will require judgment on several factors such as; complexity in geological and geometric conditions of the slope to be analyzed, availability of required parameter data for analysis, purpose for which slope stability analysis has to be made, limitations and effectiveness of an analytical method, availability of computational facilities and the capacity and skill of an evaluator.

2.8. Limit equilibrium method

In Limit equilibrium methods investigate the equilibrium of a soil mass tending to slide down under the influence of gravity. Transitional or rotational movement is considered on an assumed or known potential slip surface below the soil or rock mass. The limit equilibrium is statically indeterminate analysis. As the stress strain relationship along assume surface are not known, so necessary that system becomes statically determinant and it can be analyzed easily using the equation of equilibrium (Salunkhe, 2017). Following assumption are generally made, (Arora, 2008)

a. The stress system is assumed to be two-dimensional. The stresses in the third direction (perpendicular to the section of the soil mass) are taken as zero.
b. It is assumed that the column equation for shear strength is applicable and the strength parameters c and φ are known.

c. It is further assumed that the seepage conditions and water level are known, and the corresponding pore water pressure can be estimated.
d. The condition of plastic failure as assumed to be satisfied along the critical surface in other word shearing strains at all points of the critical surface are large enough to mobilize all the available shear strength.
e. Depending upon the method of analysis some additional assumptions are made regarding the magnitude and distribution of forces along various planes.

A state of limit equilibrium is obtained by expressing the mobilized shear stress (τ) as a fraction of the shear strength (τf). The available shear strength depends on the type of soil and the effective normal stress, whereas the mobilized shear stress depends on the external and internal forces acting on the soil mass. Hence, limit equilibrium analysis defines FS as a ratio of shear strength (τf) to mobilized shear stress (τ) (BRAJA, 2010).

Shear strength available (τf) = $c' + \sigma' \tan\emptyset'$…………………….……Equation 4

Mobilized shear stress (τ) = $\frac{\tau f}{FS} = \frac{c'+\sigma' tan\emptyset'}{FS}$…………………………..Equation 5

All LE methods are founded on certain assumptions for the shear and interslice normal forces, and what makes them unlike is how these forces are determined or presumed. The slope stabilityanalysis in the limit equilibrium method is through iterative aspect till the critical slip surface isfound out, where the slip surface with the lowest FS is the critical slip surface.

Slope stability analysis in limit equilibrium can be performed by mass procedure and methodof slices. The mass procedure is used when the soil that forms the slope is assumed to behomogeneous which considers the mass of the soil above the surface of sliding is as a unit (Braja, 2010). In the method of slices, the soil above the surface of sliding is divided into several vertical parallel slices as shown in Figure 3 and the stability of each slice is calculated separately. This is a versatile technique as the non-homogeneity of the soils, pore water pressure, and the variation of the normal stress along the potential failure surface can be taken into consideration.

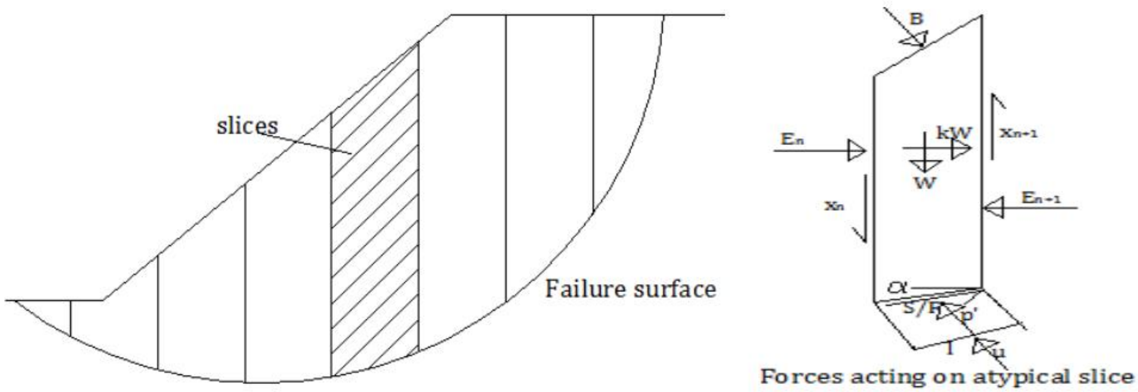


Figure 2.3: The method of slice and forces acting on a typical slice (Burman et al., 2015)

As shown in figure above the forces acting on a typical slice include the weight of slice (W), seismic force applied to the center of a slice (*kW*), mobilized shear force at the base of a slice (*S*/*F*), effective normal forces on base (*P*'), water pressure force on base (*U*), resultant top boundary force (*B)*, vertical side force (*X)* and horizontal slide force (*E*). There are different methods of LE analysis proposed by scholars and their basic difference is the way they are determining or assumed the interslice normal and shear forces.

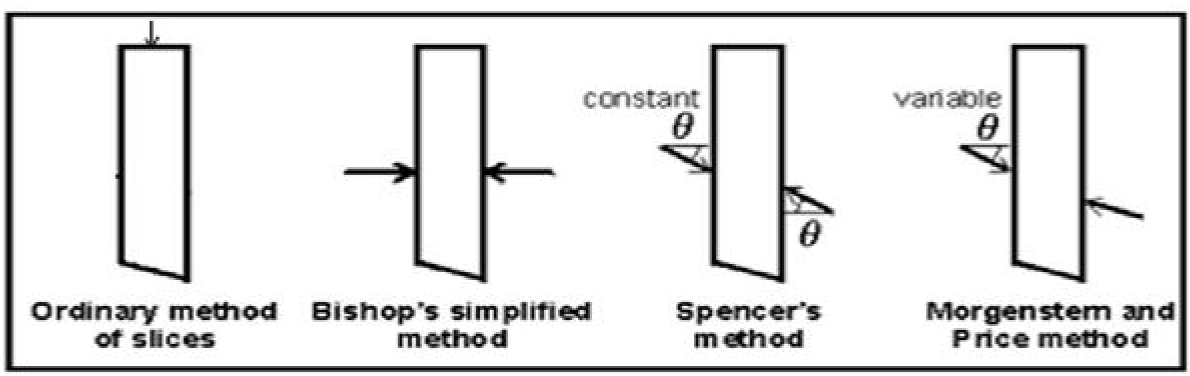
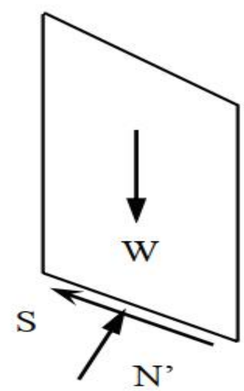
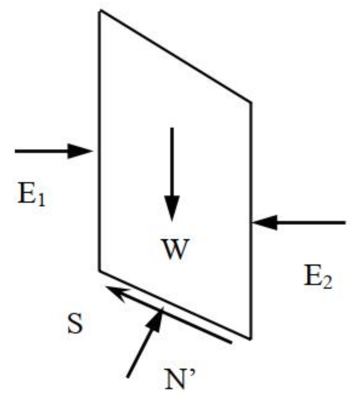
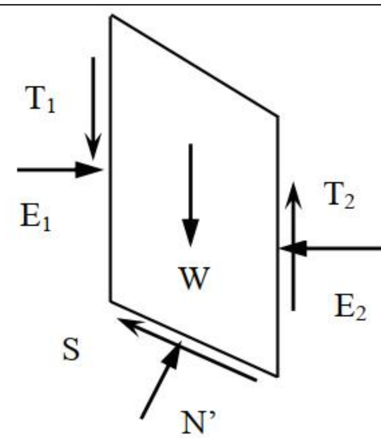


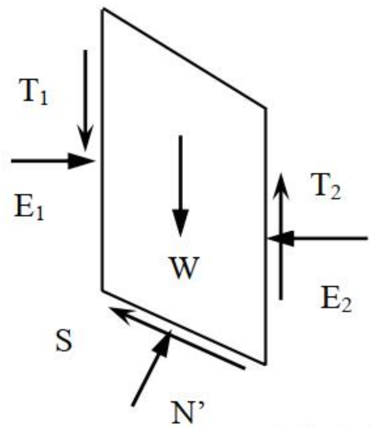
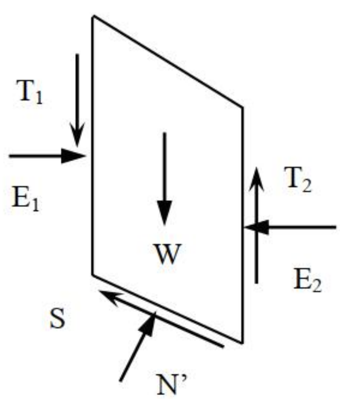
Figure 2.4: Differences in assumptions regarding side forces in common methods of slope (Burman et al., 2015)

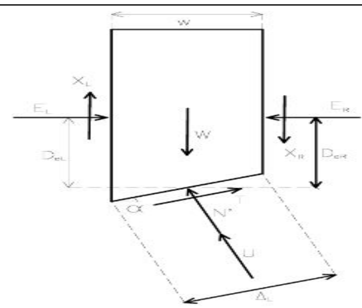
Below is summary of the method of slices while working with limit equilibrium method.

Table 2.4 : Summary of some LE methods (Abramson et al., 2001; Aryal, 2006)

Method of slices	**The ordinary method of a slice (Fellenius, 1927)**
Detail	•Moment equilibrium about the center of the circle. •neglects the interslice normal and shear forces, gives the most conservative FOS, and is useful only for demonstrations.
Slip surface	Circular slip surface
Use	•Applicable to non-homogeneous slopes and c-ø soils where slip surface can be approximated by a circle. •Very convenient for hand calculations. •Inaccurate for effective stress analyses at high pore water pressures.
Sketch	
Method of slices	**Bishop's Modified Method (Bishop, 1955)**
Detail	• satisfies moment equilibrium for FOS, • satisfies vertical force equilibrium for N, •considers interslice normal force, • more common in practice, and • applies mostly for circular shear surfaces.
Slip surface	Circular
Use	•Applicable to non-homogeneous slopes and c-ø soils where slip surface can be approximated by a circle. • More precise than the conventional Ordinary Method of slices (Fellenius, 1927), mostly for analysis with high pore water pressures. •Calculations are feasible by hand or spreadsheet

Sketch	
Method of slices	**Janbu's Generalized Procedure of Slices (Janbu, 1968)**
Detail	•Force equilibrium (vertical and horizontal). considers both inter slice forces, • assumes a line of thrust for inter slice forces, • satisfies both force and moment equilibriums, • handles complex geometry and failure surfaces, • is an advanced method among LE methods
Slip surface	Any shape
Use	•Applicable to non-round slip surfaces. Also, for shallow and long planar breaking surfaces that are not parallel to the ground surface.
Sketch	
Method of slices	**Morgenstern & Price's Method (Morgenstern & Price's, 1965)**
Detail	• All conditions of equilibrium. • considers both inter slice forces, • assumes a inter slice force function, f(x), • allows selection for inter slice force function, • computes FOS for both force and moment equilibrium

Slip surface	Any shape
Use	• A precise procedure that can be applied to almost all slope geometries and soil profiles. • Rigorous and complete equilibrium procedure.
Sketch	
Method of slices	**Spencer’s Method (Spencer, 1967)**
Detail	•All conditions of equilibrium. • considers both inter slice forces, • assumes a constant inter slice force function, and • satisfies both moment and force equilibrium, and • computes FOS for force and moment equilibrium
Slip surface	Any shape
Use	•A precise procedure that can be applied to almost all slope geometries and soil profiles. •The simplest complete equilibrium procedure for computing FS.
Sketch	
Method of slices	**Generalized Limit Equilibrium (GLE) (Ferdlund et al., 1981)**

Detail	•Satisfies both force and moment equilibrium. • considers both inter slice normal and shear forces,
Slip surface	Both circular and non-circular
Use	• allows selection for inter slice force function, and • shows comparison of most common and advanced LE methods • Encompasses most of the assumptions used by various LEM.
Sketch	

2.9. Finite element method

Conventional limit equilibrium techniques are the most commonly used analysis methods. But recently, the considerable computing and memory resources available to the geotechnical engineer, combined with low costs, have made the finite element method (FEM) an influential and viable alternative. The ability to analyze both linear and nonlinear problems, ability to analyze any geometry of slope in two and three-dimension, ability to give FS without having assumptions about the shape and position of the sliding plane, the ability to employ various soil material models, and its ability to consider the stress-strain relationship of the material make the FEM preferable analysis over conventional limit equilibrium method (Burman et al., 2015).

There are two methods of slope stability analysis using the finite element method (i.e. Stress increase and shear strength reduction method) (Chen, 2018). Gravity force increased until the slope fails in the stress increase method and FS is determined as a ratio of gravitational acceleration at the failure of the actual gravitational acceleration. In the shear strength reduction method, the soil strength parameters are reduced until the slope becomes unstable and FS is determined as a ratio of the initial strength parameter to the critical strength parameter. In the SSR method, the FS of a slope is defined as the factor by which the original shear strength parameters must be divided to bring the slope to the point of failure (Burman et al., 2015). The strength reduction factor (SRF) is defined as:

$$c'f = \frac{c'}{SRF} \text{ and } \emptyset' f = \tan^{-1}\left(\frac{\tan\emptyset'}{SRF}\right) \text{..............Equation 3}$$

Where $\emptyset' and c'$ are original shear strength parameters, $c'f$ and $\emptyset'f$ is reduced shear strength parameters and SRF is a strength reduction factor its value at the failure is equal to the factor of safety of slope. The method has the same behavior as the Mohr-Coulomb model where a constant stiffness modulus is used (Chen, 2018).

2.10. Some related previous works

Table 2.5 : Some previous works of different authors

Author	**Samuel, (2017)**
Title	Slope stability analysis of rainfall-induced landslides on Abay gorge
Methods used	LEM slope stability analysis using SLOPE/W and pore water distribution is conducted using seep/w
Results and conclusion	The result shows a reduction of FS from 1.506 to 1.29 due to rainfall infiltration from its initial dry state. The FoS is dropped as low as to 1.4 on days of high rainfall.
Author	**Hammouri et al., (2008)**
Title	Stability analysis of slopes using the finite element method and limiting equilibrium approach
Methods used	SSR approach and Mohr-Coulomb failure criteria is used to analyze the slope using LEM and FEM (SAS-MCT 4.0 versus PLAXIS 8.0 respectively)
Results and conclusion	The differences in the values of safety factors achieved for different conditions were found to be small and the critical slip surfaces obtained using both methods compared well. And recommended that the engineer should analyse critical slopes using both FEM and LEM.
Author	**Bushira et al., (2018)**
Title	Cut soil slope stability analysis along National Highway at Wozeka–Gidole Road, Ethiopia
Methods used	LEM-based geo-slope and FEM-based plaxis software are used
Results and conclusion	Except the ordinary methods, all LEM estimate higher FOS than FE analysis in plaxis. Numerical results of FEM and LEM verify the instability of the slope during the rainy season. The results of FOS also indicate that the slope is likely to fail even without reaching to the fully saturated condition which confirms the actual conditions.

Author	**Shiferaw, (2017)**
Title	Slope Stability Analysis and Mitigation Measures of the Agoro – Shahgubi Section of the Agulae - Berhale Road, Northern Ethiopia
Methods used	Limit equilibrium based software SLOPE/W
Results and conclusion	Cut Slope stability analysis was conducted for the moist conditions using 2D Limit equilibrium method using SLOPE/W. the safety factors calculated by slope/W utilizing the Morgenstern Price methods, Ordinary method, modified Bishop Method and Janbu method shows the slope is failed.
Author	**Abrha,(2020)**
Title	Geotechnical Characterization and Slope Stability Analysis of Landslide; The Case of Werie-Maykinatal Road, Tigray Region, Northern Ethiopia
Methods used	The slope model was analyzed using SLOPE/W software
Results and conclusion	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.
Author	**Beyene, (2017)**
Title	Comparison of Finite Element and Limit Equilibrium Methods for Slope Stability Analysis
Methods used	Limit equilibrium method Slope/W and finite equilibrium method PLAXIS was used
Results and conclusion	Since the finite element approach can produce results with lower values of the factor of safety than the majority of other types of limit equilibrium methods, slope stability analysis can be performed using a safer outcome than limit equilibrium analysis.

2.11. Slope instability stabilization method

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. 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 et al.,(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.

1. Slope Reduction: 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.
2. Slope Modification: Modification of a slope either by humans or by natural causes can result in changing the slope angle so that it is no longer at the angle of repose.
3. Engineering Structures for Mitigation: 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.
4. Retaining Structures: These structures are basically constructed at base and toe of the sliding surface since it increases the resistance to movement.
5. Stabilization by Drainage: stabilization by drainage has been noted as a very effective means of protecting unstable hill slopes and from further sliding.
6. Stabilization Using Vegetation: Planting with shrubs adds vegetative cover and stronger root systems, which in turn will enhance slope stability.
7. Rock slope stabilization methods: These include altering the slope geometry, installing drainage, adding reinforcement, or a using combinations of these methods
8. Reinforcement systems: These includes rock anchors, tensioned anchors, rock bolts and tension rock bolts.

But most of all avoiding any kind of construction is better but if not, necessary stabilization methods must be conducted.

CHAPTER THREE

3. MATERIALS AND METHODS

3.1. General

Over the years, numerous different solutions methods have been developed for slope stability analysis. One of the earliest types of numerical analysis in geotechnical engineering is the analysis of slope stability. The Limit Equilibrium Method and the Finite Element Method have been compared in this research.

Three distinct soil samples from the route that runs from Addisu Gebeya to Sululta, or more accurately from Ftesha to Dil Ber, have been chosen for study using both methodologies which are limit equilibrium and finite element methods. This road is located in the central part of Ethiopia and it is in the northern Addis Ababa zone. The local environmental conditions that exist promote the formation of different kinds of soil. A region's geology, climatic circumstances, and geographic layout all have an effect on how a soil mass forms. Identification of the study area's description, climate, population, geology, and soil characteristics is necessary.

3.2. Description of study area

Although the road connects sululta and addisu gebeya and many more cities it is mainly known for being the Addis Ababa – Bahirdar road. The location considered is in a district located in northern suburb of the city, near the Mount Entoto and Entoto Natural Park. It borders with the districts of Kolfe Keranio, Addis Ketema, Arada and Yeka.

3.2.1. Location and population of study area

The position of Gulele subcity is N9^{o}4’5.1312”, E38^{o}44’.398” (9.0680916,38.7420548) and the elevation the study area the altitude ranged from 2449 to 3016 meters above sea level. It has a population of 377,032 as per study done in 2022 on 30.18 km² Area.

3.2.2. Climate of study area

If elevation were not taken into account, the climate would be maritime because no month has mean temperatures above 22 °C (72 °F) (Metro Climate, 1961-1990, 1951-1990, and 1985-1998).

The Gulele sub city's long-term mean annual maximum and minimum temperatures are 22.8 and 10.6 degrees Celsius, respectively, and its long-term mean annual rainfall is 1180.4 millimeters.

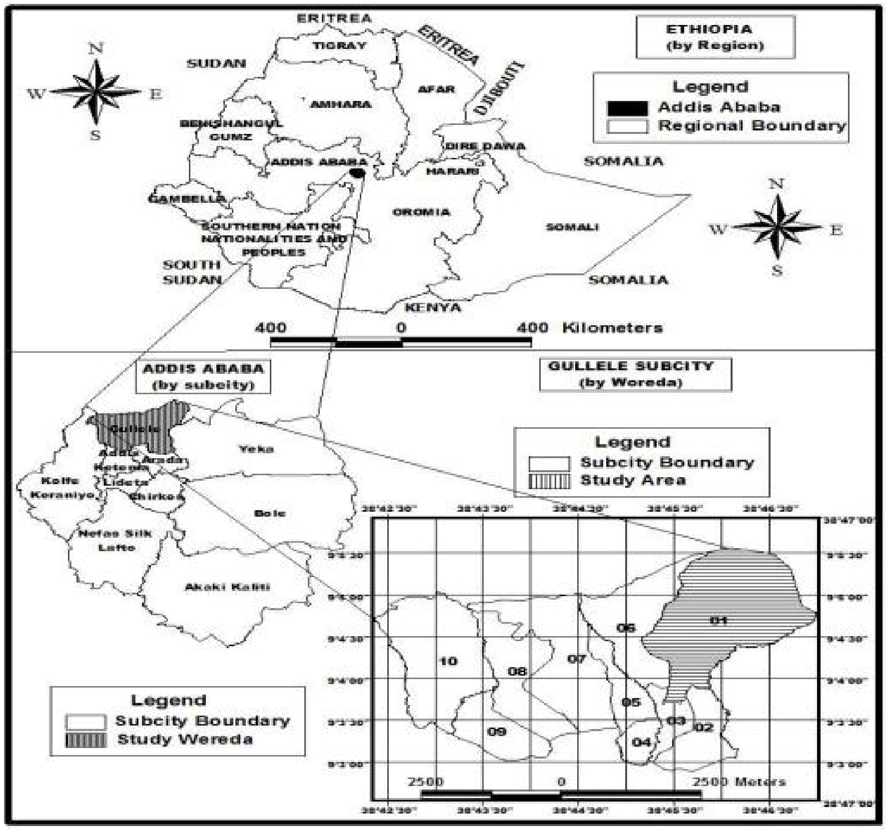


Figure 3.1: Map of study area

3.2.3. Topography and drainage of study area

Topography is the graphic fine art of precisely illustrating both the man-made and natural aspects of a location or region on maps or charts. Gulele Sub City is distinguished by various topographical features, including obvious elevation changes and steep terrain, particularly in the northern region. The research area's elevation, which has a range of 567 meters from 2449 to 3016 meters above sea level Gulele cut through with fast flowing streams. Geotechnical data from 1996 to 2014 shows red clays in Addis Ababa are extensively distributed around Gulele sub-city, Burayu and part of Kolfe Keraniyo specifically around Atanatera. Some part of Addis Ketema and Arada sub-cities are also covered with red clays(Akalu, 2017).Residual red soils mainly located in Gulele and Kolfe area from Sample tested provide grain size of 62% clay, 33% silt and 5% sand (Akalu, 2017).

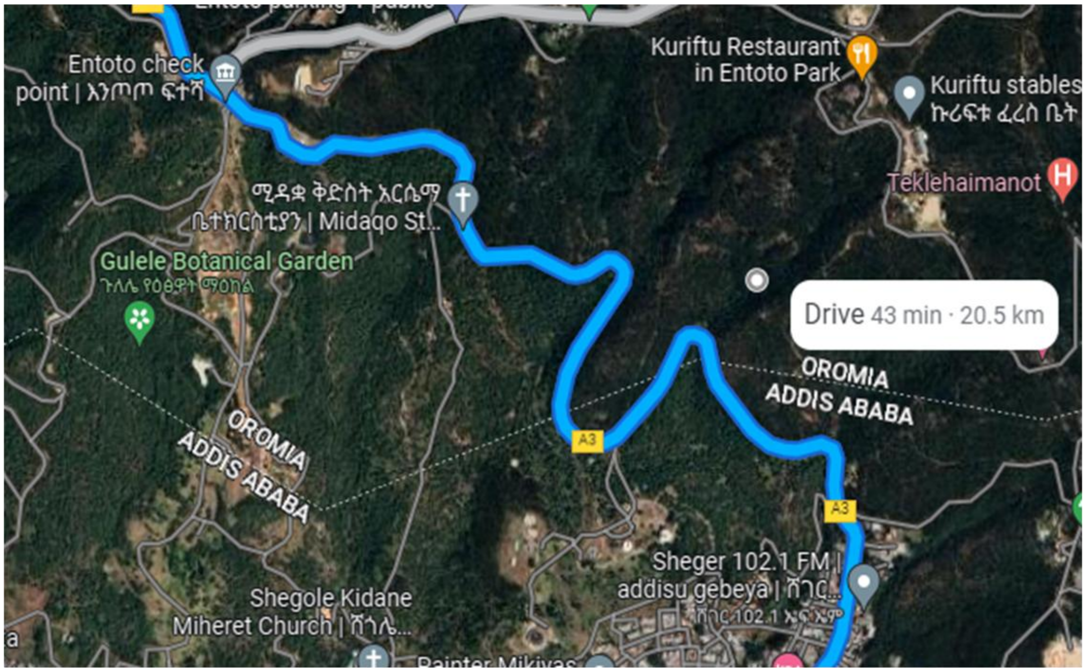


Figure 3.2: Blue line shows road from Entoto check point to Dilber

3.3. Methodology

3.3.1. Desk study

Review of different literature on the types, causes, and effects of slope stability in the world and in Ethiopia was done in order to have a knowledge about slope stability. A survey of published and unpublished geological maps, academic journals, and various methods and theories used to analyze slope stability along road sections and about their mitigation measures. Information was obtained from the area and a field reconnaissance survey was conducted in order to learn the overall slope stability conditions in the research area. Field observation and data preparation forms the important part of any scientific research.

The research was conducted by using both experimental and analytical methods. Field investigation, laboratory testing, and software analysis were used in this research. Quantitative primary data were collected through the use of GPS in the field, location of sample recorded using GPS coordinates application, laboratory analysis of representative soil samples to determine material properties, and input parameters for software analysis. Secondary data came from a variety of literature reviews.

Figure 3.3:Photos of study area from field investigation

3.3.2. Field investigation

Visual inspections for the road section from Ftesha to Dilber were made and the failed slope sections were identified and 3 locations on where more failure observed were selected for further investigations. The field survey included description of soils, documentation of slope failure affected sites (using GPS), test pit excavations and sampling of soils for laboratory analyses. When visiting the site, it was possible to observe surface (localized) erosion, slope failures, ground cracking, ground subsidence, damage to natural features, exposing of plant roots and tilting of plants. Some photos from field observation have been attached below.

3.3.3. Field Sampling and Laboratory Test Preparation

Sampling of soils took place in order to determine physical characteristics. Disturbed samples were taken from the 3 most failed slope. Depending on the geology and other important factors, samples were gathered from the road sides in regions where slides had previously occurred on failing slopes. In order to preserve the sample's moisture, samples were taken from the sampling region while they were still moist using plastic bags in order to have the in situ behaviour of soil.

The sampling was done using the traditional excavation equipment’s like Doma, Akafa, Geso and Bucket in order to remove the soil from the bottom of the pit with the help of local workers.

Figure 3.4:Traditional excavation equipment’s used for excavation

The sample information has been recorded carefully in order to avoid confusion and mix up. The sample was taken up to a depth of 1.5m in order to collect a representative sample of the soil profile after trimming the top soil in order to avoid the organic and loose soil on the top layer while carefully observing if there is any kind of layer on the soil every 20cm but the soil was homogeneous throughout the 1.5m depth. The sampling coordinates are provided in a table 3.1.

The extracted sample was air dried after separating the amount to determine the moisture content in order to proceed to the other laboratory tests.

Table 3.1: Location, type and depth of sample at the study area

S.NO	Sample code	Depth	Sample type	Location		
				Latitude	Longitude	Elevation
1	TP-1	1.5m	Disturbed sample	9.083318 N9°4'59.94444"	38.726759 E38°43'33204"	9199ft 2803.8m
2	TP-2	1.5m	Disturbed sample	9.080390 N9°4'49.40328"	38.730920 E38°43'51.3102"	9071ft 2764.8m
3	TP-3	1.5m	Disturbed sample	9.078056 N9°4'41.00088"	38.731496 E38°43'53.38524"	9048ft 2757.83m

Figure 3.5: Test pit locations on map

Figure 3.6: Test pit photos

Figure 3.7: Air drying the sample

3.4. Laboratory tests conducted on sample

To identify and characterize the problem nature of the slope material for slope stability, a range of laboratory analysis were carried out. The following laboratory tests were analyzed for investigation of geotechnical characteristics and slope stability analysis.

3.4.1. Moisture Content

This test was performed to determine the moisture content of soils. The moisture 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. 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.

3.4.2. Specific gravity

Specific gravity is a fundamental property of soils and other construction materials. This dimensionless unit is the ratio of material density to the density of water and is used to calculate soil density, void ratio, saturation, and other soil properties. This test was 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 which is 20^oC. ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by density bottle was used in order to conduct the test.

3.4.3. Atterberg's limit

The objective of this experiment is to determine the liquid limit (LL), plastic limit (PL), and the plasticity index (PI) of fine-grained soils. This test was conducted using the disturbed samples in accordance with ASTM D 4318 to determine the plastic, liquid limits and plasticity index of a fine grained soils from study area. 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 or more from the cup being dropped 10 mm in a Casagrande 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 and the plasticity index was determined by subtracting plastic limit from the liquid limit.

3.4.4. Grain size distribution

The grain size analysis test was performed to determine the percentage of each size of grain that is contained within the soil sample, and the results of the test was used to produce the grain size distribution curve. This information was used to classify the soil and to predict its behavior. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, while the hydrometer method is used to determine the distribution of the finer particles according to ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils. The hydrometer analysis of soil, based on Stokes' law, calculates the size of soil particles from the speed at which they settle out of suspension from a liquid. Results from the test show the grain size distribution for soils finer than the No. 200 (75µm) sieve. However, when combined with a sieve analysis, offer a complete gradation profile of soils containing coarser materials. Therefore, it was necessary to conduct hydrometer test for the sample.

3.4.5. 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 was used and this was done by forcing the soil to shear along an induced horizontal plane of weakness at a constant rate. Direct shear test is generally conducted on cohesion less soils and it is convenient to perform and it gives result for the strength parameters which are the c and ϕ value for the selected slope materials has a contribution to analyze the stability of the slope.

3.4.6. Unit weight (Density)

Unit weight or density was used to quantify the weight per unit volume of the soil. When it is expressed in the basic SI Unit of mass (kg/m^3), it is usually referred to as density, but when expressed in terms of weight (kN/m^3), it is usually referred to as unit weight.

3.4.7. Free swell test

Free swell or differential free swell, also termed as “free swell index”, is the increase in volume of soil without any external constraint when subjected to submergence in distilled water for a specified period and calculating the percentage increase in volume. The objective of a swelling pressure test on soil is to determine the swelling pressure of expansive soil when it is not allowed to undergo any volume change. The volume change is arrested or the soil is not allowed to swell in order to test this.

3.4.8. Compaction

The test aims to establish the maximum dry density that may be attained for a given soil with a standard amount of compaction effort. When a series of soil samples are compacted at different water content, the plot usually shows a peak. The Modified Proctor Test for soil (ASTM D1557) was conducted for the sample which is a modified version of the Standard Proctor Test that uses a heavier compaction effort and a larger mold. It was developed to better simulate the compaction effort of heavy machinery used in the field. It is generally more accurate than the Standard Proctor Test, as it better simulates the actual conditions under which the soil will be compacted.

3.5. Computer program (software’s) used for slope stability analysis

Computer software used for slope stability analysis plays a crucial role in geotechnical engineering and the assessment of risk associated with natural slopes, manmade slopes, dams, embankments, and foundations. The primary objective of these software programs is to provide engineers with a reliable and efficient way to analyze the stability of soil and rock slopes under different environmental conditions. These software helps engineers to predict potential failure mechanisms of slope and evaluate their factor of safety.

Nowadays there are several computer based geotechnical software’s used in the slope stability analysis. Whether it is working by limit equilibrium method or finite element method. It is important to note that each software program has its own strengths and weaknesses, and the best option will depend on the specific need and goal of the user.

3.5.1. SLOPE/W

The industry-leading slope stability software, SLOPE/W, analyzes both simple and complex slope stability problems for a variety of slip surface shapes, pore-water pressure conditions, soil parameters, and loading situations. It is used for slopes made of soil and rock. SLOPE/W software was used to evaluate the slope model in order to determine the slopes' current state and their factor of safety using the Limit Equilibrium Method (LEM).Abramson et al., (2002) stated that slices approach is widely utilized by many computer programs because it can accommodate the geometry of complex slopes, various soil conditions, and the influence of external boundary loads. The main idea behind the slices approach is that the potential sliding mass is divided into numerous vertical slices, and that each slice's equilibrium may be measured in terms of forces and moments. This would make estimating the permitted safety factor of a slide mass simple. In this study, three soil samples were taken and their respective cohesion and angle of friction for analysis was obtained from shear strength test for slope stability analyses.

3.5.2. PLAXIS

Plaxis is a finite element program used in geotechnical applications to simulate the behavior of the soil using soil models. The program can model different layers of soil and rock, buildings, construction stages, loads, and boundary conditions. Plaxis is a finite element software program created for the 2D or 3D analysis of deformation, stability, dynamics, and groundwater flow in geotechnical engineering. Advanced constitutive models are needed for geotechnical engineering applications to simulate the non-linear, time-dependent, and anisotropic behavior of soils and/or rock. A particular approach is needed to deal with pore pressures and (partial) saturation in the soil since soil is a multi-phase material. Although soil is a significant concern in and of itself, many geotechnical projects also take into account the interaction between the structures and the soil through structural modeling.

CHAPTER FOUR
4. MATHEMATICAL MODELLING

4.1. Slope/W

A possible mathematical model for slope stability software can be based on the concept of limit equilibrium analysis. This approach assumes that a slope will fail if the driving forces (e.g., gravity) exceed the resisting forces (e.g., shear strength).

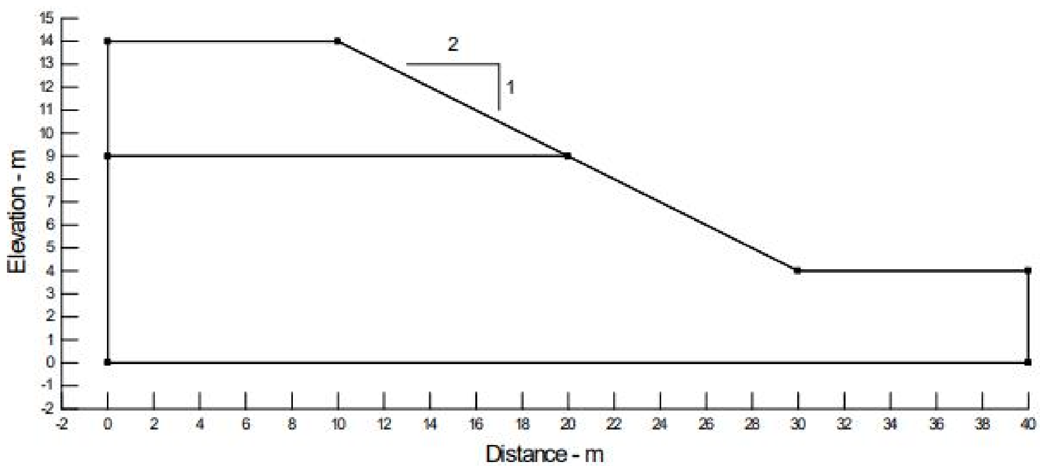


Figure 4.1: Sample model for Slope/W

The model can include the following variables:

1. Geometry of the slope: This includes parameters such as slope angle, height, width, and shape.
2. Material properties: Parameters related to the soil or rock properties, such as cohesion, friction angle, unit weight, and groundwater conditions.The strength parameters c and Ø can be total strength parameters or effective strength parameters. Slope/W makes no distinction between these two sets of parameters. Which set is appropriate for a particular analysis is project specific, and is something the software user, need to decide. The software cannot do this. From a slope stability analysis point of view, effective strength parameters give the most realistic solution, particularly with respect to the position of the critical slip surface.
3. External forces: Factors that influence the stability, such as applied loads, water pressure, and seismic forces.
4. Factor of safety (FoS): The ratio of resisting forces to driving forces. A FoS less than 1

indicates instability, while a FoS greater than 1 indicates stability.

Based on these variables, the mathematical model can be formulated as follows:

1. Calculate the driving forces acting on the slope using principles of statics and mechanics, considering the weight of the soil mass and any applied loads.
2. Determine the resisting forces by analyzing the shear strength of the soil or rock mass. This can be done using various methods such as Mohr-Coulomb theory or Hoek-Brown criterion.

The most common way of describing the shear strength of geotechnical materials is by Coulomb’s equation which is:

$$\tau = c + \sigma n' \tan\varphi$$

3. Calculate the factor of safety (FoS) by dividing the resisting forces by the driving forces. The FoS can be calculated for different failure modes such as sliding, overturning, or bearing capacity failure.
4. Evaluate the stability of the slope by comparing the calculated FoS with a predefined threshold value. If the FoS is greater than the threshold, the slope is considered stable; otherwise, it is considered unstable.
5. Sensitivity analysis: Perform sensitivity analysis to assess how changes in input parameters (e.g., slope angle, material properties) affect the stability of the slope.

This mathematical model forms the basis for slope stability software, which allows engineers to input the necessary parameters and obtain a quantitative assessment of slope stability.

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces in order for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a factor of safety. Using the analysis method explained in the literature review an analysis will be carried out using the ordinary methods of slice, the Spencermethod, the bishops method, the janbus’s method and the morgenstern & price method for limit equilibrium method by slope/w program.

4.2. Plaxis 2D

A possible mathematical model for PLAXIS 2D stability software can be based on the concept of finite element analysis. This approach involves discretizing the slope into a mesh of finite elements and solving the equilibrium equations for each element.

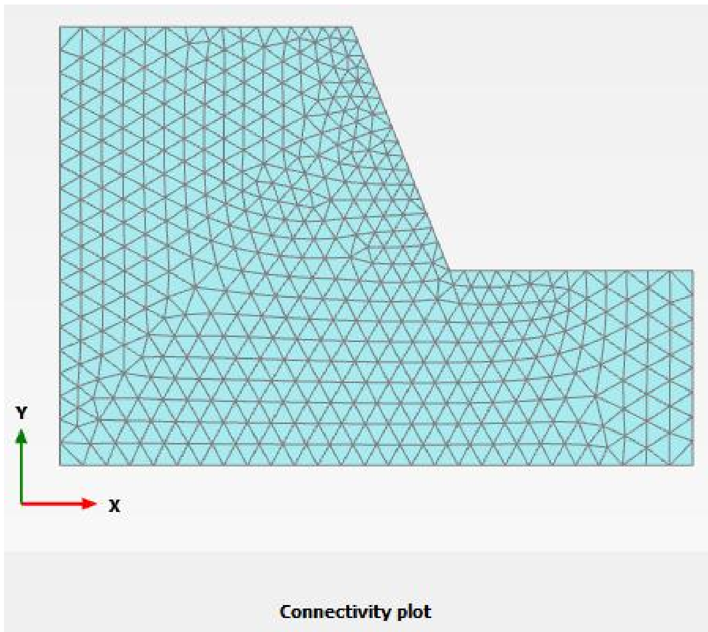


Figure 4.2: Sample model by Plaxis 2D

The model can include the following variables:

1. Geometry of the slope: This includes parameters such as slope angle, height, width, and shape. These parameters are used to define the geometry of the finite element mesh.
2. Material properties: Parameters related to the soil or rock properties, such as cohesion, friction angle, unit weight, and groundwater conditions. These parameters are assigned to each finite element in the mesh.
3. External forces: Factors that influence the stability, such as applied loads, water pressure, and seismic forces. These forces are applied to the finite element mesh.
4. Boundary conditions: Constraints applied to the model, such as fixed boundaries or prescribed displacements. These conditions are used to simulate the interaction between the slope and its surroundings.

Based on these variables, the mathematical model can be formulated as follows:

1. Discretize the slope into a mesh of finite elements. The size and shape of the elements can be chosen based on the complexity of the slope geometry.The powerful 15-node element provides an accurate calculation of stresses and failure loads.
2. Assign material properties to each finite element in the mesh. These properties can be obtained from laboratory tests or empirical correlations.
3. Apply external forces to the finite element mesh. These forces can be applied as point loads, distributed loads, or pressure loads.
4. Define boundary conditions for the model. These conditions can be used to simulate fixed boundaries or prescribed displacements at specific locations.
5. Solve the equilibrium equations for each finite element in the mesh using numerical methods such as the finite element method or the finite difference method.
6. Calculate the factor of safety (FoS) for each element by comparing the driving forces (e.g., shear stress) with the resisting forces (e.g., shear strength). The FoS can be calculated using failure criteria such as the Mohr-Coulomb theory or the Hoek-Brown criterion.
7. Evaluate the stability of the slope by analyzing the FoS distribution across the mesh. If the FoS is greater than a predefined threshold for all elements, the slope is considered stable; otherwise, it is considered unstable.
8. Perform sensitivity analysis to assess how changes in input parameters (e.g., material properties, boundary conditions) affect the stability of the slope.

This mathematical model forms the basis for PLAXIS 2D stability software, which allows engineers to input the necessary parameters and obtain a quantitative assessment of slope stability.The factor of safety in PLAXIS was computed using Phi-c reduction at each case of slope modeling. In this type of calculation, the load advancement number of steps procedure is followed. The incremental multiplier Msf is used to specify the increment of the strength reduction of the first calculation step. The strength parameters are reduced successively in each step until all the steps have been performed.

CHAPTER FIVE

5. RESULTS AND ANALYSIS

5.1. Laboratory test result and analysis

5.1.1.1.Moisture content

Natural water content is the in-situ water content of soils determined using freshly excavated soil samples without exposing them to air. On the basis of the ASTM D 2216 Standard test technique, the test was carried out on fresh soil samples that were gathered from three distinct locations. The location experienced a high rainfall and the soil sample extracted will reflect the actual condition. It can be seen from the below result that soil sample collected from TP- 2 has higher water content and it even took longer time to be air dried for other test procedures. It holds the water within and appears wet for longer time. Although TP-1 and TP-3 were somewhat similar in color TP-1 was found very wet and slippery as compared to TP-1.

Table 5.1: Natural moisture content values

Test pits	Depth	NMC(%)
TP-1	1.5m	46
TP-2	1.5m	68
TP-3	1.5m	32

5.1.1.2.Specific gravity

A measurement of soil density relative to a certain volume of freshwater is its specific gravity. A pycnometer was used to calculate the specific gravities of various soils using ASTM D 854-00 as the standard.

According to Atkinson, (2004) the specific gravity of clay and silty clay ranges from 2.67-2.9 which shows that the soils in the area are generally fine grained which does not have the strength to withstand slope failure.

Table 5.2:Specific gravity results of soil samples

Test pits	Depth	Specific gravity(Gs)
TP-1	1.5m	2.832
TP-2	1.5m	2.597
TP-3	1.5m	2.747

5.1.1.3.Atterberg’s limit

For the determination of Atterberg limits, soil samples that passed through a 425-micrometer sieve were in use. The Casagrande apparatus was used to calculate the liquid limit (LL). A soil sample was rolled in 3mm thread until it started to disintegrate in order to determine the plastic limit (PL). The difference between the liquid limit (LL) and the plastic limit (PL) is called the plasticity index (PI).The Casagrande plasticity chart is based on the plasticity index, which is crucial in identifying fine-grained soils. The more plasticity index there is, the more engineering issues there will be with employing soil as a construction material. The soil material has a liquid limit range from45% to 64% as it can be seen from the results while plasticity ranges from 18% to 21% and these results can be plotted on the plasticity chart.

TP-1 and TP-2 range in high plasticity ranges while TP-3 ranges in low plasticity.

Table 5.3: Liquid limit, Plastic limit and Plasticity Index of soil samples

Test pit	Depth (m)	LL (%)	PL (%)	PI (%)
TP-1	1.5	54	33	21
TP-2	1.5	64	45	19
TP-3	1.5	45	27	18

5.1.1.4.Grain size distribution

The particle-size distribution of the soil samples was determined using two different techniques: sieve analysis for particles bigger than 0.075 mm (No. 200) in diameter, and hydrometer analysis for particles smaller than 0.075 mm in diameter. Sodium hexametaphosphate (NaPO3) was utilized as a dispersion agent throughout the hydrometer study, and the ASTM D422 method was followed for the analysis. The following diameter or size range is taken directly from ASTM: >

4.75 mm (No. 4) gravel, 0.075 mm (No. 200) sand, 0.075 mm (No. 200) to 0.002 mm silt, and 0.002 mm clay.

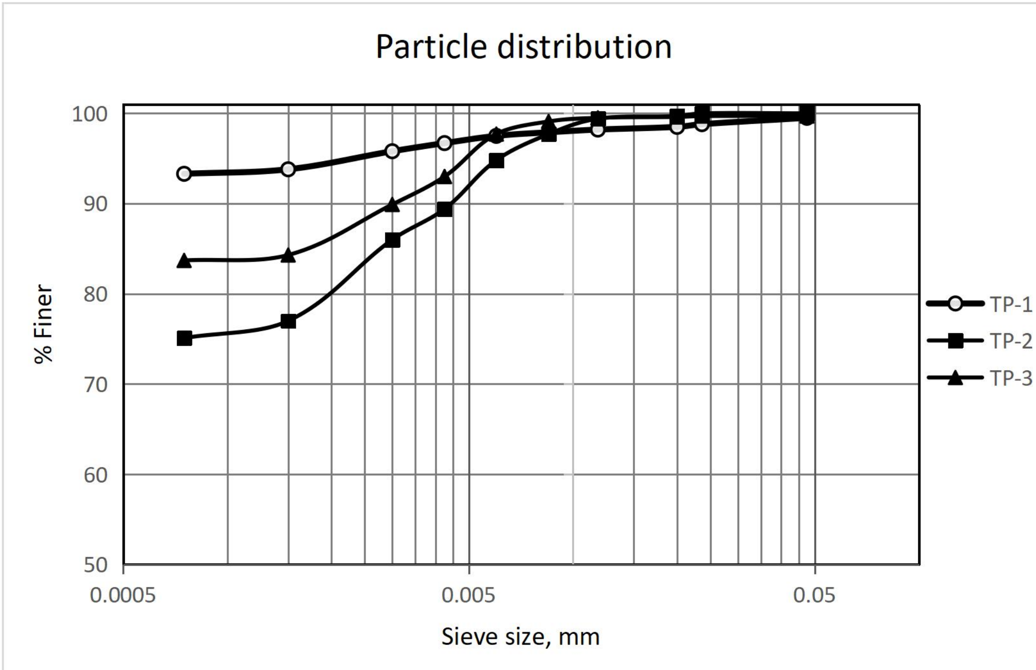


Figure 5.1: Particle distribution plot

TP-1 on the first test pit on 4.75mm sieve 0.5% retained while 99.5% have passed while on 0.075mm sieve 6.7% have been retained and 93.3% have passed.

TP-2 on the second test pit on 4.75mm sieve 0% retained while 100% have passed while on 0.075mm sieve 24.9% have been retained and 75.1% have passed.

TP-3 on the third test pit on 4.75mm sieve 0.3% retained while 99.7% have passed while on 0.075mm sieve 16.3% have been retained and 83.7% have passed. More information on sieve analysis will be attached on APPENDIX 1.

From the above results we can see that the soil sample contains finer particles having big percentages therefore a hydrometer was conducted.

Table 5.4: Particle size analysis (Hydrometer) summary

No.	Sample	Depth (m)	Gravel% (>2.0mm)	Sand %(2.0mm-0.06mm)	Silt %(0.06mm-0.002mm)	Clay %(<0.002)
1.	TP-1	1.5	1	19	20	60
2.	TP-2	1.5	0	36	28	36
3.	TP-3	1.5	0	29	27	44

TP-1,TP-2 and TP-3 have clay content of 60%, 36% and 44% respectively. The results shows that TP-2 has highest percentage of sand particle, TP-3 highest percentage of silt and clay particle when compared to TP-2 but lower than TP-1.

5.1.1.5.Direct shear test

One of the shear strength tests used to assess the strength properties of samples of undisturbed or remolded soil materials is the direct shear test. For the purpose of defining the Mohr-Coulomb failure criteria, the direct shear test was used to establish the shear strength parameter internal angle of friction (φ) and cohesion(c). In order to conduct this research, three disturbed soil samples were tested using ASTM D 3080 on a chosen failure surface.

Table 5.5:Shear Strength parameter of soil samples

Test pit	Depth (m)	Cohesion, C (kN/m^2)	Angle of internal friction, ϕ (°)
TP-1	1.5	62	15
TP-2	1.5	37	22
TP-3	1.5	35	29

5.1.1.6.Unit weight (density)

Bulk density and unit weight of the sample soils of the study area are given below. The value of bulk density of the soils varies from 1.585 to 1.886 g/cm^3, unit weight 19.42 to 20.4 KN/m^3 and dry density 1.229 to 1.639g/cm^3.

Table 5.6: Density values of samples

Test pit	TP-1	TP-2	TP-3
Bulk density(gm/cm^3)	1.802	1.585	1.886
Unit weight(KN/m^3)	17.67	15.55	18.50
Dry density(gm/cm^3)	1.536	1.229	1.639

5.1.1.7.Free swell test

The swelling tendency was also determined from the samples passing 425-micrometer sieve and oven dried. The 10ml of soil sample was put in water for 24 hours and swelling was examined as a percentage of volume change to the original. Free swell test results indicate the potential expansiveness of soil samples without being loaded.

Table 5.7:Free swell test result

Test pit (TP)	Depth (m)	Free swell (%)
1	1.5	60
2	1.5	60
3	1.5	40

5.1.1.8.Compaction

Modified Proctor compaction test is used to provide a guide for specifications on field compaction to obtain the moisture-dry density relationship of the specific soil samples and used to simulate heavy compacting effort. The maximum dry density (MDD) and optimum moisture content (OMC) are determined. Detail results information has been attached on APPENDIX.

Table 5.8: Compaction results from soil samples

Test pit (TP)	Depth (m)	MDD (gm/cc)	OMC (%)
1	1.5	1.583	17.5
2	1.5	1.310	29.0
3	1.5	1.680	15.0

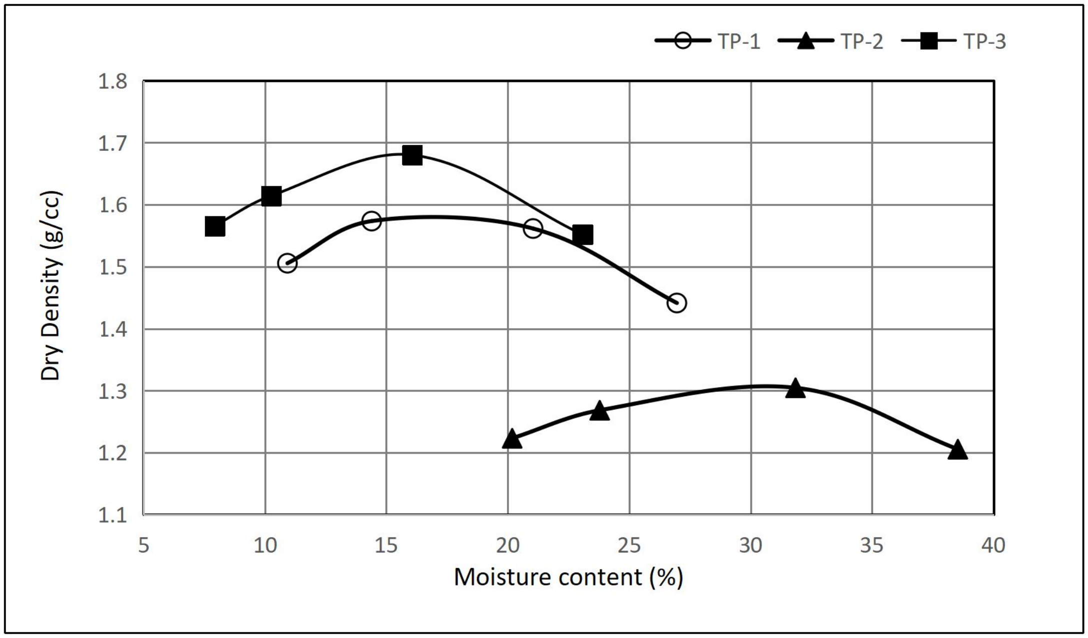


Figure 5.2:Moisture content vs Dry Density plot

The soil test summary has been summarized below.

Table 5.9: Soil test summary

TP	Depth	LL(%)	PL(%)	PI(%)	AASHTO soil class	Gs	NMC (%)	FS (%)
1	1.5m	54	33	21	A-7-5(15)	2.832	46	60
2	1.5m	64	45	19	A-7-5(16)	2.597	68	60
3	1.5m	45	27	18	A-7-6(12)	2.747	32	40

5.2. Software analysis and results

Stability of the three critical slopes was analyzed using the FEM and LEM for comparison and verification of the slope condition. Using the LEM and FEM software, factor of safety (FOS) and slope deformation were determined. The comparison was made using modeling data by Geo slope 2018 version 9.01(LEM based) and Plaxis 2D (FEM-based).

5.2.1. SLOPE/W

The factor of safety is determined from a two-dimensional equilibrium state such that the factor of safety is the ratio of resistance force or moment to driving force or moment.

$$\mathrm{FOS} = \frac{\text{resisting force(moment)}}{\text{driving force(moment)}}$$

The geometry was created using the Slope/W software.FOS was resolved using the Morgenstern-Price, Ordinary, Spencer, Bishop, and Janbu procedures. The necessary datas for Slope/W stability analysis are mentioned in the table below.

Table 5.10: Necessary inputs for SLOPE/W software

Slope (TP)	Slope (H:V)	Slope height (m)	Cohesion, C (kN/m^2)	Angle of friction(°)	Unit weight, ϒ (kN/m^3)
1	1:2.7	19	62	15	17.67
2	1:2.5	15	37	22	15.5
3	1:2.5 1:2.3	15 7	35	29	18.5

5.2.1.1.Stability analysis of critical section at TP-1

The critical slope section on TP-1 has resulted a factor of safety 1.281 using the default Morgenstern price method.

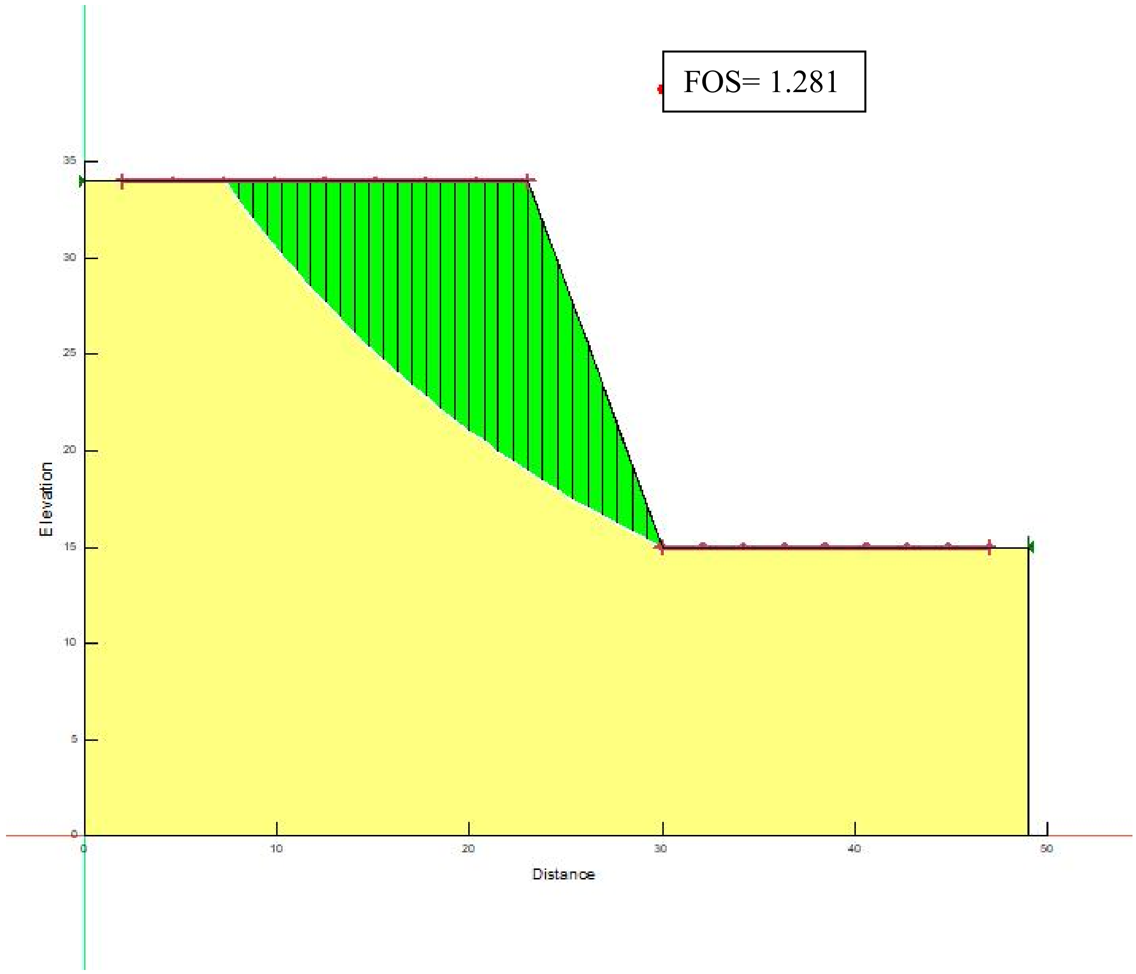


Figure 5.3:Slip surface for TP-1

This factor of safety which 1.281 shows that the slope is marginally stable since its result is greater than 1 but it could potentially fail under certain conditions and changes around the area. The slope still shows signs of potential instability therefore the slope should be monitored.

Figure 5.4 shows all the slip surfaces using different color map in order to show the possible factor of safety for each slip surfaces. Each color shows a different range of factor of safety on the slope from safe to possible failure.

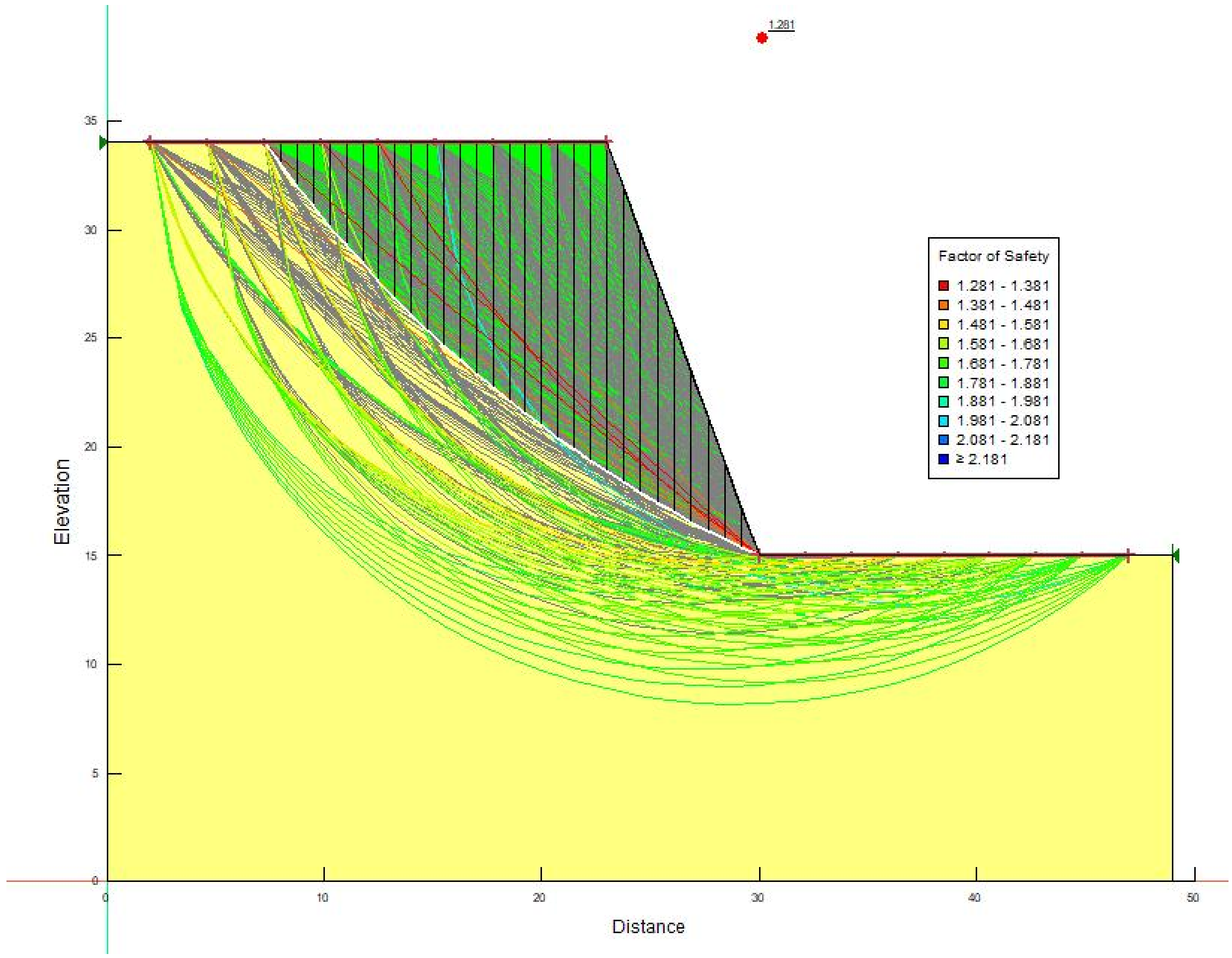


Figure 5.4: Slip surface color map of TP-1

Different limit equilibrium methods result different factor of safety for the same slope section because of the differences in their assumptions. Some of the methods and their respective values can be seen below.

Table 5.11: Different limit equilibrium methods and their FOS results for TP-1

MP method	Spencer method	Bishop Method	Janbu method	Ordinary Method
1.28	1.318	1.212	1.288	1.243

From Table 5.11 we can see Bishop Method has given the least FOS value and the ordinary method the maximum value of FOS from the other methods which shows 8.4% difference between the methods since the factor of safety calculation depends on assumptions of each method.

5.2.1.2.Stability analysis of critical section at TP-2

The critical slope section on TP-2 has resulted a factor of safety 1.415 using the default Morgenstern price method. Although there was seen some slope failures on TP-2 the soil material has a property closer to rock which makes it stronger than the soil found abundantly on the location.

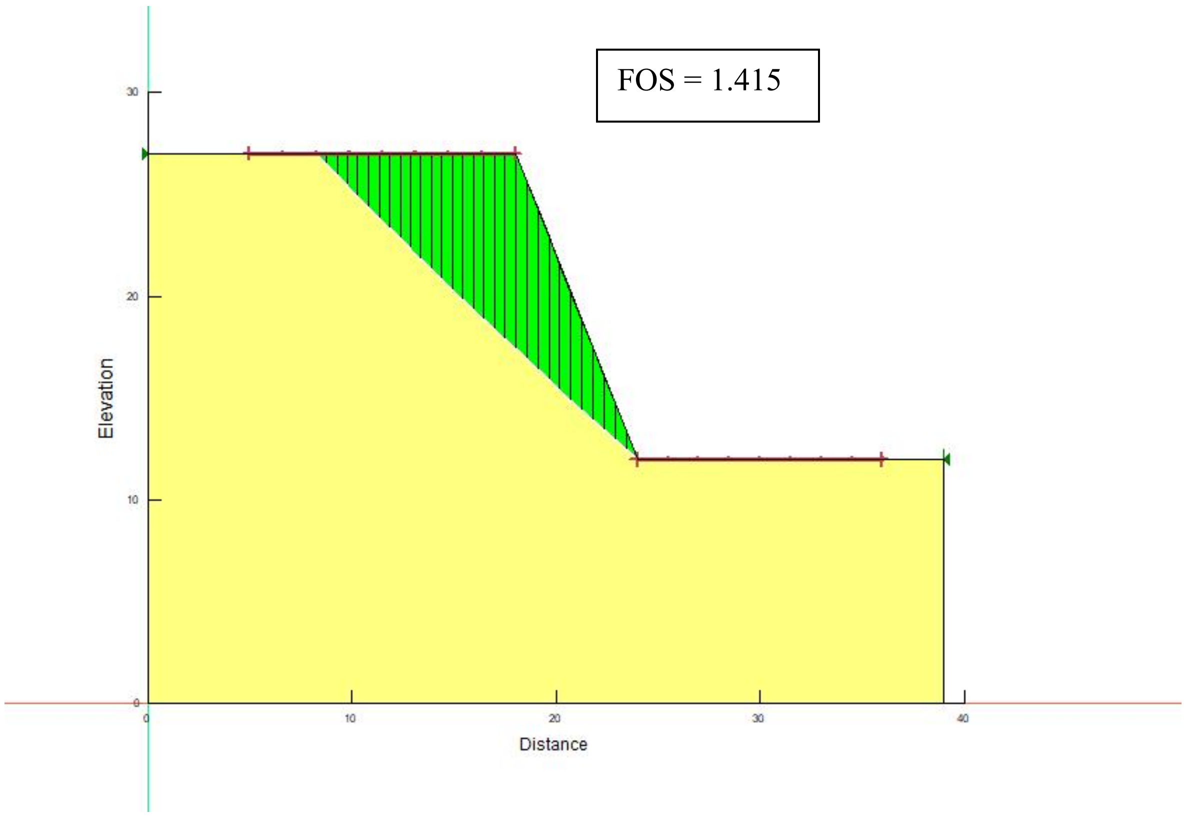


Figure 5.5:Slip surface for TP-2

Although this FS shows safety but in reality, slope failures are occurring which shows it just a matter of time for a major slope failure to happen which may damage the nearby road and become obstacle for transportation.

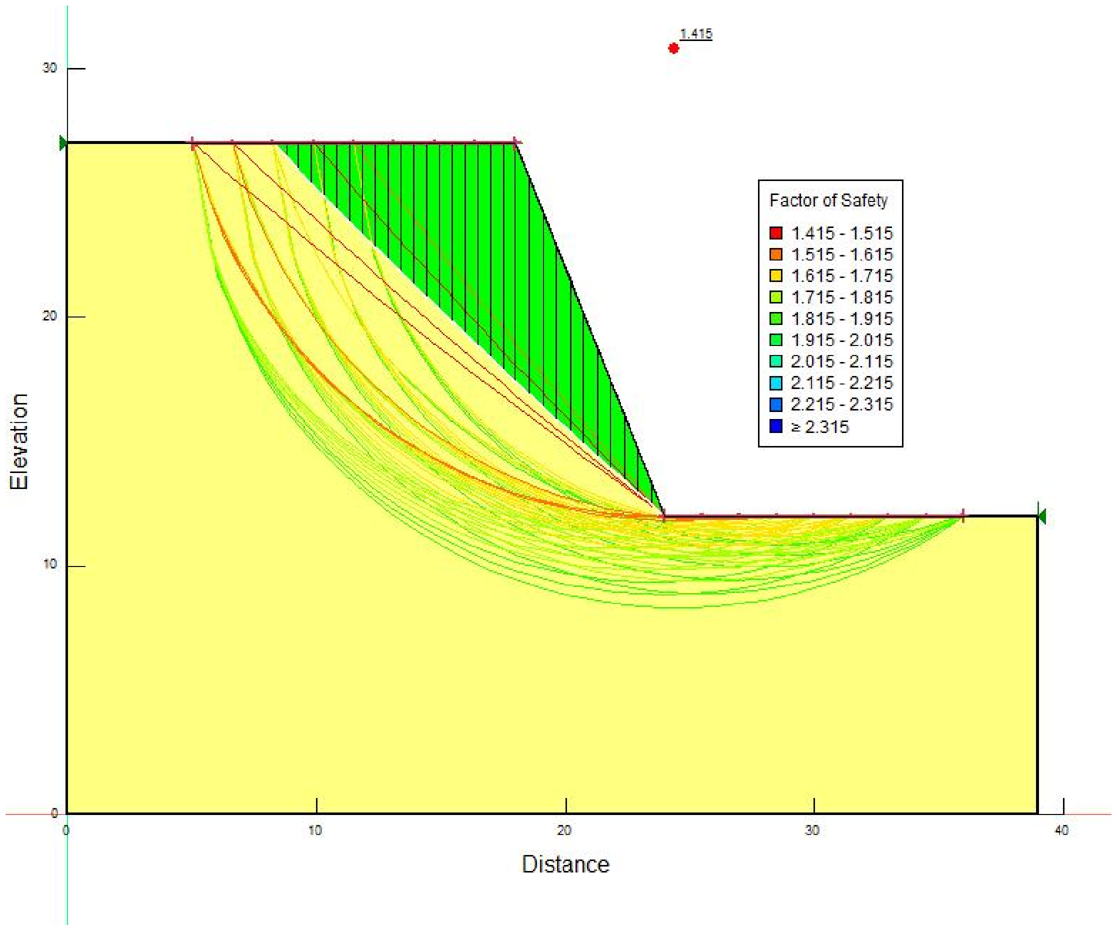


Figure 5.6:Slip surface color map of TP-2

Figure 5.6 shows all the slip surfaces using different color map in order to show the possible factor of safety for each slip surfaces. Each color shows a different range of factor of safety on the slope from safe to possible failure.

Different limit equilibrium methods result different factor of safety for the same slope section because of the differences in their assumptions. Some of the methods and their respective values can be seen below on table 5.12.

Table 5.12: Different limit equilibrium methods and their FOS results for TP-2

MP method	Spencer method	Bishop Method	Janbu method	Ordinary Method
1.415	1.363	1.249	1.307	1.280

Table 18 shows that Morgenstern Price method has results highest factor of safety of 1.415 while ordinary method has resulted a lowest factor of safety of 1.249 this shows a 12.5 % difference between maximum and minimum results from the methods.

5.2.1.3.Stability analysis of critical section at TP-3

The critical slope section on TP-3 has resulted a factor of safety 1.352 using the default Morgenstern price method.

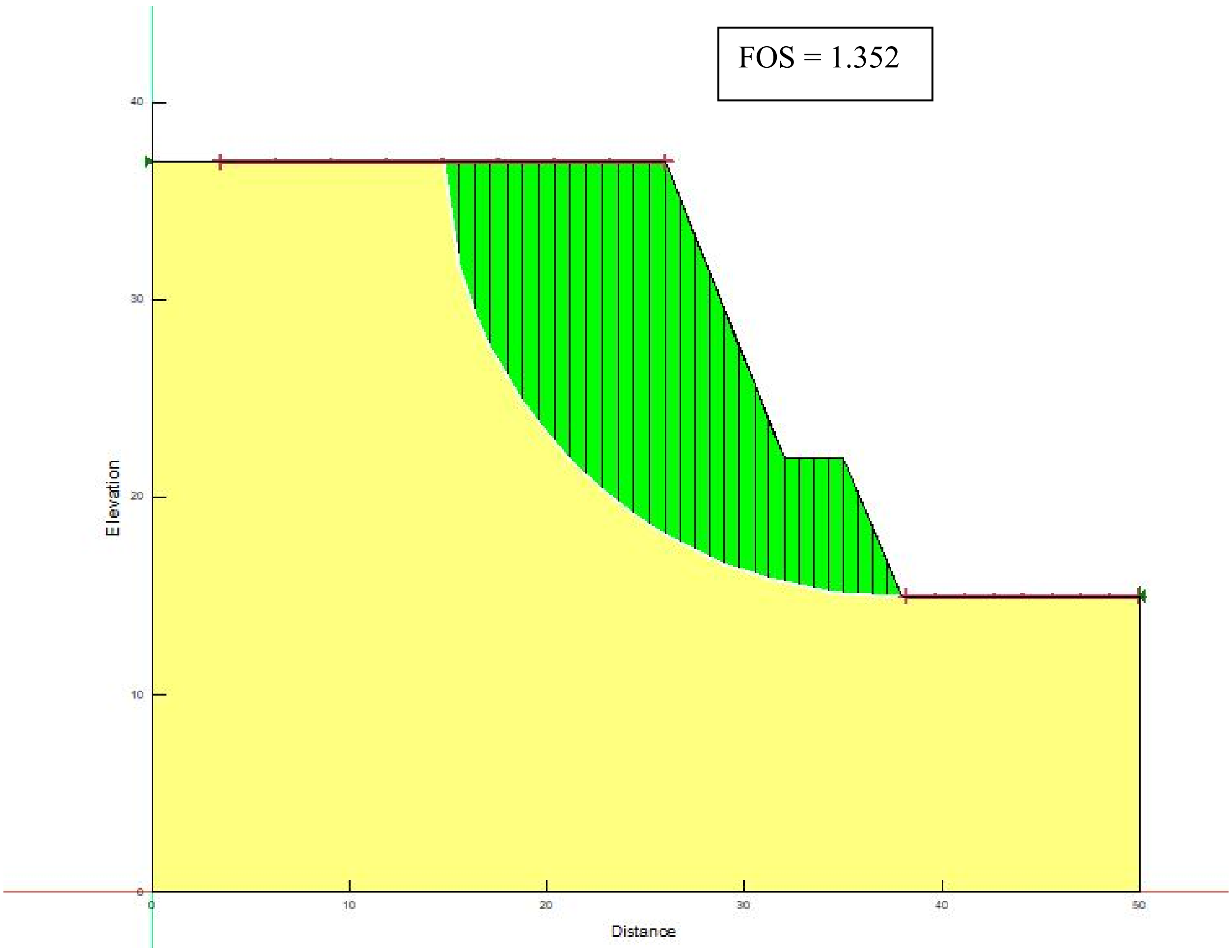


Figure 5.7:Slip surface for TP-3

This factor of safety which 1.352 shows that the slope is marginally stable since its result is greater than 1 but it could potentially fail under certain conditions and changes around the area. The slope still shows signs of potential instability therefore the slope should be monitored.

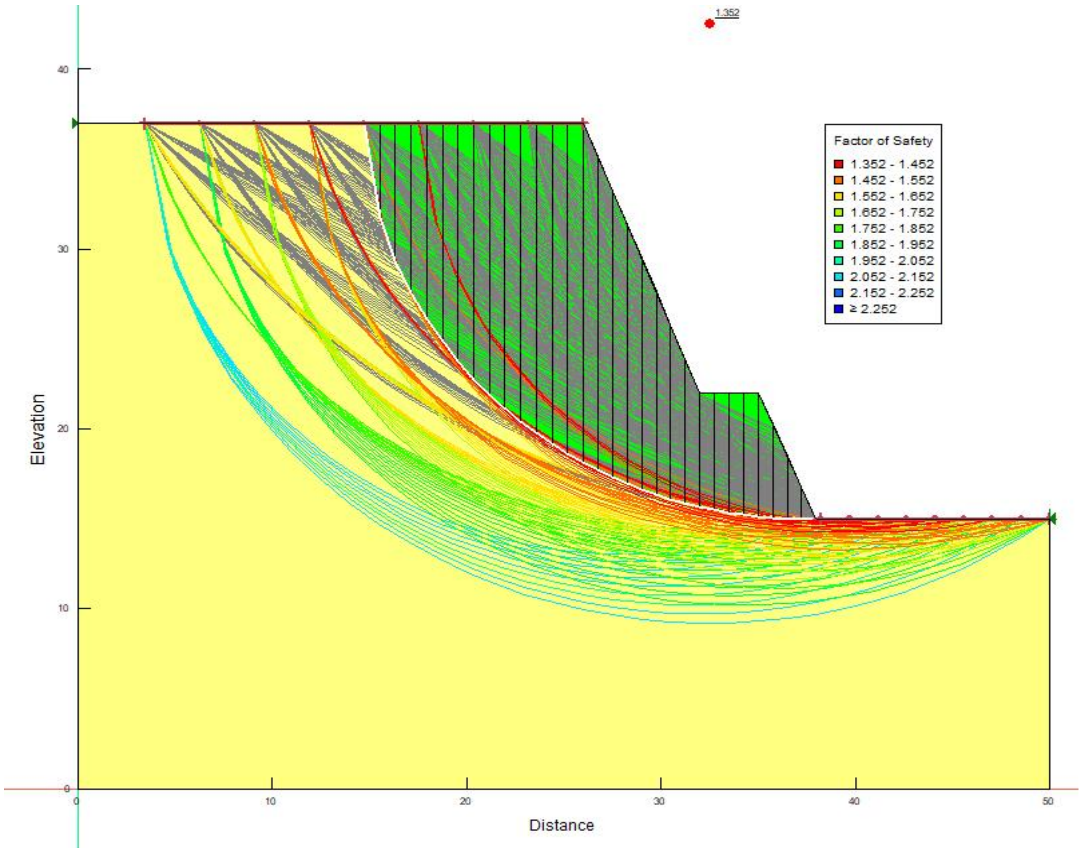


Figure 5.8:Slip surface color map of TP-3

Figure 5.8 shows all the slip surfaces using different color map in order to show the possible factor of safety for each slip surfaces. Each color shows a different range of factor of safety on the slope from safe to possible failure.

Different limit equilibrium methods result different factor of safety for the same slope section because of the differences in their assumptions. Some of the methods and their respective values can be seen below.

Table 5.13: Different limit equilibrium methods and their FOS results for TP-3

MP method	Spencer method	Bishop Method	Janbu method	Ordinary Method
1.352	1.357	1.354	1.307	1.293

Table 5.13 shows Spencer method has resulted maximum factor of safety when compared to others and ordinary methods resulted the minimum factor of safety and a percentage difference of 4.8% between maximum and minimum from the following methods.

5.2.2. PLAXIS 2D

The finite element program Plaxis 2D is particularly helpful for solving the FOS of the slope. Numerous geotechnical issues, including as stability studies and computations of steady-state groundwater flow, can be solved using the computer program. There are four primary sub-routines and many FE models in this software. Inputs, calculations, outputs, and curve graphs are among these routines. The following were considered for analysis:

- Plain strain mode;
- 15 noded triangular elements
- Finer mesh sizes
- Mohr – Coulomb model

Table 5.14:Necessary inputs for slope stability analysis on Plaxis 2D

TP	Slope (H:V)	Slope height (m)	Cohesion ,C (kN/m^2)	Angle of friction(°)	Modulus of elasticity (kN/m^2)	Poisons ratio(v)	Moist Unit weight, ϒ(kN/m^3)	Dry Unit weight, ϒdry(kN/m^3)
1	1:2.7	19	62	15	6500	0.325	17.67	15.068
2	1:2.5	15	37	22	5500	0.325	15.5	12.05
3	1:2.5 1:2.3	15 7	35	29	5500	0.325	18.5	16.078

5.2.2.1.Stability analysis of critical section at TP-1 using Plaxis

Figure 5.9 shows the generated mesh on the Plaxis for the slope area plotted and a finer mesh was used to get better results.

Factor of safety result using finite element method software Plaxis 2D resulted FS= 1.237 which is lesser when compared to the limit equilibrium methods. Factor of safety greater than 1 might show the slope stable but until the factor is greater than 1.5 the slope can be concluded as a marginally stable slope and 1.237 shows there is still failure on the slope which implies the slope failure found on ground with certain changes on conditions of slope failures can be expected.

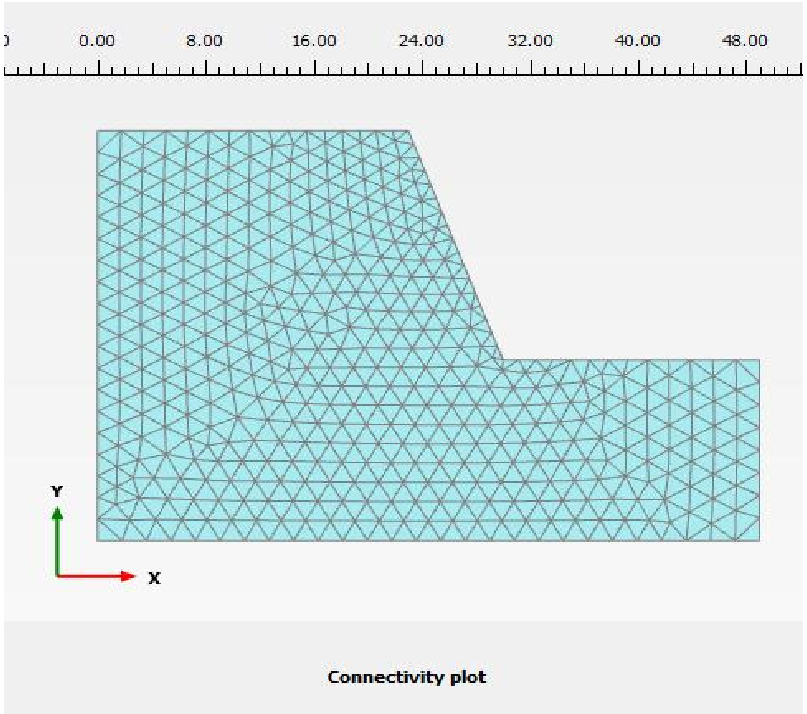


Figure 5.9:Initial Mesh formation for TP-1

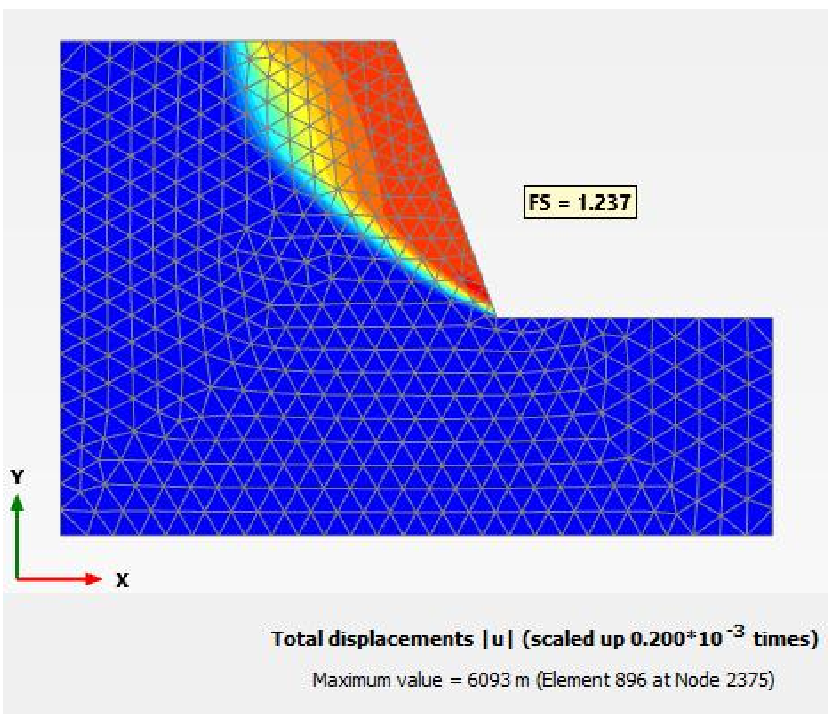


Figure 5.10:FOS result of TP-1 using FEM method Plaxis 2D

5.2.2.2.Stability analysis of critical section at TP-2 using Plaxis

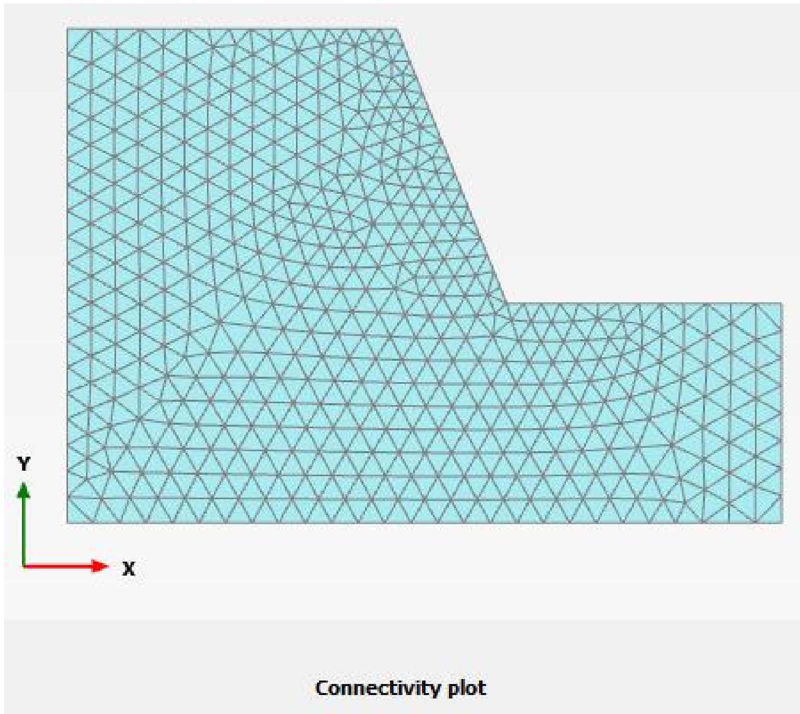


Figure 5.11: Initial Mesh formation for TP-2

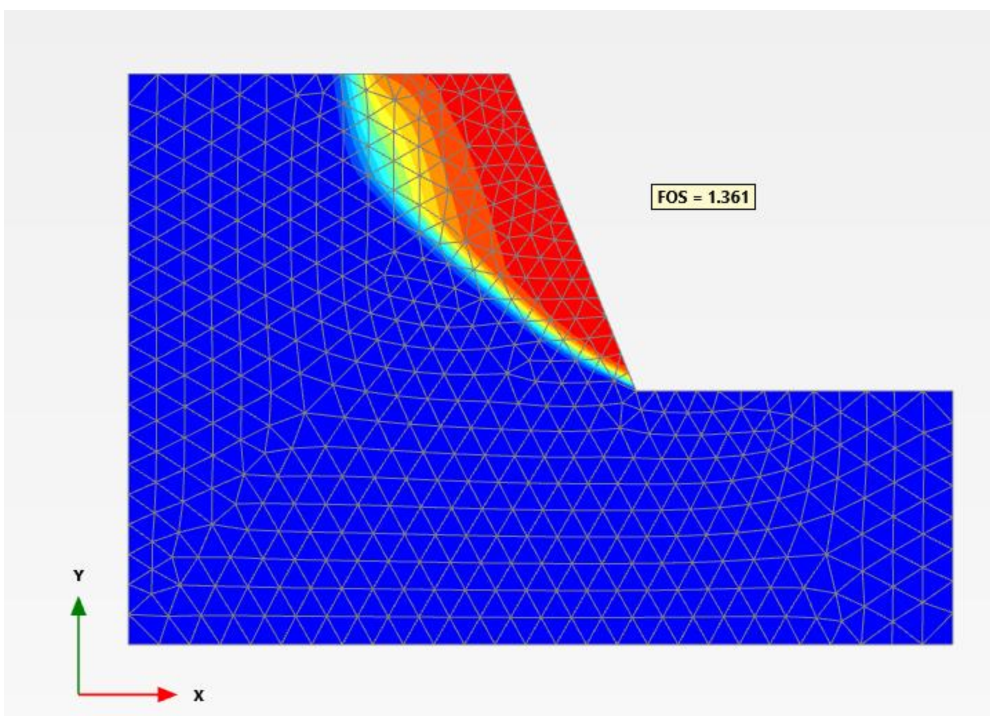


Figure 5.12: FOS result of TP-2 using FEM method Plaxis 2D

Figure 5.11 shows the generated mesh on the Plaxis for the slope area plotted and a finer mesh was used to get better results.

Factor of safety result using finite element method software Plaxis 2D resulted FS= 1.361 which is lesser when compared to the limit equilibrium methods. Factor of safety greater than 1 might show the slope stable but until the factor is greater than 1.5 the slope can be concluded as a marginally stable slope and 1.361 shows there is still failure on the slope which implies the slope failure found on ground with certain changes on conditions of slope failures can be expected.

5.2.2.3.Stability analysis of critical section at TP-3 using Plaxis

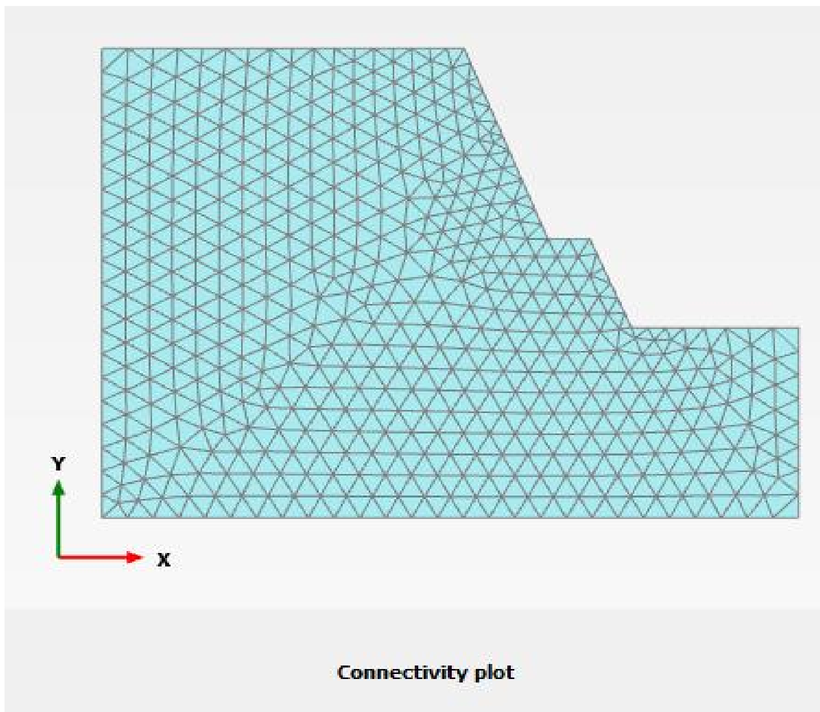


Figure 5.13: Initial Mesh formation for TP-3

Figure 5.13 shows the generated mesh on the Plaxis for the slope area plotted and a finer mesh was used to get better results for the analysis.

Factor of safety result using finite element method software Plaxis 2D resulted FS= 1.140 which is lesser when compared to the limit equilibrium methods. Factor of safety result using finite element method software Plaxis 2D resulted FS= 1.140 which is lesser when compared to the limit equilibrium methods. And the value is closer to 1 which implies this slope has more possibility of failure than the other two slopes used in analysis.

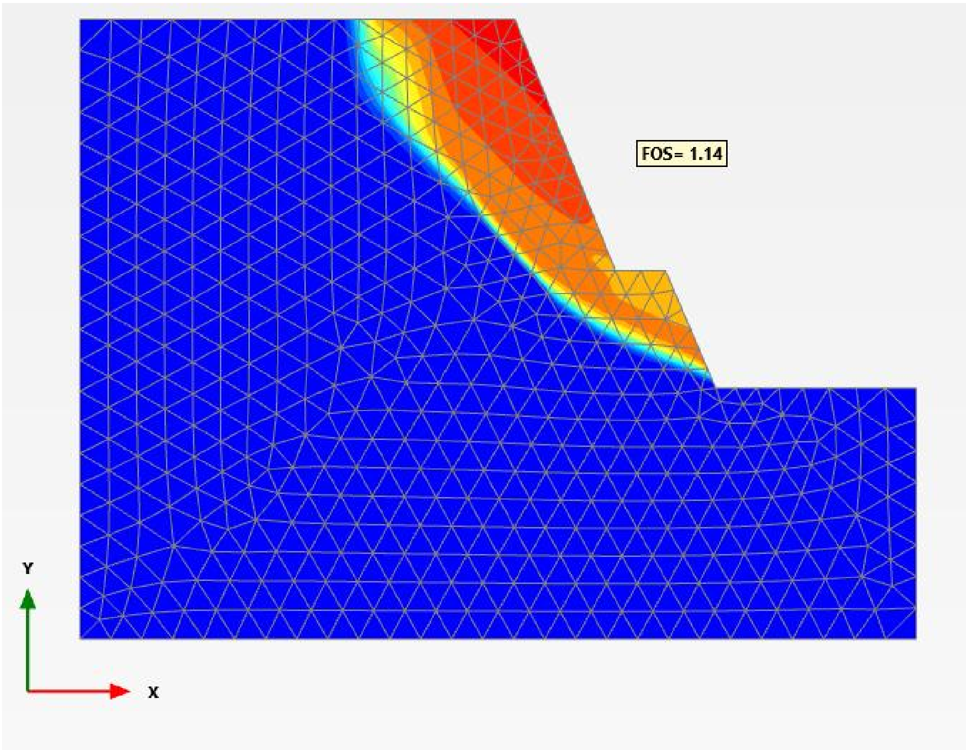


Figure 5.14:FOS result of TP-3 using FEM method Plaxis 2D

5.3. Remedial measures for the slope instability

The main cause for the slope stability in the study area is slope geometry of the terrain which increases its susceptibility to failure, another cause is the long period rainfall which causes characteristic changes on the clayey soil found in the area and also the absence of drainage patterns in order to give relief to the slope when there is water.

The following strategies are suggested to prevent or lessen the effects of a landslide in the research region, depending on the findings of FOS of the slope.

Modification of Slope Geometry- The slope of the research area increases the tangential gravity force since it is steep, which in turn creates the maximum value of shear stress, then results in slope instability. Reducing the overall slope angle, adding material to the area retaining stability, and removing material from the area causing the landslide are all examples of changing the slope geometry.

As it can be seen a sample from the slopes was taken from the slopes and was the slope was modified to 1:1(H:V) and resulted an increment of FS from 1.14 to 2.778 which is safe slope geometry.

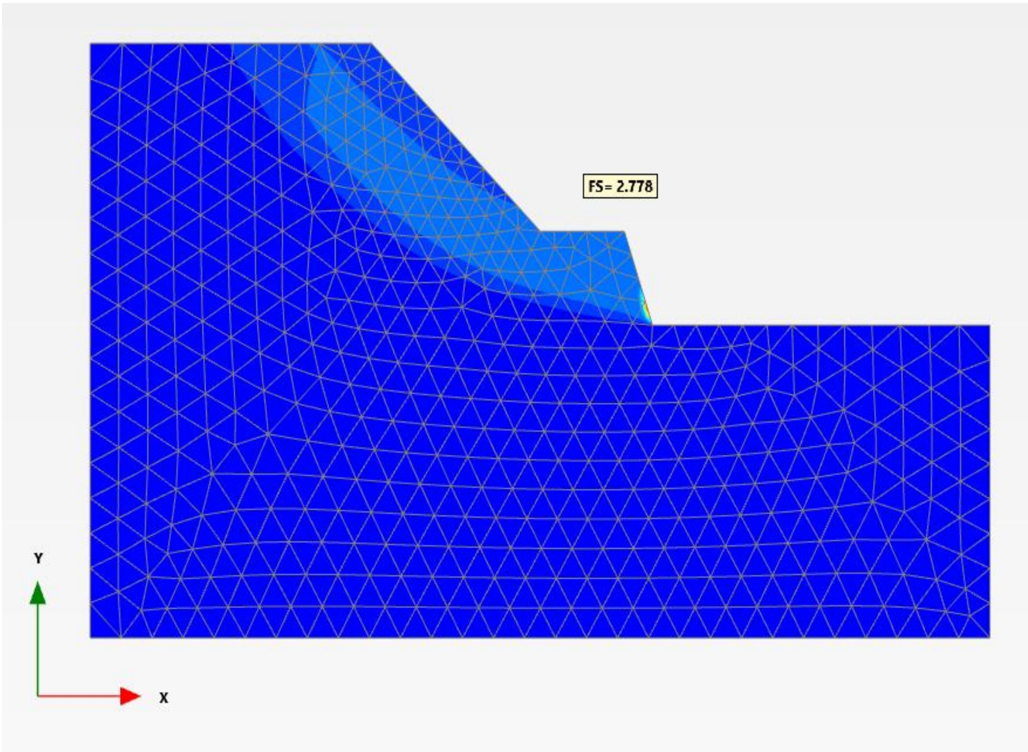


Figure 5.15:Slope Geometry modification by 1:1 onTP-3

Drainage- Surface drains- collecting ditches and pipes, shallow or deep trench drains filled with freely draining geo materials (coarse granular fills and geo synthetics) considering should be applied since the soil at the location is clay soil which exhibits plastic behavior when wet and has poor permeability which leads to poor drainage which makes it difficult to become stable quickly.

Retaining Structures- although there are gabions on some of slope failure location, they need to have continuous and firmly made Gabion walls in order to provide long lasting stability not discontinuous walls found on the study area.

Removing Potentially Unstable Materials- try to substitute the material at study area which better strength material.

Increase vegetation cover- most of the slopes are stable because of the vegetation coverage found on the area, at the time field investigation it was observed the roots of the trees are the ones holding the soil together it has been exposed due to erosion and has no support and protection from failing.

CHAPTER SIX

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

Clay slopes are dangerous and unpredictable because of the complex properties of clay. Although red clays have high soil strength and great slope stability in dry conditions, clay has swelling behavior, it acts as plastic when wet and becomes challenging to work with, and it has low permeability and takes time to lose water and gain strength.

The study area consisted a silty clay as determined from laboratory investigations. It has a natural moisture content of 46%,68% and 32% at TP-1, TP-2 and TP-3 respectively. And a specific gravity of 2.832, 2.597 and 2.747. Moreover, results are presented in chapter four but the soil had a clay content of 60%, 36% and 44% for TP-1, TP-2 and TP-3 respectively.

In conclusion, the laboratory results suggest that the soil samples have varying moisture content, plasticity, specific gravity, particle size distribution, shear strength, density, and compaction characteristics. These variations can significantly impact slope stability along the road.

The factor of safety from Slope/W resulted 1.281, 1.415, and 1.352 while Plaxis resulted a factor of safety 1.237, 1.361 and 1.14 respectively for the slopes TP-1, TP-2 and TP-3 although the factor of safety result is above 1 a water condition was not considered for analysis and the study area has intense rainfall most of the time therefore the slopes factor of safety will decrease than these values. A factor of safety less than 1 indicates that the slope is likely to fail, while a factor of safety greater than 1 indicates stability. In this case the factor of safety values are close to or slightly above 1, suggesting that the slope may be at risk of failure. Since the finite element approach can produce results with lower values of the factor of safety than the majority of other types of limit equilibrium methods, slope stability analysis using that method can produce results that are safer than those obtained using the limit equilibrium method. Increasing the fineness of the mesh improves the precision of the calculations for better result.

Limit equilibrium approach makes a number of assumptions before doing the analysis, such as inserting the entry and exit of critical slip surface, it has more restrictions than the finite element method whereas finite element method creates the critical slip surface and takes more input about the soil sample for better calculation and output.

The causes of slope failures in a study area can vary depending on the specific conditions and geological characteristics. However, some common causes on the study area include: Geological factors: the presence of weak or unstable geological formations, such as clay or loose soil layers, contribute to slope failures. These materials are prone to erosion and can easily become saturated with water, leading to the reduced stability.Weather conditions: Heavy rainfall,or prolonged periods of wet weather increase the water content in the slope, reducing its strength and triggering failures. And this effects the study area since it can cause damage for the infrastructure to area, triggers disruption of transportation near the road , the over flow water and soil due to erosion and blockage of drains on the road results destruction of the road and once a slope has failed, it may become more susceptible to future failures due to weakened soil or altered slope geometry.

Some remedial measures recommended are slope geometry modification, providing drainage, providing drainage and increasing vegetation cover to avoid erosion. Regular monitoring will also help detect any signs of instability and allow for timely intervention if necessary.

6.2. Recommendation

- ✓ Experts in the field should be contacted to determine and identify slope failures to recommend the best and most affordable ways to prevent the catastrophic impacts of failure as time goes on.
- ✓ Additional research can be done using different soil mass conditions subjected to different failure processes, which could produce different results. Due to time constraints this study is limited to examining three slope failures.
- ✓ More accurate results can be obtained by doing detail laboratory and field experiments on undisturbed samples for better representation of area.

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APPENDIX 1: Laboratory Test Results

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	Document No.: LAB-F-003	Revision No. 0	Effective Date: 10/01/2013

Project - MSC Thesis
Client - Rihanna Ansuar
Station - TP-1
Depth - 1.5M
Description - Reddish brown Silty CLAY

Date Sampled - 15/05/23
Date Tested - 22/05/23
Sampled by Client
Test specified by Client

TEST METHOD : AASHTO T- 180 METHOD A

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4755	4870	4960	4900		
C	Wt. of Mold	grams	3175	3175	3175	3175		
D	Wt. Wet Soil	grams	1580	1695	1785	1725		
E	Volume of Mold	cu.cm.	944	944	944	944		
F	Wet Density	gr/cu.cm.	1.67	1.80	1.89	1.83		

G	Container	No.	H	A	W	A9		
H	Wt. Cont + Wet soil	grams	229.65	263.81	273.73	237.23		
I	Wt. Cont + Dry soil	grams	211.77	237.81	232.51	197.50		
J	Weight of Water	grams	17.88	26.00	41.22	39.73		
K	Weight of Container	grams	48.15	57.20	36.61	50.15		
L	Weight of Dry Soil	grams	163.62	180.61	195.90	147.35		

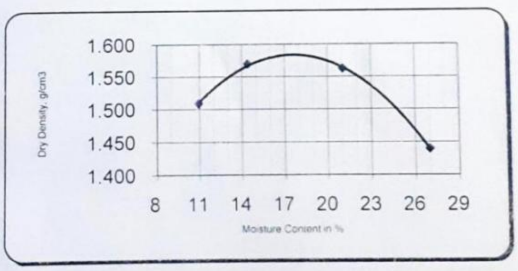
M	Moisture Content	%	10.9	14.4	21.0	27.0		
N	Dry Density	gr/cu.cm.	1.509	1.570	1.562	1.439		

Maximum Dry Density :

MDD = 1.583 gm/cc

Optimum Moisture Content :

OMC = 17.5 %



REMARKS:-

Tested by : meikamu
Date:- 22/05/23

Checked by : Musluseta
Date:- 22-05-23

Approved by :
Date:- 22/05/23
Samson H.

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Figure A.1: Compaction test TP-1

ጎንድዋና ኢ.ጂ.ነሪንግ ኃ.የተ.የግል ኩባንያ Gondwana Engineering PLC.	Title	Moisture Density Relation of Soil	
	Document No.: LAB-F-003	Revision No. 0	Effective Date: 10/01/2013

Project :- MSC Thesis
Client :- Rihanna Ansuar
Station :- TP-2
Depth :- 1.5M
Description :- Yellowish Brown Silty CLAY

Date Sampled :- 15/05/23
Date Tested :- 22/05/23
Sampled by: Client
Test specified by: Client

TEST METHOD : AASHTO T- 180 METHOD A

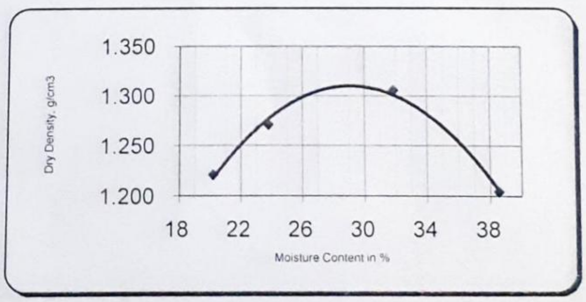
A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4560	4660	4800	4750		
C	Wt. of Mold	grams	3175	3175	3175	3175		
D	Wt. Wet Soil	grams	1385	1485	1625	1575		
E	Volume of Mold	cu.cm.	944	944	944	944		
F	Wet Density	gr/cu.cm.	1.47	1.57	1.72	1.67		
G	Container	No.	A1	A	A	B6		
H	Wt. Cont + Wet soil	grams	209.99	231.24	258.26	230.86		
I	Wt. Cont + Dry soil	grams	184.04	193.51	203.11	182.63		
J	Weight of Water	grams	25.95	37.73	55.15	48.23		
K	Weight of Container	grams	55.46	34.88	29.94	57.46		
L	Weight of Dry Soil	grams	128.58	158.63	173.17	125.17		
M	Moisture Content	%	20.2	23.8	31.8	38.5		
N	Dry Density	gr/cu.cm.	1.221	1.271	1.306	1.204		

Maximum Dry Density :

MDD = 1.310 gm/cc

Optimum Moisture Content :

OMC = 29.0 %



REMARKS:-

Tested by: Melkamu
Date:- 22/05/23

Checked by: Mulugeta
Date:- 22-05-23

Approved by:
Date: 22/05/23
Samson H.

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Figure A.2:Compaction test results of TP-2

ጎንድዋና ኢንጂነሪንግ ኃላፊነቱ የተወሰነ የግል ማኅበር / Gondwana Engineering PLC.	Title	Moisture Density Relation of Soil		
	Document No.: LAB-F-003	Revision No. 0	Effective Date: 10/01/2013	

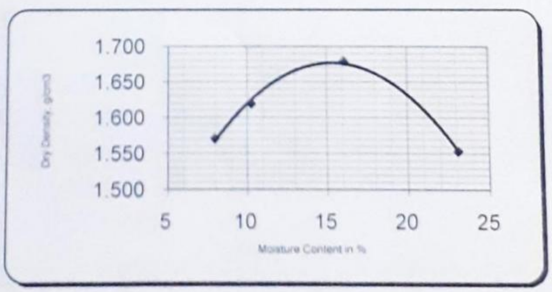
Project:- MSC Thesis
Client:- Rihanna Ansuar
Station:- TP-3
Depth:- 1.5M
Description:- Reddish brown Silty CLAY

Date Sampled:- 15/05/23
Date Tested:- 22/05/23
Sampled by: Client
Test specified by: Client

TEST METHOD : AASHTO T- 180 METHOD A

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4440	4525	4680	4645		
C	Wt. of Mold	grams	2840	2840	2840	2840		
D	Wt. Wet Soil	grams	1600	1685	1840	1805		
E	Volume of Mold	cu.cm.	944	944	944	944		
F	Wet Density	gr/cu.cm.	1.69	1.78	1.95	1.91		
G	Container	No.	E	A	C	A		
H	Wt. Cont + Wet soil	grams	219.49	278.41	244.47	357.22		
I	Wt. Cont + Dry soil	grams	206.99	257.15	217.21	304.34		
J	Weight of Water	grams	12.50	21.26	27.26	52.88		
K	Weight of Container	grams	49.37	49.93	47.52	75.20		
L	Weight of Dry Soil	grams	157.62	207.22	169.69	229.14		
M	Moisture Content	%	7.9	10.3	16.1	23.1		
N	Dry Density	gr/cu.cm.	1.570	1.619	1.679	1.554		

Maximum Dry Density :
MDD = 1.680 gm/cc
Optimum Moisture Content :
OMC = 15.0 %



REMARKS:-

Tested by meikam
Date:- 22/05/23
Checked by: Mulugeta
Date:- 22-05-23
Approved by
Date:- 22/05/23 G.C
Samson H.

ጎንድዋና ኢንጂነሪንግ ኃላፊነቱ የተወሰነ የግል ማኅበር GONDWANA ENGINEERING Private Limited Company

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Figure A.3: Compaction test result for TP-3

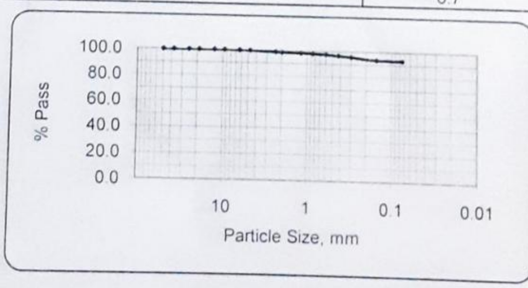
ጎንድዋና ኢንጂነሪንግ ኃ.የተ. የግል ኩባንያ Gondwana Engineering PLC	Title	Wet Sieve Analysis (Grading)	
	Document No.: LAB-F-012	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 27

Project - Msc Thesis Project
Client - Rihanna Ansuar
Station - TP-1
Sampled by:- Client
Test specified by: Client
Description of Sample:- Reddish brown Silty CLAY
Lab. Number:- MT. Grad. 1/2023
Depth:- 1.5M
Date Started:- 05/04/23
Date Issued:- 22/05/23

Weight Before Washing 401 gm Weight After Washing 26.7 gm

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	1.1	0.3	99.7
4.75	1.9	0.5	99.5
2.36	4.7	1.2	98.8
2	6.0	1.5	98.5
1.18	7.2	1.8	98.2
0.85	8.6	2.1	97.9
0.6	10.1	2.5	97.5
0.425	13.4	3.3	96.7
0.3	16.8	4.2	95.8
0.15	24.7	6.2	93.8
0.075	26.7	6.7	93.3



Remark:-

Reported by:- Lab. Engineer MUSIU9HO 22-05-23

Approved by:- Mat. Engineer 22/05/23 G.C Samson H.

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Figure A.4:Grading results of TP-1

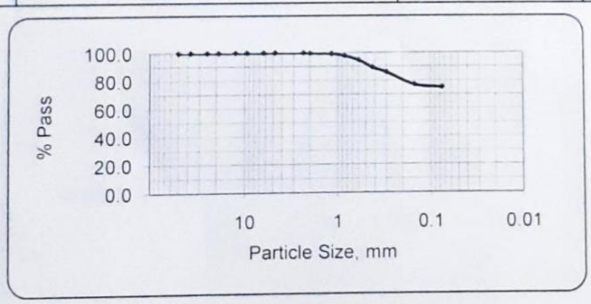
ጎንድዋና ኢንጂነሪንግ ኃ.የተ. የግል ኩባንያ Gondwana Engineering PLC	Title	Wet Sieve Analysis (Grading)	
	Document No.: LAB-F-012	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 27

Project - Msc Thesis Project
Client - Rihanna Ansuar
Station - TP-2
Sampled by - Client
Test specified by Client
Description of Sample - Yellowish Brown Silty CLAY
Lab Number - MT Grad 2/2023
Depth - 1.5M
Date Started - 05/04/23
Date Issued - 22/05/23

Weight Before Washing 341 gm Weight After Washing 84.9 gm

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	0.0	0.0	100.0
2	1.1	0.3	99.7
1.18	2.1	0.6	99.4
0.85	7.7	2.3	97.7
0.6	17.8	5.2	94.8
0.425	36.2	10.6	89.4
0.3	47.7	14.0	86.0
0.15	78.3	23.0	77.0
0.075	84.9	24.9	75.1



Remark:-

Reported by:- Lab. Engineer Mulugeta 22-05-23

Approved by:- Mat. Engineer 22/05/23 G.C Samson H.

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Figure A.5:Grading results of TP-2

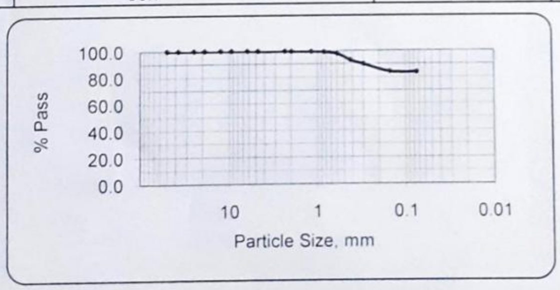
ጎንድዋና ኢንጂነሪንግ ኃ.የተ. የግል ኩባንያ Gondwana Engineering PLC	Title	Wet Sieve Analysis (Grading)	
	Document No.: LAB-F-012	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 27

Project:- Msc Thesis Project
Client:- Rihanna Ansuar
Station:- TP-3
Sampled by:- Client
Test specified by: Client
Description of Sample:- Reddish brown Silty CLAY
Lab. Number:- MT. Grad. 3/2023
Depth:- 1.5M
Date Started:- 05/04/23
Date Issued:- 22/05/23

Weight Before Washing 409 gm Weight After Washing 66.7 gm

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	1.1	0.3	99.7
2.36	1.1	0.3	99.7
2	1.6	0.4	99.6
1.18	2.1	0.5	99.5
0.85	3.7	0.9	99.1
0.6	9.3	2.3	97.7
0.425	28.6	7.0	93.0
0.3	41.4	10.1	89.9
0.15	64.3	15.7	84.3
0.075	66.7	16.3	83.7



Remark:-

Reported by:- Mulugeta 22-05-23
Lab. Engineer

Approved by:-
Mat. Engineer
22/05/23 G.C
Samson H.

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Figure A.6:Grading results of TP-3

ጎንድዋና ኢንጂነሪንግ ኃ.የተ.የግል ኩባንያ Gondwana Engineering PLC	Title	Particle Size Analysis (Hydrometer) Summary		ISO 9001 : 2015 Certified
	Document No.:	Revision No.	Effective Date:	
	LAB-F-014	0	10/01/2013	

Project:- Msc Thesis

Client:- Rihanna Ansuar

Sample by:- Client

Test Specified by:- Client

Date Issued:- 22/05/23

No	Sample type	Depth Sampled(M)	Gravel % (>2.0mm)	Sand % (2.0mm - 0.06mm)	Silt % (0.06mm - 0.002mm)	Clay %(<0.002)	Remark
1	TP-1	1.5	1	19	20	60	
2	TP-2	1.5	0	36	28	36	
3	TP-3	1.5	0	29	27	44	

Reported by:- Melkamu
(Lab. Eng.)
22/05/23

Approved by:-
(Mat. Eng.)
22/05/23 G.C
Samson H.

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Figure A.7:Hydrometer test results

ጎንድዋና ኢንጂነሪንግ ኃላ.የተ.የግል ኩባንያ Gondwana Engineering PLC.	Title	Direct Shear Test		ISO 9001 : 2015 Certified
	Document No.: LAB-F-064	Revision No. 0	Effective Date: 22/04/2015	

Project:- MSC Thesis
Client:- Rihanna Ansuar
Station:- TP-1
Sample of:- -
Depth:- 1.5M

Date Issued:- 22/05/23
Sampled by:- Client
Test Specified by:- Client
Lab. Number:- MSC- Remolded -DS. 1/2023

Thickness of sample,mm	20
radius of sample,mm	31.5
original area of sample,mm²	3117.2

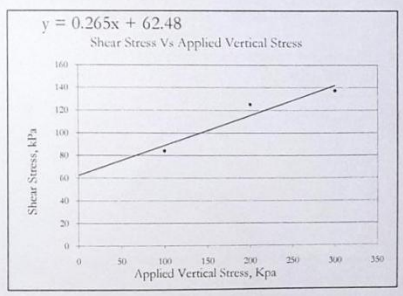
Ring Calibration Factor(N/div)	1.606
Bulk density (gm/cm3)	1.802
Dry density (gm/cm3)	1.536
Moisture (%)	17.3

Ring mass, g	54.78

		Mass 1 (for 100 Kpa), g		169.63	Mass 2 (for 200 Kpa), g		167.61	Mass 3 (for 300 Kpa), g		164.13
		Applied Vertical Stress								
		100 kpa			200 kpa			300 kpa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0.0	3117.2	0	0	0	0	0	0	0	0	0
0.2	3116.9	7	11	4	5	8	3	11	18	6
0.4	3115.5	13	21	7	17	27	9	33	53	17
0.6	3113.5	27	43	14	35	56	18	75	120	39
0.8	3111.1	35	56	18	46	74	24	112	180	58
1.0	3108.2	50	80	26	78	125	40	145	233	75
1.2	3105.1	80	128	41	105	169	54	170	273	88
1.4	3101.7	112	180	58	136	218	70	192	308	99
1.6	3097.9	139	223	72	161	259	83	205	329	106
1.8	3094.0	155	249	80	184	296	96	219	352	114
2.0	3089.8	162	260	84	203	326	106	232	373	121
2.4	3083.1	156	251	81	222	357	116	242	389	126
2.8	3073.4	152	244	79	238	382	124	249	400	130
3.2	3063.0	150	241	79	239	384	125	255	410	134
3.6	3052.0	148	238	78	238	382	125	260	418	137
4.0	3040.3				237	381	125	260	418	137
4.4	3028.0				235	377	125	255	410	135
4.8	3015.2				234	376	125	255	410	136
5.2	3001.8							250	402	134
5.6	2987.9							240	385	129
6.0	2973.6									
6.4	2958.8									
6.8	2943.6									
7.2	2927.9									
7.6	2911.9									
8.0	2895.5									
8.4	2878.7									
8.8	2861.6									

Applied Vertical Stress,Kpa	100	200	300
Shear Stress,Kpa	84	125	137

Cohesion,C (KN/m2)	62
Angle of Internal Friction,ϕ	15



Remark -
Tested by:- Melkamu
Date:- 22/05/23
Checked by:- Melkamu
Date:- 22/05/23
Approved by:-
Date:-
Samson H.
22/05/23 G.C

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Figure A.8:Direct Shear result of TP-1

	ጎንድዋና ኢንጂነሪንግ ኃ.የተ.የግል ኩባንያ Gondwana Engineering PLC.	Title	Direct Shear Test		ISO 9001 : 2015 Certified
		Document No.: LAB-F-064	Revision No. 0	Effective Date: 22/04/2015	

Project.- MSC Thesis
Client:- Rihanna Ansuar
Station:- TP-2
Sample of:- -
Depth:- 1.5M

Date Issued:- 22/05/23
Sampled by:- Client
Test Specified by:- Client
Lab. Number:- MSC- Remolded -DS. 2/2023

Thickness of sample,mm	20
radius of sample,mm	31.5
original area of sample,mm^2	3117.2

Ring Calibration Factor(N/div)	1.606
Bulk density (gm/cm3)	1.585
Dry density (gm/cm3)	1.229
Moisture (%)	29.0

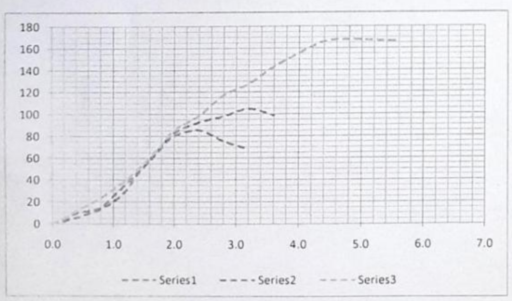
Ring mass, g	54.78

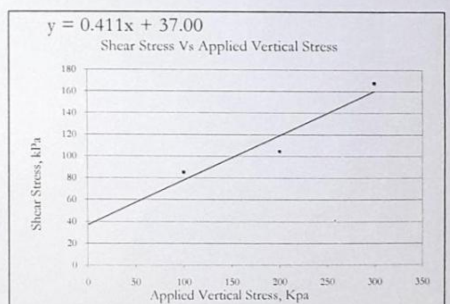
Mass 1 (for 100 Kpa), g	158.11
Mass 2 (for 200 Kpa), g	152.3
Mass 3 (for 300 Kpa), g	150.44

		Applied Vertical Stress								
		100 kpa			200 kpa			300 kpa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0.0	3117.2	0	0	0	0	0	0	0	0	0
0.2	3116.9	5	8	3	8	13	4	10	16	5
0.4	3115.5	11	18	6	19	31	10	23	37	12
0.6	3113.5	18	29	9	23	37	12	34	55	18
0.8	3111.1	26	42	13	29	47	15	46	74	24
1.0	3108.2	39	63	20	49	79	25	61	98	32
1.2	3105.1	57	92	29	69	111	36	76	122	39
1.4	3101.7	87	140	45	90	145	47	95	153	49
1.6	3097.9	110	177	57	113	181	59	116	186	60
1.8	3094.0	134	215	70	138	222	72	140	225	73
2.0	3089.8	154	247	80	160	257	83	164	263	85
2.4	3083.1	164	263	85	178	286	93	189	304	98
2.8	3073.4	144	231	75	188	302	98	224	360	117
3.2	3063.0	130	209	68	200	321	105	243	390	127
3.6	3052.0				188	302	99	272	437	143
4.0	3040.3							294	472	155
4.4	3028.0							313	503	166
4.8	3015.2							315	506	168
5.2	3001.8							312	501	167
5.6	2987.9							309	496	166
6.0	2973.6									
6.4	2958.8									
6.8	2943.6									
7.2	2927.9									
7.4	2915.9									

Applied Vertical Stress,Kpa	100	200	300
Shear Stress,Kpa	85	105	168

Cohesion,C (KN/m2)	37
Angle of Internal Friction,ϕ	22





Remark:-
Tested by:- Melkamu
Date:- 22/05/23
Checked by:- Melkamu
Date:- 22/05/23
Approved by:-
Date:- 22/05/23 G.C
Samson H.

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Figure A.9:Direct Shear result of TP-2

ጎንድዋና ኢንጂነሪንግ ኃ.የተ.የግል ኩባንያ
Gondwana Engineering PLC.

Title	Soil Test Summary	
Document No.: LAB-F-035	Revision No. 0	Effective Date: 10/01/2013

ISO 9001 : 2015 Certified

Project :- MSC Thesis

Client:- Rihanna Ansuar

Sample by:- Client

Test Specified by:- Client

Date 22/05/23

Sr. No	Station	Material Description	Depth (M)	Sieve Pass % (mm)				LL %	PL %	PI %	AASHTO Soil Class	Specific Gravity (GS)	NMC %	FS %
				4.750	2.000	0.425	0.075							
1	TP-1	Reddish brown Silty CLAY	1.5	100	99	97	93	54	33	21	A-7-5 (15)	2.832	46	60
2	TP-2	Yellowish Brown Silty CLAY	1.5	100	100	89	75	64	45	19	A-7-5 (16)	2.597	68	60
3	TP-3	Reddish brown Silty CLAY	1.5	100	100	93	84	45	27	18	A-7-6 (12)	2.747	32	40

Remark

Reported by:- (Lab. Eng.) Mulugeta 22-05-23

GONDWANA ENGINEERING Private Limited Company

Approved by:- (Mat. Eng.) 22/05/2023 G.C Samson H.

Figure A.10:Soil test summary

APPENDIX 2: Software Results

Slope stability of TP-1

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File Information

File Version: 9.01

Revision Number: 9

Date: 03/06/2023

Time: 18:12:46

Tool Version: 9.1.1.16749

File Name: Analysis TP-1.gsz

Directory: C:\Users\Ibro\Desktop\KUKI\Telegram Desktop\

Last Solved Date: 03/06/2023

Last Solved Time: 18:12:49

Project Settings

Unit System: International System of Units (SI)

Analysis Settings

TP-1

Description: This is the first slope section chosen for the slope stability analysis on road from Entoto ftesha to Dilber (Gulele subcity, Addisu Gebeya)

Kind: SLOPE/W

Method: Morgenstern-Price

Settings

Side Function

Interslice force function option: Half-Sine

PWP Conditions from: (none)

Unit Weight of Water: 9.807 kN/m³

Slip Surface

Direction of movement: Left to Right

Use Passive Mode: No

Slip Surface Option: Entry and Exit

Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No

Tension Crack Option: (none)

Distribution

F of S Calculation Option: Constant

Advanced

Geometry Settings

Minimum Slip Surface Depth: 0.1 m

Number of Slices: 30

Factor of Safety Convergence Settings

Maximum Number of Iterations: 100

Tolerable difference in F of S: 0.001

Solution Settings

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Homogeneous soil layer

Model: Mohr-Coulomb

Unit Weight: 17.67 kN/m³

Cohesion': 62 kPa

Phi': 15 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Type: Range

Left-Zone Left Coordinate: (2, 34) m

Left-Zone Right Coordinate: (23, 34) m

Left-Zone Increment: 8

Right Type: Range

Right-Zone Left Coordinate: (30, 15) m

Right-Zone Right Coordinate: (47, 15) m

Right-Zone Increment: 8

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 34) m

Right Coordinate: (49, 15) m

Points

	X	Y
Point 1	0 m	0 m
Point 2	0 m	34 m
Point 3	23 m	34 m
Point 4	30 m	15 m
Point 5	49 m	15 m
Point 6	49 m	0 m

Regions

	Material	Points	Area
Region 1	Homogeneous soil layer	1,2,3,4,5,6	1,238.5 m²

Slip Results

Slip Surfaces Analysed: 61 of 405 converged

Current Slip Surface

Slip Surface: 92

Factor of Safety: 1.281

Volume: 187.37251 m³

Weight: 3,310.8722 kN

Resisting Moment: 149,436.53 kN·m

Activating Moment: 116,715.79 kN·m

Resisting Force: 1,965.6015 kN

Activating Force: 1,534.0353 kN

Slip Rank: 1 of 405 slip surfaces

Exit: (30, 15) m

Entry: (7.25, 34) m

Radius: 58.564818 m

Center: (54.943937, 67.987148) m

Slip Slices

Table A2.1:Slip slice information of TP-1

No.	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	7.625 m	33.485686 m	0 kPa	-32.911684 kPa	-8.818659 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 2	8.375 m	32.479551 m	0 kPa	-3.1667775 kPa	-0.84853547 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 3	9.125 m	31.516608 m	0 kPa	17.175989 kPa	4.6022925 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 4	9.875 m	30.593514 m	0 kPa	32.066198 kPa	8.5921119 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 5	10.625 m	29.707381 m	0 kPa	43.608904 kPa	11.684971 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 6	11.375 m	28.855695 m	0 kPa	53.021531 kPa	14.207076 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 7	12.125 m	28.036249 m	0 kPa	61.055606 kPa	16.3598 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 8	12.875 m	27.247096 m	0 kPa	68.202417 kPa	18.274783 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 9	13.625 m	26.486506 m	0 kPa	74.810395 kPa	20.045385 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 10	14.375 m	25.752934 m	0 kPa	81.17969 kPa	21.752032 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 11	15.125 m	25.044994 m	0 kPa	87.604183 kPa	23.47347 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 12	15.875 m	24.361438 m	0 kPa	94.162538 kPa	25.230776 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 13	16.625 m	23.701134 m	0 kPa	100.60997 kPa	26.958361 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 14	17.375 m	23.063058 m	0 kPa	106.95639 kPa	28.658877 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 15	18.125 m	22.446274 m	0 kPa	113.6633 kPa	30.45599 kPa	62 kPa	0 kPa	Homogeneous soil layer

Slice 16	18.875 m	21.849928 m	0 kPa	121.08008 kPa	32.443311 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 17	19.625 m	21.273237 m	0 kPa	129.27925 kPa	34.640272 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 18	20.375 m	20.71548 m	0 kPa	138.28187 kPa	37.052514 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 19	21.125 m	20.175995 m	0 kPa	148.15714 kPa	39.698587 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 20	21.875 m	19.65417 m	0 kPa	159.00823 kPa	42.606127 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 21	22.625 m	19.149438 m	0 kPa	170.9513 kPa	45.806263 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 22	23.38889 m	18.652539 m	0 kPa	173.24846 kPa	46.421785 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 23	24.16667 m	18.163574 m	0 kPa	163.97299 kPa	43.936432 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 24	24.94444 m	17.69139 m	0 kPa	152.6171 kPa	40.893628 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 25	25.72222 m	17.235519 m	0 kPa	138.49512 kPa	37.109657 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 26	26.5 m	16.795524 m	0 kPa	120.86332 kPa	32.38523 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 27	27.27778 m	16.370999 m	0 kPa	98.94958 kPa	26.51346 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 28	28.05556 m	15.961566 m	0 kPa	72.006733 kPa	19.294146 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 29	28.83333 m	15.566871 m	0 kPa	39.393379 kPa	10.555424 kPa	62 kPa	0 kPa	Homogeneous soil layer
Slice 30	29.61111 m	15.186582 m	0 kPa	0.6815637 kPa	0.18262444 kPa	62 kPa	0 kPa	Homogeneous soil layer

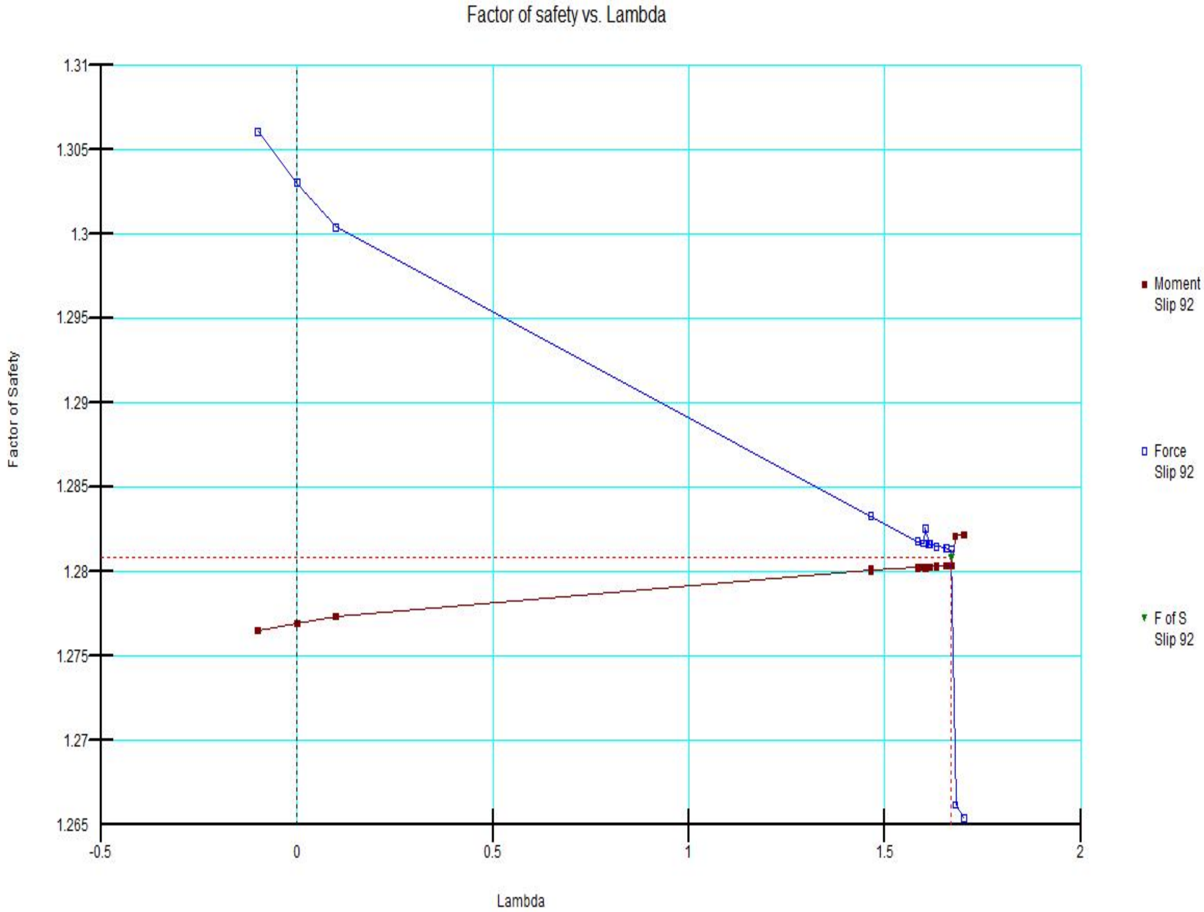

Figure A2.1: FS vs Lambda plot TP-1

Slope Stability for TP-2

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File Information

File Version: 9.01

Revision Number: 4

Date: 03/06/2023

Time: 18:38:37

Tool Version: 9.1.1.16749

File Name: Analysis TP-2.gsz

Directory: C:\Users\Ibro\Desktop\KUKI\Telegram Desktop\

Last Solved Date: 03/06/2023

Last Solved Time: 18:38:40

Project Settings

Unit System: International System of Units (SI)

Analysis Settings

Slope Stability for TP-2

Description: This is the second slope section chosen for the slope stability analysis where failure was observed on road from Entoto ftesha to Dilber (Gulele subcity, Addisu Gebeya) yellowish brown silty clay

Kind: SLOPE/W

Method: Morgenstern-Price

Settings

Side Function

Interslice force function option: Half-Sine

PWP Conditions from: (none)

Unit Weight of Water: 9.807 kN/m³

Slip Surface

Direction of movement: Left to Right

Use Passive Mode: No

Slip Surface Option: Entry and Exit

Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No

Tension Crack Option: (none)

Distribution

F of S Calculation Option: Constant

Advanced

Geometry Settings

Minimum Slip Surface Depth: 0.1 m

Number of Slices: 30

Factor of Safety Convergence Settings

Maximum Number of Iterations: 100

Tolerable difference in F of S: 0.001

Solution Settings

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Homogeneous layer TP-2

Model: Mohr-Coulomb

Unit Weight: 15.5 kN/m³

Cohesion': 37 kPa

Phi': 22 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Type: Range

Left-Zone Left Coordinate: (5, 27) m

Left-Zone Right Coordinate: (18, 27) m

Left-Zone Increment: 8

Right Type: Range

Right-Zone Left Coordinate: (24, 12) m

Right-Zone Right Coordinate: (36, 12) m

Right-Zone Increment: 8

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 27) m

Right Coordinate: (39, 12) m

Points

	X	Y
Point 1	0 m	0 m
Point 2	0 m	27 m
Point 3	18 m	27 m
Point 4	24 m	12 m
Point 5	39 m	12 m
Point 6	39 m	0 m

Regions

	Material	Points	Area
Region 1	Homogeneous layer TP-2	1,2,3,4,5,6	783 m^2

Slip Results

Slip Surfaces Analysed: 52 of 405 converged

Current Slip Surface

Slip Surface: 91

Factor of Safety: 1.415

Volume: 76.500569 m^3

Weight: 1,185.7588 kN

Resisting Moment: 292,565.23 kN·m

Activating Moment: 206,707.05 kN·m

Resisting Force: 835.39445 kN

Activating Force: 590.25366 kN

Slip Rank: 1 of 405 slip surfaces

Exit: (24, 12) m

Entry: (8.25, 27) m

Radius: 253.86652 m

Center: (191.04464, 203.16563) m

Slip Slices

Table A2.2: Slip slice information of TP-2

No.	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	8.5065789 m	26.73454 m	0 kPa	14.390315 kPa	5.8140648 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 2	9.0197368 m	26.205158 m	0 kPa	2.9232558 kPa	1.181072 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 3	9.5328947 m	25.678838 m	0 kPa	6.5154325 kPa	2.6324056 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 4	10.046053 m	25.155554 m	0 kPa	14.354951 kPa	5.7997765 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 5	10.559211 m	24.635279 m	0 kPa	20.93782 kPa	8.4594282 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 6	11.072368 m	24.117986 m	0 kPa	26.537635 kPa	10.721901 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 7	11.585526 m	23.60365 m	0 kPa	31.373663 kPa	12.675783 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 8	12.098684 m	23.092246 m	0 kPa	35.622987 kPa	14.392621 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 9	12.611842 m	22.583748 m	0 kPa	39.430767 kPa	15.931064 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 10	13.125 m	22.078132 m	0 kPa	42.91911 kPa	17.340446 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 11	13.638158 m	21.575374 m	0 kPa	46.195033 kPa	18.664005 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 12	14.151316 m	21.07545 m	0 kPa	49.357299 kPa	19.941643 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 13	14.664474 m	20.578338 m	0 kPa	52.500955 kPa	21.211763 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 14	15.177632 m	20.084013 m	0 kPa	55.718092 kPa	22.51157 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 15	15.690789 m	19.592454 m	0 kPa	59.094999 kPa	23.875929 kPa	37 kPa	0 kPa	Homogeneous layer TP-2

Slice 16	16.203947 m	19.103639 m	0 kPa	62.708521 kPa	25.335887 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 17	16.717105 m	18.617545 m	0 kPa	66.625987 kPa	26.918646 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 18	17.230263 m	18.134151 m	0 kPa	70.911571 kPa	28.650134 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 19	17.743421 m	17.653436 m	0 kPa	75.637827 kPa	30.559666 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 20	18.272727 m	17.160422 m	0 kPa	75.551288 kPa	30.524702 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 21	18.818182 m	16.655257 m	0 kPa	70.292819 kPa	28.400143 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 22	19.363636 m	16.153048 m	0 kPa	64.900524 kPa	26.221514 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 23	19.909091 m	15.653771 m	0 kPa	59.213932 kPa	23.923981 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 24	20.454545 m	15.157402 m	0 kPa	53.043657 kPa	21.431029 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 25	21 m	14.663918 m	0 kPa	46.163735 kPa	18.651359 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 26	21.545455 m	14.173298 m	0 kPa	38.309087 kPa	15.477876 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 27	22.090909 m	13.685517 m	0 kPa	29.173285 kPa	11.786772 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 28	22.636364 m	13.200556 m	0 kPa	18.404518 kPa	7.4359079 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 29	23.181818 m	12.718391 m	0 kPa	5.6015026 kPa	2.263154 kPa	37 kPa	0 kPa	Homogeneous layer TP-2
Slice 30	23.727273 m	12.239003 m	0 kPa	9.6866892 kPa	3.9136765 kPa	37 kPa	0 kPa	Homogeneous layer TP-2

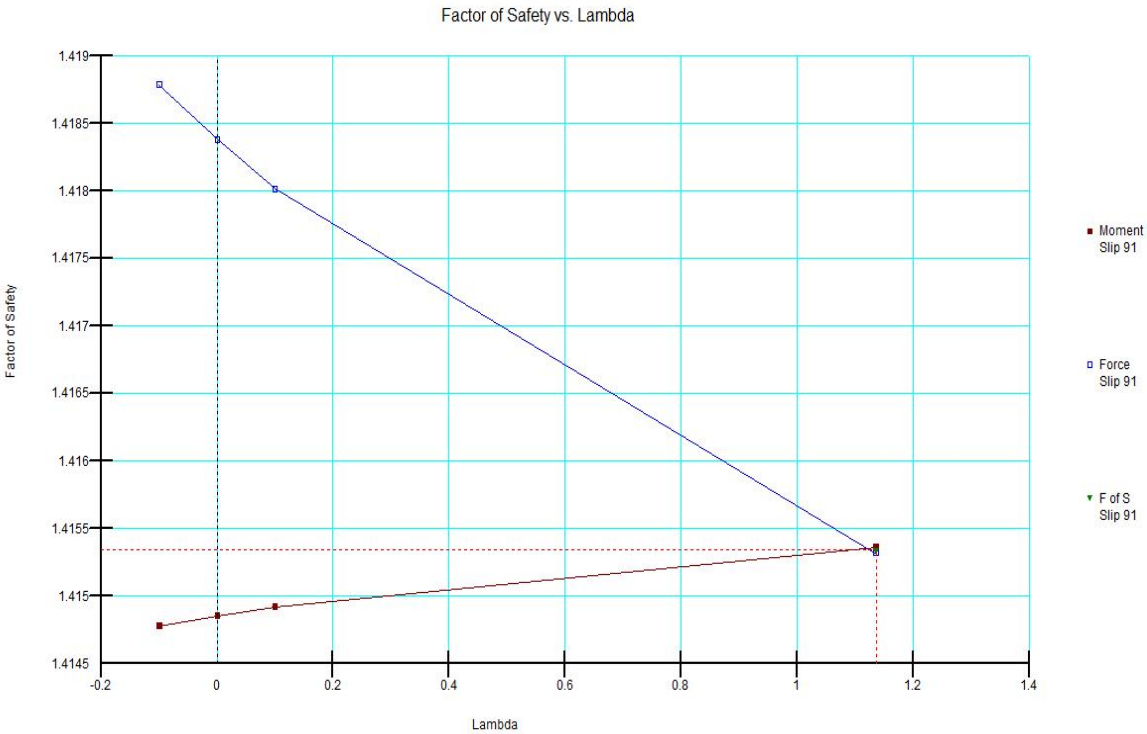

Figure A2.2: FS vs Lambda plot TP-2

Slope Stability of TP-3

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File Information

File Version: 9.01

Revision Number: 10

Date: 03/06/2023

Time: 22:10:16

Tool Version: 9.1.1.16749

File Name: Analysis TP-3.gsz

Directory: C:\Users\Ibro\Desktop\KUKI\Telegram Desktop\

Last Solved Date: 03/06/2023

Last Solved Time: 22:10:18

Project Settings

Unit System: International System of Units (SI)

Analysis Settings

Slope Stability of TP-3

Description: This is thethird slope section chosen for the slope stability analysis where failure was observed on road from Entoto ftesha to Dilber (Gulele subcity, Addisu Gebeya) reddish brown silty clay

Kind: SLOPE/W

Method: Morgenstern-Price

Settings

Side Function

Interslice force function option: Half-Sine

PWP Conditions from: (none)

Unit Weight of Water: 9.807 kN/m³

Slip Surface

Direction of movement: Left to Right

Use Passive Mode: No

Slip Surface Option: Entry and Exit

Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No

Tension Crack Option: (none)

Distribution

F of S Calculation Option: Constant

Advanced

Geometry Settings

Minimum Slip Surface Depth: 0.1 m

Number of Slices: 30

Factor of Safety Convergence Settings

Maximum Number of Iterations: 100

Tolerable difference in F of S: 0.001

Solution Settings

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Homogeneous layer TP-3

Model: Mohr-Coulomb

Unit Weight: 18.5 kN/m³

Cohesion': 35.33 kPa

Phi': 29 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Type: Range

Left-Zone Left Coordinate: (3.4642, 37) m

Left-Zone Right Coordinate: (26, 37) m

Left-Zone Increment: 8

Right Type: Range

Right-Zone Left Coordinate: (38.2546, 15) m

Right-Zone Right Coordinate: (50, 15) m

Right-Zone Increment: 8

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 37) m

Right Coordinate: (50, 15) m

Points

	X	Y
Point 1	0 m	0 m
Point 2	0 m	37 m
Point 3	26 m	37 m
Point 4	32 m	22 m
Point 5	35 m	22 m
Point 6	38 m	15 m
Point 7	50 m	15 m
Point 8	50 m	0 m

Regions

	Material	Points	Area
Region 1	Homogeneous layer TP-3	1,2,3,4,5,6,7,8	1,440.5 m^2

Slip Results

Slip Surfaces Analysed: 90 of 405 converged

Current Slip Surface

Slip Surface: 185

Factor of Safety: 1.352

Volume: 254.18707 m^3

Weight: 4,702.4609 kN

Resisting Moment: 79,119.739 kN·m

Activating Moment: 58,514.646 kN·m

Resisting Force: 2,669.7599 kN

Activating Force: 1,974.8542 kN

Slip Rank: 1 of 405 slip surfaces

Exit: (38.2546, 15) m

Entry: (14.7321, 37) m

Radius: 22.851168 m

Center: (37.567793, 37.840844) m

Slip Slices

Table A2.3: Slip slice information of TP-3

No.	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	15.134525 m	34.386536 m	0 kPa	-30.717342 kPa	-17.026901 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 2	15.939375 m	30.575141 m	0 kPa	21.37334 kPa	11.847436 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 3	16.744225 m	28.481825 m	0 kPa	51.558615 kPa	28.579407 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 4	17.549075 m	26.853247 m	0 kPa	75.395079 kPa	41.792175 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 5	18.353925 m	25.493818 m	0 kPa	95.858034 kPa	53.134976 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 6	19.158775 m	24.319683 m	0 kPa	114.37295 kPa	63.397961 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 7	19.963625 m	23.285078 m	0 kPa	131.77981 kPa	73.046743 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 8	20.768475 m	22.361629 m	0 kPa	148.62706 kPa	82.385323 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 9	21.573325 m	21.530259 m	0 kPa	165.29173 kPa	91.622704 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 10	22.378175 m	20.7774 m	0 kPa	182.03596 kPa	100.90418 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 11	23.183025 m	20.092999 m	0 kPa	199.03487 kPa	110.32683 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 12	23.987875 m	19.469364 m	0 kPa	216.38999 kPa	119.94693 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 13	24.792725 m	18.900467 m	0 kPa	234.13454 kPa	129.78289 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 14	25.597575 m	18.381488 m	0 kPa	252.23415 kPa	139.81567 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 15	26.375 m	17.923208 m	0 kPa	256.5138 kPa	142.18792 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 16	27.125 m	17.519767 m	0 kPa	245.52575 kPa	136.09715 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice	27.875	17.151375	0	232.72702	129.00269	35.33	0 kPa	Homogeneou

17	m	m	kPa	kPa	kPa	kPa		s layer TP-3
Slice 18	28.625 m	16.816186 m	0 kPa	217.89583 kPa	120.78163 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 19	29.375 m	16.512632 m	0 kPa	200.83717 kPa	111.32586 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 20	30.125 m	16.239377 m	0 kPa	181.40081 kPa	100.55211 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 21	30.875 m	15.995282 m	0 kPa	159.49919 kPa	88.411844 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 22	31.625 m	15.779379 m	0 kPa	135.1236 kPa	74.900234 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 23	32.375 m	15.590844 m	0 kPa	124.87624 kPa	69.220029 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 24	33.125 m	15.428988 m	0 kPa	129.67531 kPa	71.880197 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 25	33.875 m	15.293233 m	0 kPa	133.16094 kPa	73.812313 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 26	34.625 m	15.183112 m	0 kPa	135.2735 kPa	74.983328 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 27	35.375 m	15.09825 m	0 kPa	119.80925 kPa	66.411352 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 28	36.125 m	15.038366 m	0 kPa	86.756605 kPa	48.089971 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 29	36.875 m	15.003262 m	0 kPa	52.822405 kPa	29.279937 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 30	37.625 m	14.992825 m	0 kPa	18.497037 kPa	10.253075 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3
Slice 31	38.1273 m	14.996882 m	0 kPa	1.0573196 kPa	0.58608183 kPa	35.33 kPa	0 kPa	Homogeneous layer TP-3

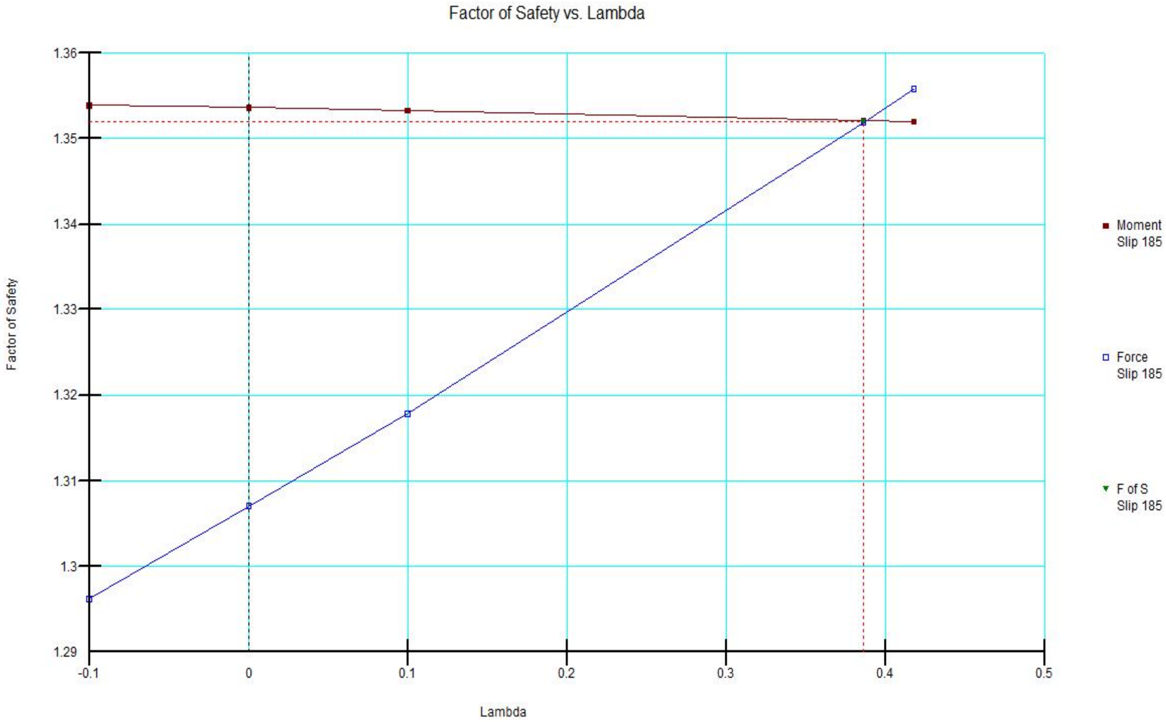

Figure A2.3: FS vs Lambda plot TP-3

Plaxis 2D outputs

Tp-1

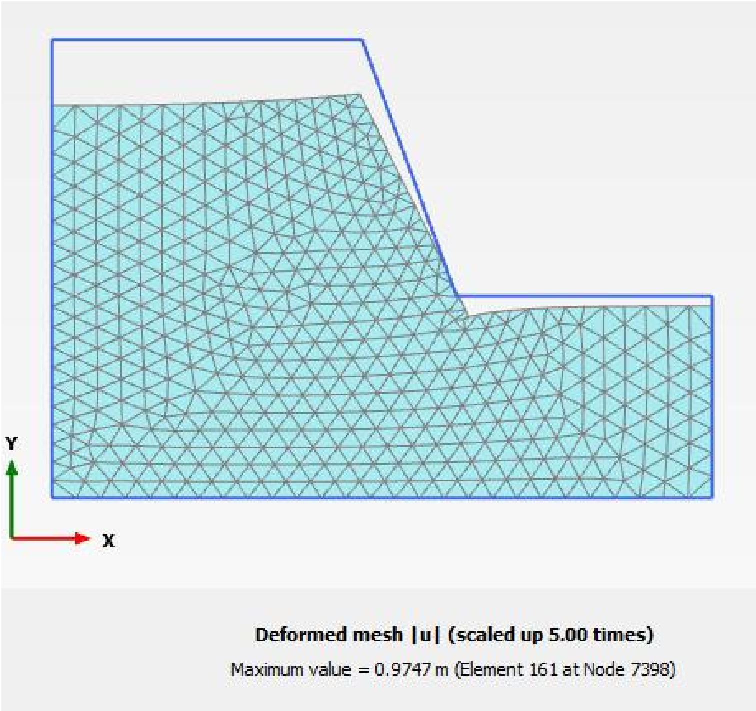

Figure A2.4:Deformation of initial phase of mesh after gravity loading for TP-1

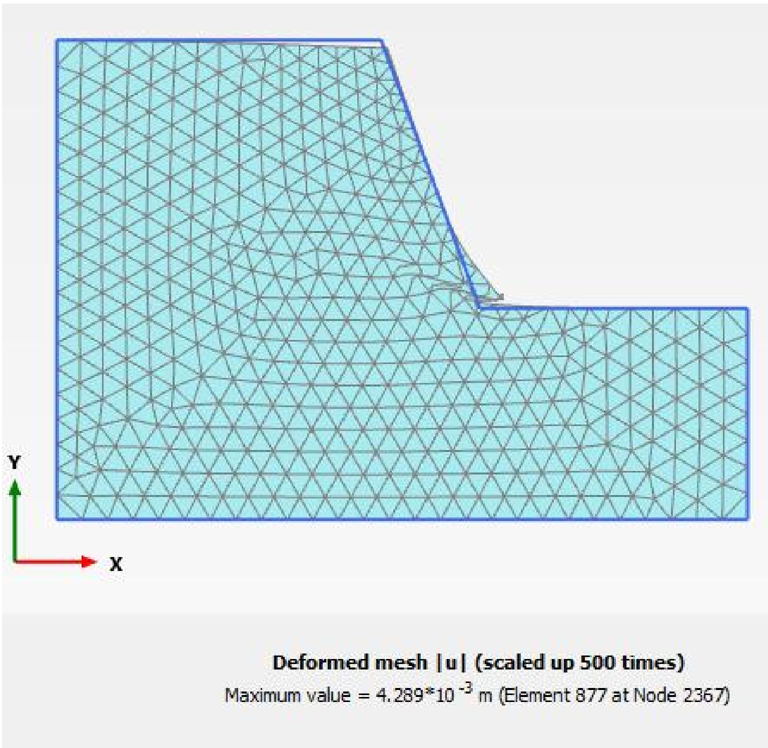

Figure A2.5: Deformed mesh of deformation phase

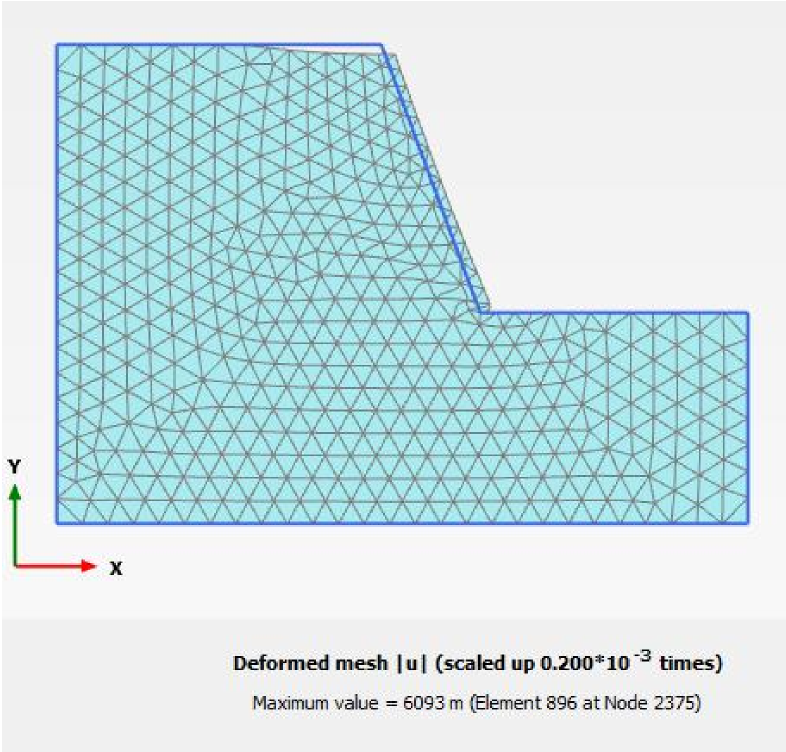

Figure A2.6: Deformed mesh of FOS phase of TP-1

TP-2

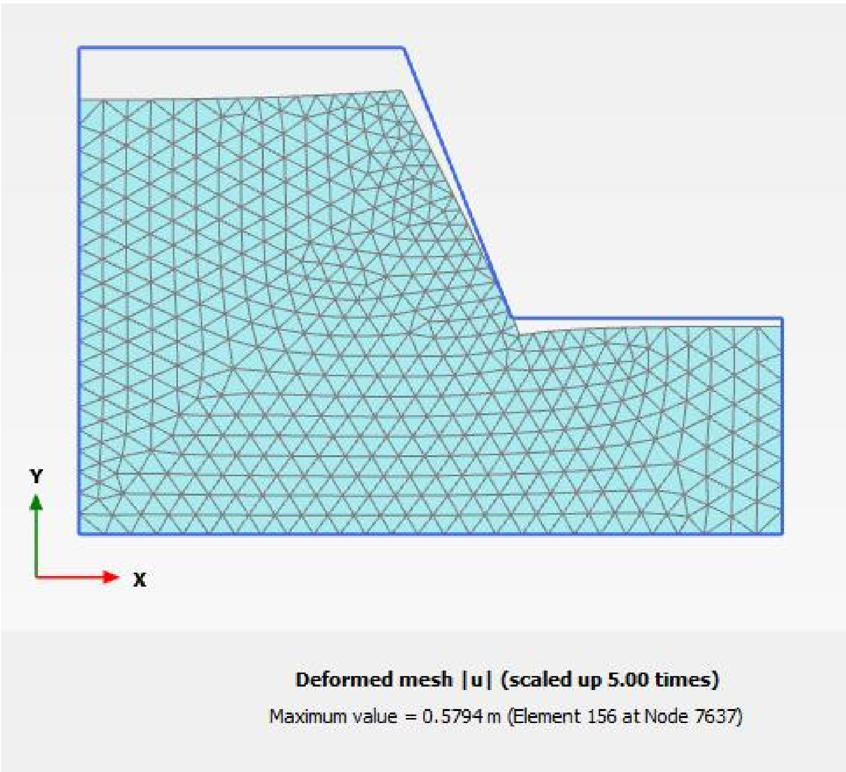

Figure A2.7:Deformation of initial phase of mesh after gravity loading for TP-2

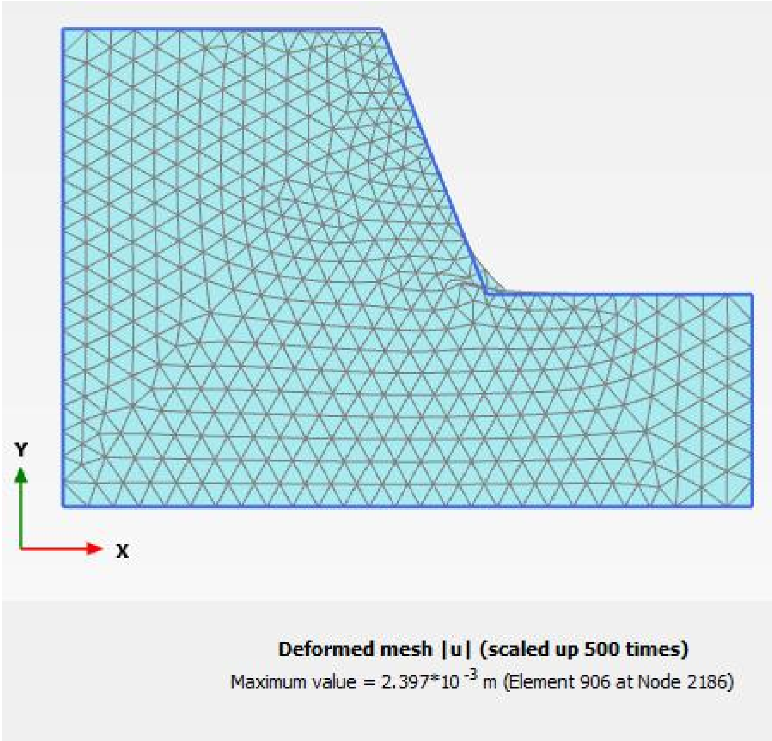

Figure A2.8: Deformed mesh of FOS phase of TP-2

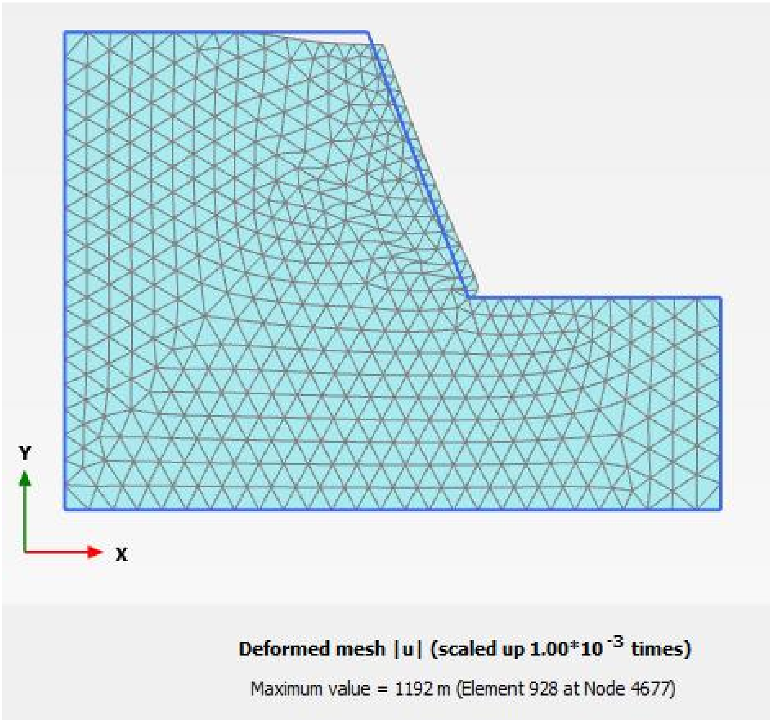

Figure A2.9:Deformed mesh of FOS phase of TP-2

TP-3

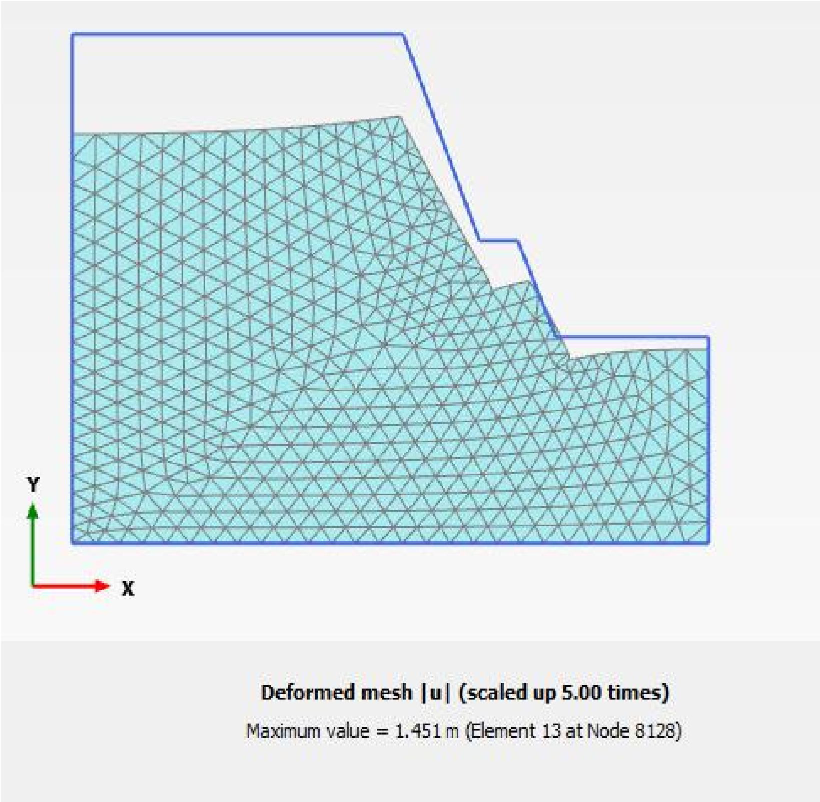

Figure A2.10: Deformation of initial phase of mesh after gravity loading for TP-3

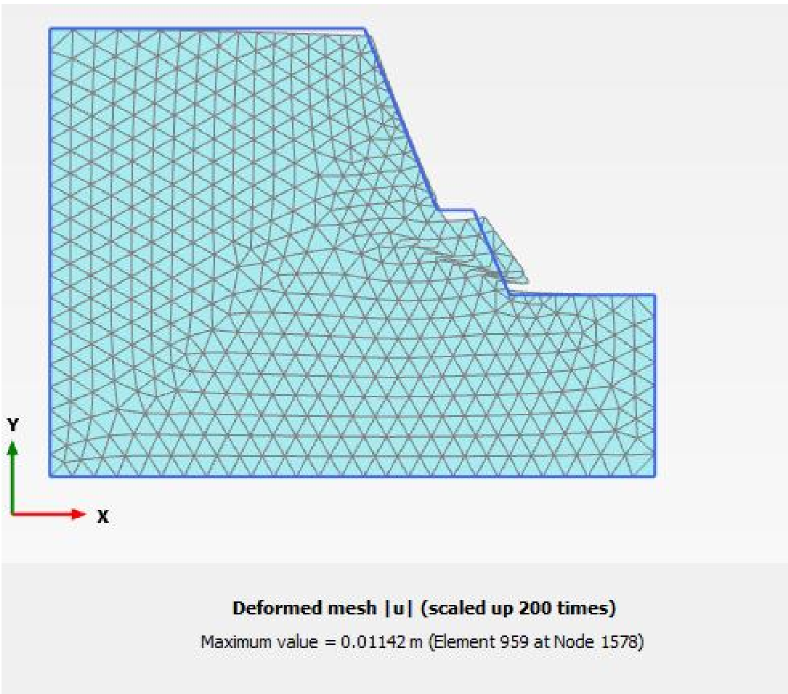

Figure A2.11: Deformed mesh of FOS phase of TP-3

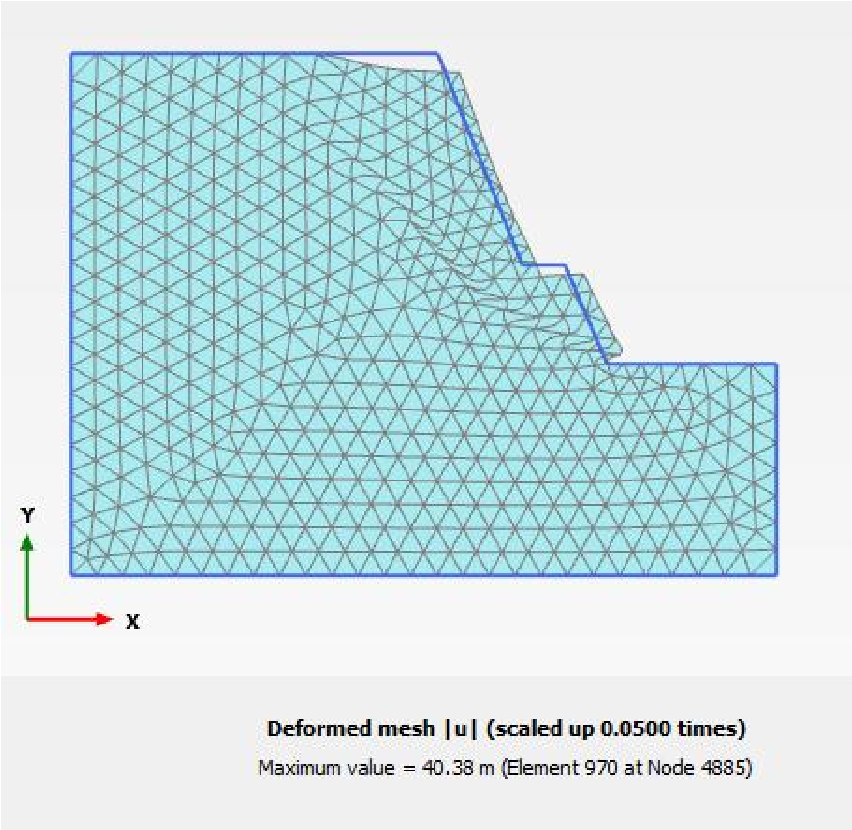

Figure A2.12:Deformed mesh of FOS phase of TP-3

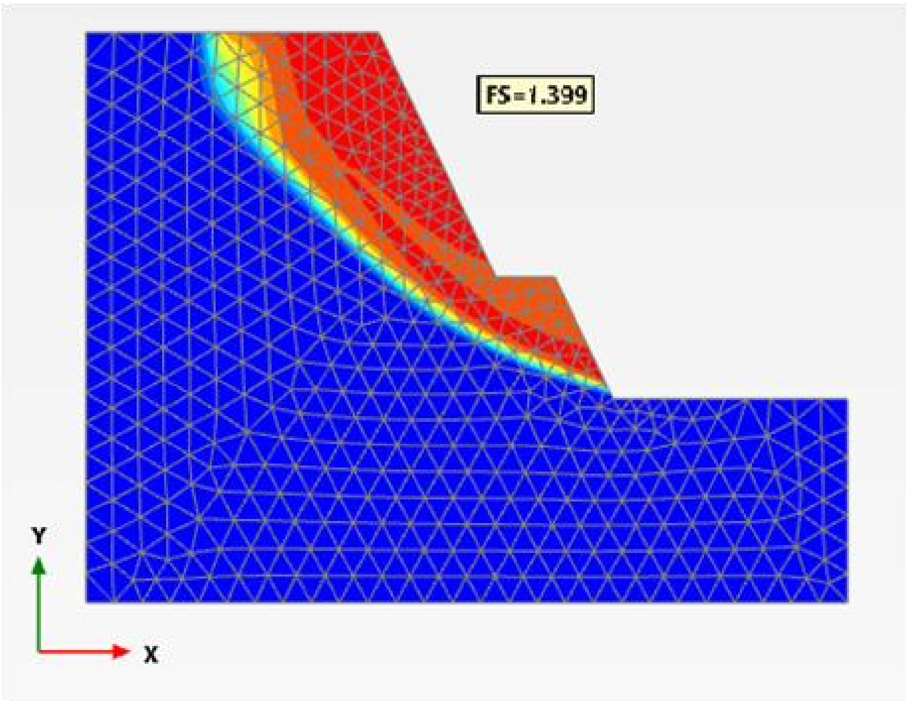

Figure A2.13:Slight geometry modification by 1:2 (Remedial Measure)

NUMERICAL VALIDATION

Validating using hand calculation

The formulation of SLOPE/W is based on limit equilibrium theory. Therefore, hand computations can be used to confirm the forces exerted on a sliding mass. Additionally, the verification procedure could improve the analyst's certainty as well as understanding of the software.

A sample hand calculation has been added by using the ordinary method of slices for the first slope analysis which contained 30 slices in total and the analysis is conducted on excel sheet. Details of each slice is taken from report output of Slope/W.

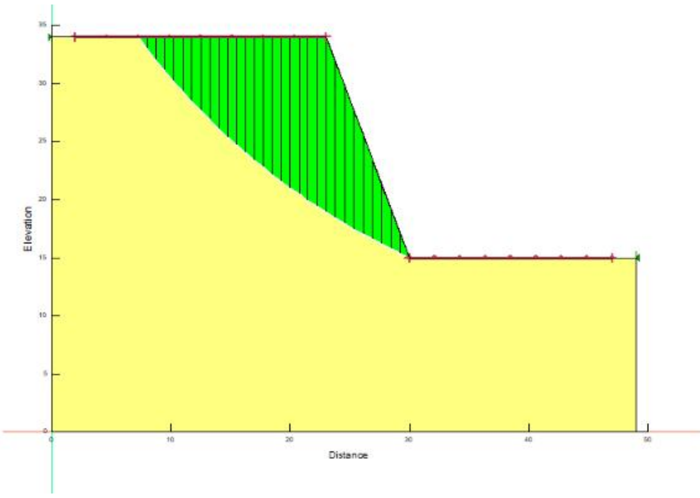

Figure A2.14:Slope/w model

The slice height, slice width (b), base inclination angle (α), The slice weight (W) and slice base length (L) were computed in the spreadsheet using the following equations:

$$W = h * b * \gamma$$

$$FOS = \frac{\sum[c'L + (W\cos\alpha - uL\cos2\alpha)\tan\phi']}{\sum W\sin\alpha}$$

Table A2.4:Geometric data for the slices

Slice #	Height(m)	b(m)	Base α	Base L(m)	W(KN)
1	0.72682	0.58333	-68.135	1.5663	7.49165434

2	2.098	0.58333	-65.648	1.4147	21.6250114
3	3.3245	0.58333	-64.382	1.302	34.2670879
4	4.4388	0.58333	-61.283	1.2141	45.7526696
5	5.4628	0.58333	-59.317	1.1432	56.3074892
6	6.4114	0.58333	-57.459	1.0845	66.0851279
7	7.296	0.58333	-55.691	1.0349	75.2030903
8	8.1248	0.58333	-54	0.99244	83.7458974
9	8.9047	0.58333	-53.376	0.95553	91.7846708
10	9.6408	0.58333	-50.809	0.92312	99.3719782
11	10.338	0.58333	-49.293	0.89442	106.558326
12	10.998	0.58333	-47.822	0.86879	113.361237
13	11.627	0.58333	-46.392	0.84575	119.844618
14	12.224	0.58333	-44.998	0.82494	125.99816
15	12.794	0.58333	-43.638	0.80603	131.873401
16	13.338	0.58333	-42.308	0.78878	137.480649
17	13.857	0.58333	-41.005	0.77299	142.830211
18	14.353	0.58333	-39.728	0.75847	147.942702
19	14.035	0.58333	-38.474	0.7451	144.664936
20	12.906	0.58333	-37.241	0.73274	133.027835
21	11.756	0.58333	-36.028	0.7213	121.174278
22	10.588	0.58333	-34.834	0.71068	109.135186
23	9.4016	0.58333	-33.657	0.70081	96.9064382

24	8.1983	0.58333	-32.495	0.69162	84.5034944
25	6.9784	0.58333	-31.349	0.68305	71.929447
26	5.7426	0.58333	-30.216	0.67505	59.1915113
27	4.4914	0.58333	-29.096	0.66758	46.294841
28	3.2254	0.58333	-27.988	0.66059	33.2456205
29	1.945	0.58333	-26.892	0.65406	20.0479729
30	0.65063	0.58333	-25.806	0.64795	6.7063304

The value of $uLcos2\alpha$ = 0 since u is considered for the analysis on the table below.

Table A2.5:Factor of safety calculations for the Ordinary method.

Slice #	$Wsin\alpha$**(KN)**	CxL**(KN)**	$Wcos\alpha$**(KN)**	$uLcos2\alpha$	**Resistance(KN)**
1	6.95268	97.1106	2.79005	0	97.8581911
2	19.70102	87.7114	8.91688	0	90.1006691
3	30.898523	80.724	14.81603	0	84.6939404
4	40.125258	75.2742	21.983413	0	81.1646335
5	48.42465	70.8784	28.73302	0	78.577384
6	55.7102	67.239	35.54738	0	76.7638849
7	62.118486	64.1638	42.3886	0	75.521783
8	67.751856	61.53128	49.2246	0	74.7209623
9	73.663407	59.24286	54.75516	0	73.9144504
10	77.017633	57.23344	62.7939	0	74.0590027
11	80.777	55.45404	69.4963	0	74.0755041

12	85.00773	53.86498	76.1148	0	74.2598645
13	86.776546	52.4365	82.65929	0	74.5849741
14	89.091071	51.14628	89.09729	0	75.0198098
15	91.00579	49.97386	95.438669	0	75.5465559
16	92.54036	48.90436	101.672	0	76.1472707
17	93.71445	47.92538	107.7871	0	76.8068257
18	94.55665	47.02514	113.78085	0	77.512605
19	90.0046	46.1962	113.256785	0	76.5432423
20	80.5043	45.42988	105.903	0	73.8064829
21	71.27236	44.7206	97.997249	0	70.9788649
22	62.3381	44.06216	89.5793	0	68.0648439
23	53.70747	43.45022	80.66204	0	65.063533
24	45.39747	42.88044	71.27348	0	61.9780977
25	37.42127	42.3491	61.4287	0	58.8088587
26	29.78879	41.8531	51.1494	0	55.5585306
27	22.512	41.38996	40.4527	0	52.2292205
28	15.60172	40.95658	29.3574	0	48.822866
29	9.99124	40.55172	17.8799	0	45.3426213
30	2.9194	40.1729	6.037529	0	41.7906499
Sum	**1717.29203**				**2130.31612**

$$FOS = \frac{\sum[c'L + (W\cos\alpha - uL\cos 2\alpha)\tan\phi']}{\sum W\sin\alpha} = \frac{Resistance}{\sum W\sin\alpha} = \frac{2130.31612}{1717.29203} = 1.24$$

The results of the FS from Slope/W calculation using the ordinary slice method and the results of the hand calculation are practically identical at 1.237 and 1.24, respectively. It is suitable to trust the results because the hand computation and software output demonstrate a good agreement.

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