Road Cut Slope Stability Analysis for Static & Pseudo-static Conditions and Comparative Study of Slope Profiles

(Case Study: Ambo to Gedo road project, Guder).

Leta Meko Kejela

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 Master's in Geotechnical Engineering

Office of Graduate Studies

Adama Science and Technology University

June - 2021

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APPROVAL PAGE

I, the advisors of the thesis entitled "Road Cut Slope Stability Analysis for Static & Pseudo-Static loading Conditions and Comparative Study of Slope Profiles (Case study: Ambo to Gedo road project, Guder)" and developed by Leta Meko, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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Major Advisor	Signature	Date
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Co-Advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis by Leta Meko have read and evaluated the thesis entitled "Road Cut Slope Stability Analysis for Static & Pseudo-static Conditions and Comparative Study of Slope Profiles (Case study: Ambo to Gedo road project, Guder)" and examined the candidate during the 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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Final approval and acceptance of the thesis is contingent upon submission of its final copy to the Office of Postgraduate Studies (OPGS) through the Department Graduate Council (DGC) and School Graduate Committee (SGC).

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DECLARATION

I hereby declare that this Master Thesis entitled "Road Cut Slope Stability Analysis for Static & Pseudo-static loading Conditions and Comparative Study of Slope Profiles (Case study: Ambo to Gedo road project, Guder)" is my original work. That is, it has not been submitted for the award of any 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.

Leta Meko Kejela__________	____________________	___________________
(Name of the student)	Signature	Date

RECOMMENDATION

I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "Road Cut Slope Stability Analysis for Static & Pseudo-static Conditions and Comparative Study of Slope Profiles (Case study: Ambo to Gedo road project, Guder)" prepared under my guidance by Leta Meko submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Geotechnical Engineering. Therefore, I recommend the submission of a revised version of the thesis to the department following the applicable procedures.

Yadeta Chemdesa (PhD)	____________________	____________________
Major Advisor	Signature	Date
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Co-Advisor	Signature	Date

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LIST OF ABBREVIATIONS AND ACRONYMS

ASTM	American Society for Testing Materials Standard
c	Cohesion
D	Disturbance factor
E	Modulus of elasticity
EBCS	Ethiopian building code of standard
FEM	Finite Element Method
FS	Factor of Safety
GSI	Geological Strength Index
HB	Hoeks Brown
LEM	Limit Equilibrium Method
mb	Rock mass coefficient
MC	Mohr-Coulomb
mi	Intact rock coefficient
R	Rebound number
RCSS	Rock critical slope section
SCSS	Soil critical slope section
SRF	Strength Reduction Factor
UCS	Unconfined compressive strength
$\emptyset$	The angle of internal friction
ψ	Angle of dilatancy
v	Poissons ratio

ABSTRACT

Slope instability has been identified as one of the most natural and man-made disasters that can lead to the life of humans in danger and enormous loss of capital. The roads which pass on the hilly and mountainous terrains are frequently affected by slope failures since hilly and mountainous areas are characterized by variable topographical, geological, hydrological, and land-use conditions. Slope stability analysis and comparative study of slope profiles (slope modification methods) are studied in this research work. The stability analysis was carried out using FEM (plaxis 2d) and LEM (slide software). The stability analysis covers both static and dynamic (in pseudo-static analysis) loading conditions on selected soil and rock critical slope sections. Two soil critical slope sections and two rock critical slope sections are analyzed in this study. The strength reduction method and method of slice with Mohr-Coulomb model were used in FEM and LEM respectively. The slope stability analysis was carried out for both dry and wet states of the soil and rock critical slope sections(i.e.static dry, dynamic dry, static wet, and dynamic wet). The analysis result shows soil critical slope sections one and two are stable in dry conditions and unstable in wet conditions. Rock critical slope sections are stable during the dry and wet states of the slope. From the comparative study of slope modification methods flattening the slope (from real site condition i.e. 40º) to 35º, 30º, and 25º increases the factor of safety of the slope by 9.89%, 18.87%, and 32.88% respectively. The use of 2m,3m, 4m, and 5m bench width by keeping the slope angle constant and slope height of 10m increases the factor of safety by 8.23%, 14.87%, 17.04%, and 27.54% respectively when two numbers of the bench are used. Using multi-slope profile by keeping the lower stiff material with a constant slope and changing the slope of upper slope material. The use of upper layer to lower layer slope angle ratio of 1.05, 1.13, 1.2, 1.4, 1.6 increases the factor of safety of the slope by 9.31%, 12.68%, 17.53%, 24.64%, and 41.63% respectively. A soil nail with 15º inclination, 2m nail spacing, and an increase of soil nail length start from 0.7H is used in soil nailing method. A FS of the slope increases with increasing of the soil nail length until the effective length of the soil nail and remains constant when more length of the nail is used. The use of a combination of surface drainage with benching is recommended which means lateral drainage (bench drain) to collect rainwater from each bench slope and drain it laterally to spillways.

Key words: *slope stability, Slope failure, FEM, LEM, Slope profile, Bench slope, Multi slope*

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CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Slope stability problems and associated disastrous event have been faced throughout history when human or nature has disrupted the easily damaged balance of natural soil and rock slopes. Slope instability has been identified as one of the most natural disasters that can lead to the life of humans in danger and enormous loss of capital (Bushira, 2018). Although improvement in identification, prediction and mitigation measures is advanced, slope stability problem still triggers economic and environmental crises in a mountainous region. This is partly due to the complicity of the factors causes slope failure and our inadequate knowledge for surface and sub-surface condition.

The roads which pass on the hilly and mountainous terrains are frequently affected by slope failures since hilly and mountainous areas are characterized by variable topographical, geological, and hydrological, and land-use conditions. This is because, the road crosses on hilly and mountainous terrains experience both deep and shallow excavations in the construction stage, that disturb the inherent nature of rock and soil slopes. In addition, several factors affect the failure of rock or soil slopes. Earthquakes, high ground water pressure (after heavy rains), geological factors, and human activity can cause large rock or earth blocks or even large stone slabs to fall on the lower side of the road (Youssef *et al*., 2012).

As both natural and human activities are responsible for the failure of slopes, it is difficult to prevent the problem entirely. However, the level of damage can be significantly reduced by assessing the stability condition and adopting different preventive measures on it. There are various remedial measures used to reduce the impact of slope failures and these can be grouped into four general classes (i.e., geometric modification, drainage control, slope reinforcement, and retaining structure). It must be noted that, to design safe and economical slopes, assess the stability of existing slopes, and adopt suitable remedial measures on it, understanding the cause of failure, mode of failure, and methods of stability analysis approaches are necessary. Therefore, to assess the instability of slope every one requires to know comprehensive information about the rainfall, engineering property of the slope material, seismicity, and engineering geology of the area.

Slope stability analysis is often carried out to ensure that the analyzed slope can be made safe and the probability of slope failure is minimized (Abramson *et al*., 2001). Among several methods, the Limit equilibrium method and finite element method are the most widely used and accepted for analyzing slope stability problems. Therefore, the current study is entirely focused on Limit equilibrium and finite element method slope stability analysis. For analysis purposes in the present study plaxis 2d and Slide software have been utilized to analyze the stability of the slope along the road cut slopes in the study area. The minimum factor of safety has been computed or calculated by the software's using several parameters and for analysis, values were adopted from the soil and rock investigation reports.

1.2 Statement of the problem

The first and foremost requirement for the analysis and design of slope is to guarantee their safety and reliability during their service life. Road plays a vital role in the development of any country. Most of the constructed roads in the highland of Ethiopia cross within valleys, hilly and mountainous terrain. This leads to slope instability problems whereby any external destabilizing agents like rainfall and earthquake occur. The stability of the slope is currently an issue in our country as landslide activities cause considerable damage to human life and infrastructures, especially on road construction. As roads and highways were constructed on high altitude areas, the road cut slopes of the study area are prone to problems, especially during high rainfall infiltration. So, to reduce the failure of road cut slope the road cut slopes must be designed for future and worst conditions not only for the current situations.

The study area is around Guder where susceptibility to the earthquake is low and has high rainfall intensity during July and august which reduces the shear strength of slope material leading seepage related instability problems. Therefore, this study is proposed to investigate the stability of road cut slope under static and dynamic loading conditions(pseudo-static analysis) for the dry and wet state of the slope along Ambo to Gedo road project particularly by taking a study area. As a result, appropriate stability analysis of cut slope and performance evaluation of construction methods (use of different slope geometric modification methods) and different mitigation measures were conducted.

1.3 Significance of the study

This study has a valuable significance to understand the effect and extent of both earthquake and rainfall in the stability of road cut slopes found around Guder. This study gives the knowledge of factors affecting the road cut slope failure, slope stability analysis method of road cut slope and give the idea of taking mitigation measures if the slope under consideration is not stable.

Furthermore, the study will give the comparison of different slope modification methods by considering the degree of stability when used separately. The performance of remedies evaluated here will assist the selection and design of slope failure countermeasures. Finally, the study will serve as input for other researchers or institutions to design and analyze the stability of cut slopes especially around Guder.

1.4 Objective of the study

1.4.1 General objective

The general objective of this study is Road cut slope stability analysis for static & dynamic loading conditions (pseudo-static analysis) and comparative study of slope profiles (Case study: Ambo to Gedo road project, Guder).

1.4.2 Specific objectives

1. Identification of the critical road cut slopes along the road section.
2. Determination of the Geotechnical and Engineering geological properties of the soil and rock mass along critical slope section.
3. Conducting slope stability of critical slope sections along the road in dry and wet conditions under both static and dynamic loading conditions (pseudo-static analysis).
4. Evaluation of the effect of Geometric profiles through the comparative evaluation of slope profile modification methods.
5. To develop mitigation measures for selected slope sections of the road and to suggest alternative solutions if the critical slope sections are unstable.

1.5 Scope of the study

The scope of this study is limited to the road cut slope stability analysis and comparative study of slope profiles for roads constructed in Guder. The analysis was carried using both finite element method and limit equilibrium method by using plaxis 2D and slide software respectively on a critical road cut sections for static and dynamic loading. Suitable data for the analysis was obtained from field investigation, laboratory tests, and correlations using the literature. A comparative study was made on slope profiles (single slope, bench slope, and multi-slope) and soil nailing in the selected slope section. The modeling results are evaluated in terms of deformation and factor of safety using plaxis 2D and slide software.

1.6 Organization of the thesis (Thesis outline)

Chapter One: Present a detailed description of introduction to slope stability, background, statement of the problem, and objective of this research work.

Chapter Two: Detail of description of different parameters and methods that are going to be used in the next chapters and summary of some previous related research works.

Chapter Three: A detail of the study area, methodology used to carry out the overall research work, and the conditions for which the research is carried out.

Chapter Four: Presents a detailed explanation of field tests, laboratory tests on soil and rock samples, and correlations used to generate all parameters used in slope modeling and numerical analysis.

Chapter Five: Presents a detail of the numerical modeling procedures of critical slope sections in both LEM (slide software) and FEM (plaxis 2D). Modeling of geometric profiles for performance evaluation and validation of numerical models is also included in this chapter.

Chapter Six: Presents discussions on the stability conditions of slope and comparisons of analysis methods. Discussions of the performance of geometric profiles, comparison, and suitable conditions for them are also included in this chapter.

Chapter Seven: Provides a general conclusion and recommendations made from the study

CHAPTER TWO

LITERATURE REVIEW

2.1 General

A slope failure is a phenomenon that a slope collapses abruptly due to weakened self-retainability of the earth under the influence of rainfall or an earthquake. Slope failure is an ongoing problem all over the world causing damages to infrastructures, human life, and natural resources. To solve slope stability problems a good understanding of the driving process and mechanisms of slope failure, analytical methods, investigative tools, and stabilization measures are required. Hence, in this section review was made on mechanisms of slope failure and factors affecting slope stability, shear strength criteria, analysis methods, previous stability assessment, and stabilization methods.

2.2 Mechanisms of slope failures

Many naturally occurring factors conspiring to fail a slope. These factors include the geometry of the slope, shear strength parameters of soil or rock material, layering, surface drainage characteristics, and groundwater conditions. Depending up on the cause of failure, geometry, materials involved, and rate of movement slope failures can take place in various types of movements.

Table 2.1 Common types of soil and rock slope failures (Liu *et al.*, 2018).

Mechanisms of failure	Conditions of failure
Planar failure	Rock slope failure occurs under gravity when rock blocks rest on an inclined failure like a joint that daylight into free space. It is controlled by the relationship of slope & strike angle, dip direction & discontinuity friction angle.
Wedge failure	Occurs when two discontinuities striking obliquely into the rock slope face and slide of wedges take place along the line of intersection of two such a plane.
Topples	Tops are a force exerted by gravity and the force of adjacent elements moving apart or a rock slab forward.
Slides	A translational or rotational movement can occur on the surface of rupture if there is a zone of weakness that separates the slide material from the more stable underlying material. The slide is a common

	type of mechanism of failure in deeply weathered residual soils, colluvial soils, and alluvial deposits.
Falls	Sudden movement on steep slopes formed due to undercutting, differential weathering, and stream wearing down.
Earth flow	It occurs on liquefiable soils especially when dynamic force occurs in saturated materials
Debris flow	It is a fast mass movement of loose soil, rock, and organic matter that erodes soft soils at high altitudes and is stimulated by heavy rainfall or rapid ice flow.

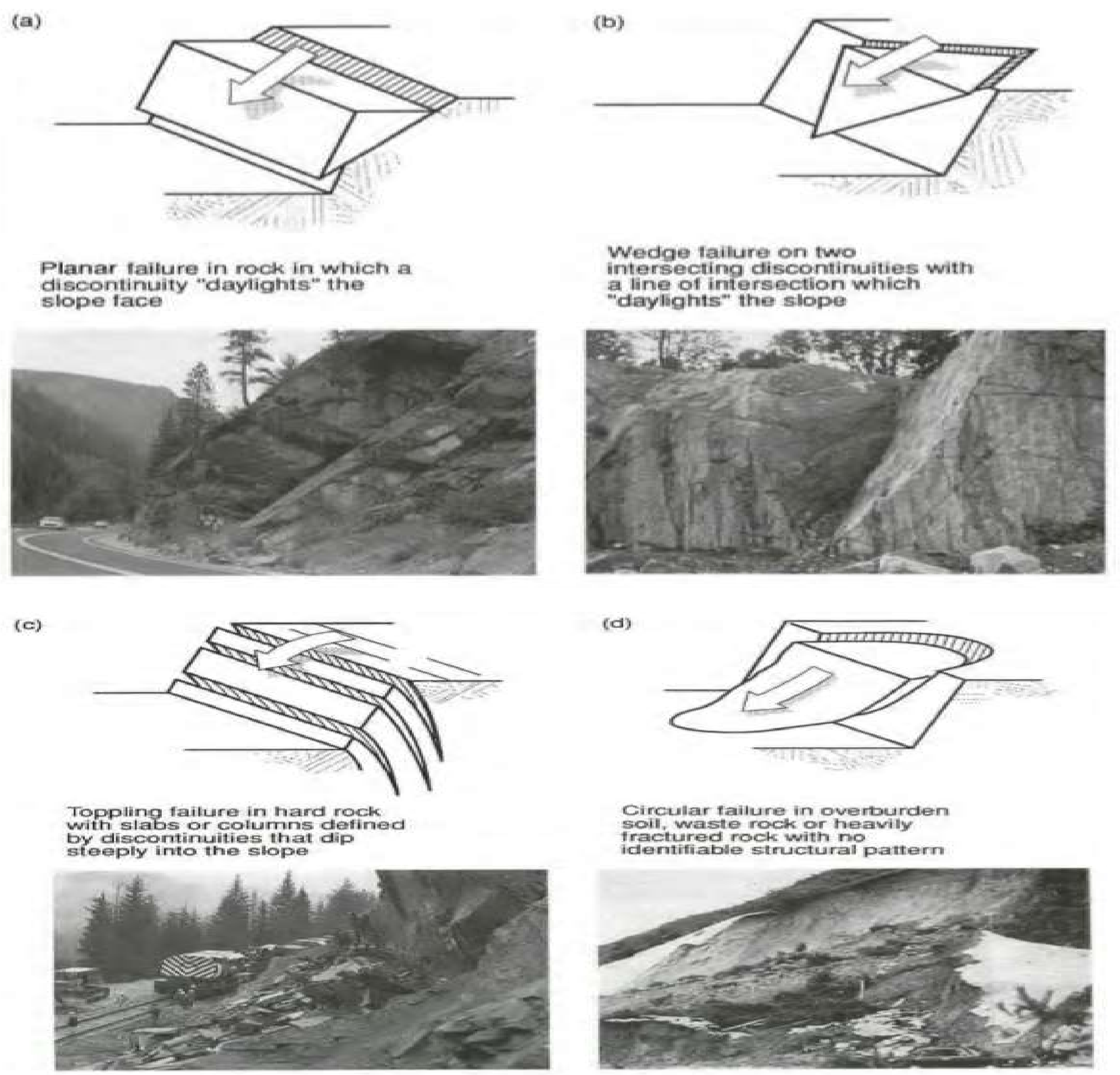


Figure 2. 1 Mechanisms of slope failures.

2.3 Factors Affecting Slope Failure

Many factors affect slope stability. A change in any one or the combination of these factors can alter the steady-state condition of the slope, decreasing its stability and leading to slope failure. When the slope is in a critical state of stability the destabilization can be generated by a relatively sudden triggering event of natural (such as an earth quake, soil saturation) and human events (undercutting slope for construction purpose). The most important factor controlling slope stability is explained here after (Duncan *et al*., 2014) and (Mulatu *et al*., 2011):

Table 2. 1 Factors affecting the stability of a slope.

Factors affecting slope stability	Descriptions
The geometry of the slope	The angle of the slope, slope profiles, and height of the slope.
Rock discontinuities	Fault, Joint, bedding plane properties.
Soil and rock properties	Index and strength properties (grain size, density, Atterberg limit, UCS, and shear strength parameters).
Groundwater and Rainfall	Groundwater conditions, rainfall, drainage pattern, and permeability.
Excavation and human activities	Cutting, drilling, and other human activities.
Dynamic forces	Seismic activity and blasting.

2.3.1 Geometry of the slope

The geometry of the Slope is a factor that plays an important role in the stability of the slope. Slope angle, slope profile, slope height, dip, and dip direction are the common geometrical slope parameters that play a key role in the stability of a slope. Due to the increase in shear stress and a decrease in normal stress on the potential rupture plane steep slopes have a greater risk of instability (Nalgire *et al.*, 2020). Moreover, the inappropriate selection and design of slope profiles (i.e. single slope, benched slope, or multi-slope) lead to slope instability problems. Similarly, the possibility of slope failure or landslide occurrence will increase with increasing slope relative relief or height.

2.3.2 Rock discontinuities

Rock mass strength is most significantly affected by the discontinuity of the rock. Strength parameters of the discontinuities and geometry of the slope are important factors that greatly affect the stability of slopes in the rock masses. Spacing, orientation, persistence, surface characteristics, separation of discontinuity surface, and nature as well as the thickness of material filled in discontinuities are the main characteristics of discontinuities that control the stability of rock slopes. The relationship of structural discontinuities within a slope like parallelism of dip direction and slope as well as the relationship between dip of discontinuity and inclination of slope controls the stability condition of rock slopes.

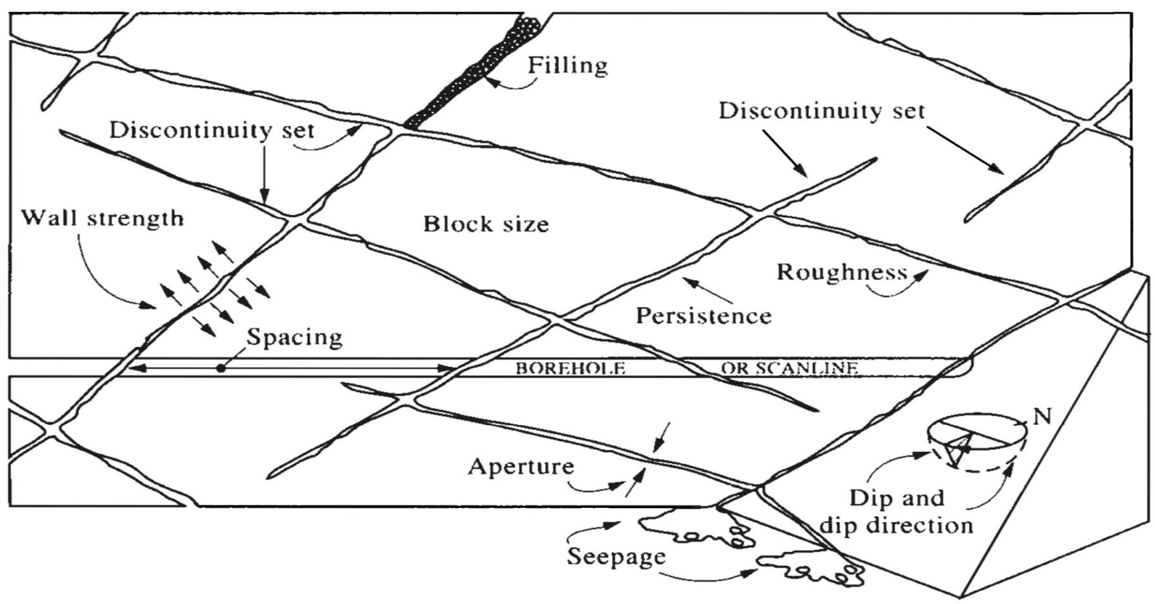


Figure 2. 2 Rock discontinuity characteristics.

2.3.3 Rainfall and groundwater condition

The infiltration of the water into the soil during the rainy season increases the water content of the soil and the pore water pressure of the soil. As the pore water pressure of soil increases, the effective stress of the soil decreases, resulting in a decrease in the shear strength of the soil and the stability of the slope decrease. As the amount of water infiltrates to the soil affects the shear strength of the soil, rainfall seepage is considered to be a factor that causes slope damage. With the water table rising, the safety factor linearly reduces, and it is likely to induce landslide.

An increase in the groundwater table along the potential failure surface reduces the shear strength and lubricates the surface facilitating the sliding process (Ermias *et al*., 2017). The water within rock discontinuity develops water forces in discontinuities leading to rock mass sliding. Whereas groundwater increases in soil slope result development of pore water pressure which reduces the shear strength in the soil mass (Rahardjo *et al*., 2016). Numerous case studies in different places show that rainfall infiltration reduces FS of slope significantly (Kristo *et al*., 2017). Rainfall increases the saturation of earth material which intern increases the pore water pressure.

According to (Hakim *et al*., 2017) and (Chung *et al*., 2019) the intensity and duration of rainfall also play a major role in the mechanism and amount of slide. Small landslides like soil slips and debris flow occur during heavy rainfall of a short period as it affects only the near-surface material. The cumulative effects of a series of storms will affect bedrock and deeper slides in unconsolidated materials. Heavy rainfall increases the water pressure on materials laying on the bed slope temporarily, which is enough to decrease strength and leading to debris slips and flows. Many roadside slope failures like debris/earth slides, debris/earth flows, and rockslides in Ethiopia have occurred during the time of continued and heavy rainfalls (Woldearegay, 2013).

2.3.4 Excavation and human activities

Human activity like improper modifications and excavations of the slope can trigger instability of the slope. Most of the time human activities which can cause the slope to fail are: cutting the slope in an unplanned manner, steepening the slope gradient, improper drilling and additional loading on the mountain toe (Malgot, 2002), and the modification of the natural environment affects the stability condition of slopes.

Barren lands are more prone to instability compared with slopes of thick vegetation cover. The root system of the vegetation improves the shear strength of the slope mass in shallow depth through mechanical reinforcement, anchoring, and compaction but it has no longer an effect on the stability of slides deeper than the penetration depth of the root.

2.3.5 Dynamic forces

Earthquake is the most destructive geological disasters from factors affecting slope instability. When an earthquake occurs, there may be other following secondary disasters such as rockfalls and landslides, that cause a chain of disasters, which threatens capital loss and human lives. Earthquake stability analysis methodology is a time-domain analysis that provides a powerful tool for designing geotechnical structures for earthquakes. However, this method requires appropriate dynamic boundary conditions and reliable constitutional models, which are often not practical (Yang *et al.*, 2018). These difficulties limit the application of the dynamic approach. Because such advanced techniques only work for very large earthquake zone the conventional pseudo-static approach is still widely used for simplified engineering design.

Ethiopia is located in the East African Rift Valley region, which has a history of major earthquakes. This can cause damages to infrastructures of active seismic zones (i.e. Afar Triangle, Main Ethiopian Rift (MER), and Southern Most Rift (SMR)) (Assefa *et al.*, 2016). The cumulative distribution of earthquake damage in Ethiopia from 1960 to 2010 is shown in Figure 2.3.

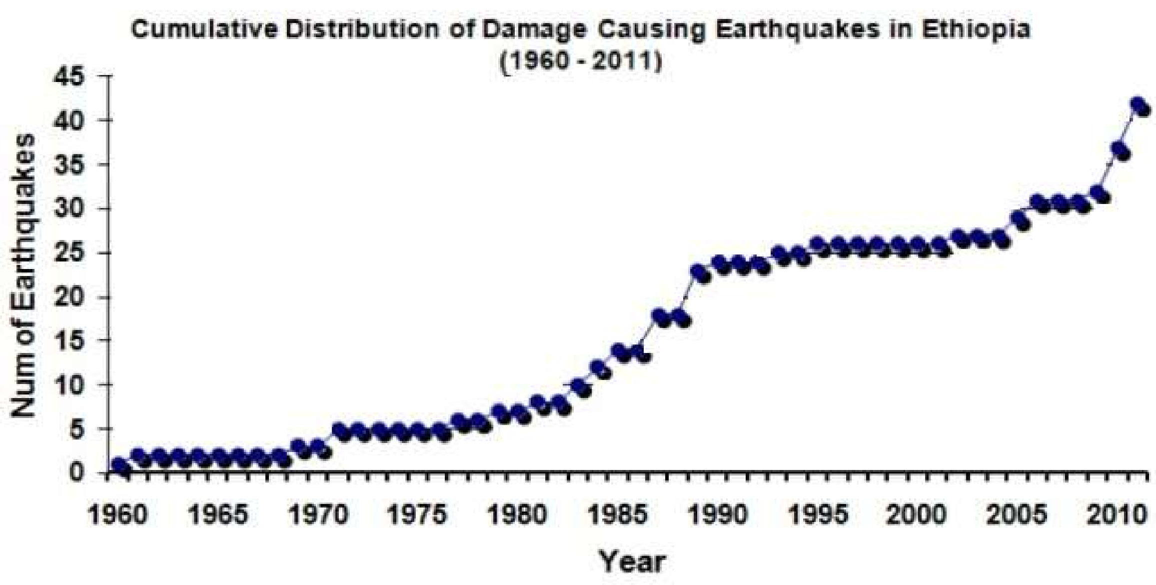


Figure 2. 3 Cumulative distribution of earthquake damage in Ethiopia(Assefa *et al*., 2016).

2.3.6 Shear strength parameter of the slope material

Cohesion and the angle of internal friction are shear strength parameters that are most important to characterize the failure of slope materials. Therefore any reduction of these shear strength parameters will significantly affect the stability of slopes.

1. Cohesion

Cohesion is the measure of the resistance of a rock or soil that is free from intermediate contact. Soil aggregation is the result of an enormous and innumerable resistance each with its own set of resilience values. the property of a rock or soil is measured by how well it withstand damage or breakage by forces such as gravity. Low bonded Slope materials(soil/rock) tend to be less stable. The following factors strengthen the cohesive force of soil/rock materials: a) Friction between particles b) Stickiness of particles can hold the soil grains together. c) Cementation of grains by calcite or silica deposition can solidify earth materials into strong rocks. d) Man-made reinforcements can prevent some movement of material.

The followings are some of the factors that weaken the cohesive strength of soil/rock materials: a) High amount of water content can weaken cooperation because too much water absorbs soil and increase the weight of a soil mass. b) Expansion during wetting and contraction during drying of water reduces the strength of cohesion, just like through expansion by freezing and contraction by thawing. c) Undercutting around the toe of the slopes. d) Earthquakes, sonic booms, blasting create vibrations that overcome cohesion and cause mass movement.

2. The angle of internal friction

One of the parameters of shear resistance of the soils/rock is the internal friction angle. This parameter is the capacity of a unit of soil or rock to resist stress. It is used to define the resistance to friction shear of organized soil with the normal effective stress and is resulting from the Mohr-Coulomb failure criterion. The angle of internal friction of the soil is the angle of an inclination concerning the horizontal axis of the Mohr-Coulomb shear resistance line in the stress plane of effective/normal shear stress. The value of the angle of internal friction is affected by particle roundness and particle size. Lower roundness or larger median particle size results in a larger friction angle.

2.4 Shear Strength Criteria of Soil and Rock

2.4.1 Shear Strength Criteria of Soil

1. Coulomb failure criteria

In Coulomb failure criteria the failure criteria occurs by impending frictional sliding on a slip surface. The soil treated as a rigid, frictional material in this soil failure criteria. Coulomb failure criteria is best used for layered or fissured overconsolidated soils or a soil where a prefailure plane exists. The test data interpretation is from direct shear.

2. Mohr-Coulomb Failure Criterion for Soil

The Mohr-Coulomb criterion is the most common failure criterion encountered in geotechnical engineering. Many geotechnical analysis methods and programs require the use of this strength model. The Mohr-Coulomb criterion describes a linear relationship between cohesion, normal, and shear stresses (or maximum and minimum principal stresses) at failure.

Total stress $$\tau = c + \sigma tan\phi \qquad (2.1)$$

Effective stress $$\tau_f = c' + \sigma' tan\phi' \qquad (2.2)$$

Where c is the cohesion, ϕ is the angle of internal friction of the soil and σ is the applied normal stress. The circle of Mohr touches the fault envelop in the event of a ground feature taken from the fault surface location while the lower feature Mohr's circle taken from a location other than the fault surface location is below the fault envelope. Keeping σ_3 constant, if vertical stress (σ_1) increases the Mohr Circle becomes larger and finally it will touch the failure envelop and failure will take place as shown in figure 2.4.

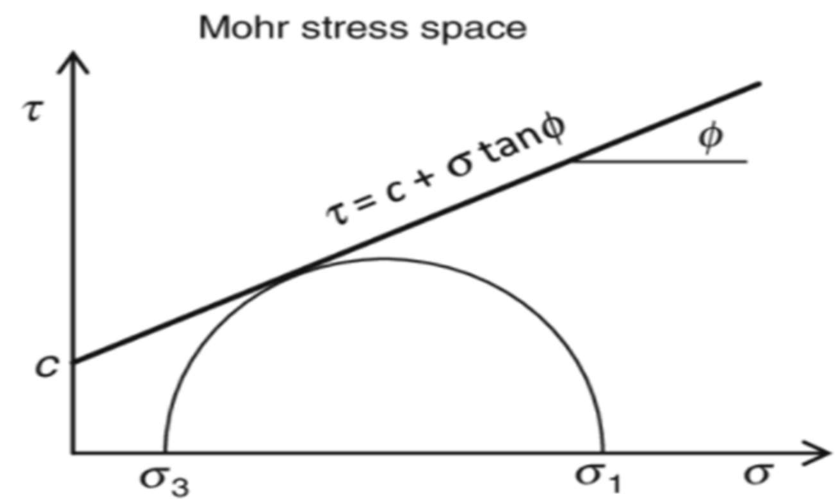


Figure 2. 4 Mohr-Coulomb failure criterion and failure envelop(Labuz and Zang, 2012).

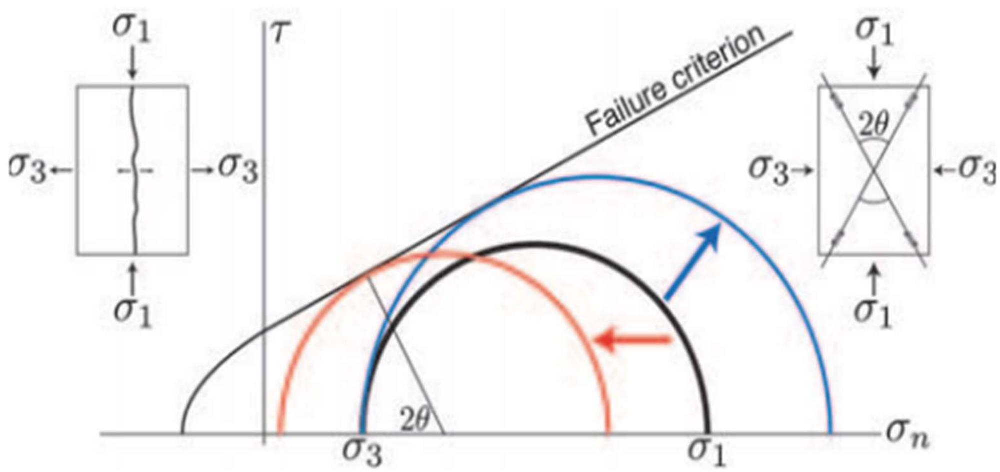


Figure 2. 5 Mohr circles for different stress conditions.

3. Tresca Failure Criteria

In this failure criteria failure occurs when one half the maximum principal stress difference is achieved. Tresca failure criteria treats soil as homogeneous solid. Tresca failure criteria best used for short term (undrained condition) strength of fine grained soils. Triaxial test used for test data interpretation.

4. Taylor Failure Criteria

Failure occurs from sliding (frictional strength) and interlocking of soil particles. Taylor failure criteria treats soil as deformable, frictional solid. It is best suited for short term and long term strength of homogeneous soil. Uses direct simple shear test for test data interpretation.

2.4.2 Shear strength Criterion of rock

Rock slope failures can be formed either along discontinuity surfaces or through the rock mass based on the scale effects and geological conditions of the rock. Therefore, the selection of an appropriate rock shear strength criteria for slope stability depends on the relative scale between the sliding surface and structural geology (Wyllie and Mah, 2017).

If the dimensions of the overall slope are much greater than the discontinuity length, the failure will occur through the jointed rock mass, and the rock mass strength is used to analyze slope stability. On the other hand, slope failures occur along the discontinuities of a single joint if the joint length is nearly equal to the height of the slope, and discontinuity rock strength is used for stability analysis.

2.4.2.1 Shear strength of discontinuities

If geologic mapping identifies discontinuities where shear failure could occur, it will be necessary to determine the friction angle and slip surface cohesion to perform stability analysis. The research program must also obtain information on characteristics of the sliding surface that can alter the shear resistance parameters. Important discontinuity characteristics include continuous length, surface roughness, the thickness and characteristics of any fill, as well as the effect of water on the fill properties.

The rock is assumed to be a Coulomb material in which the shear strength of the slip surface is expressed in terms of the cohesion (c) and the friction angle (φ) in the design of rock slope (Wyllie and Mah, 2017).

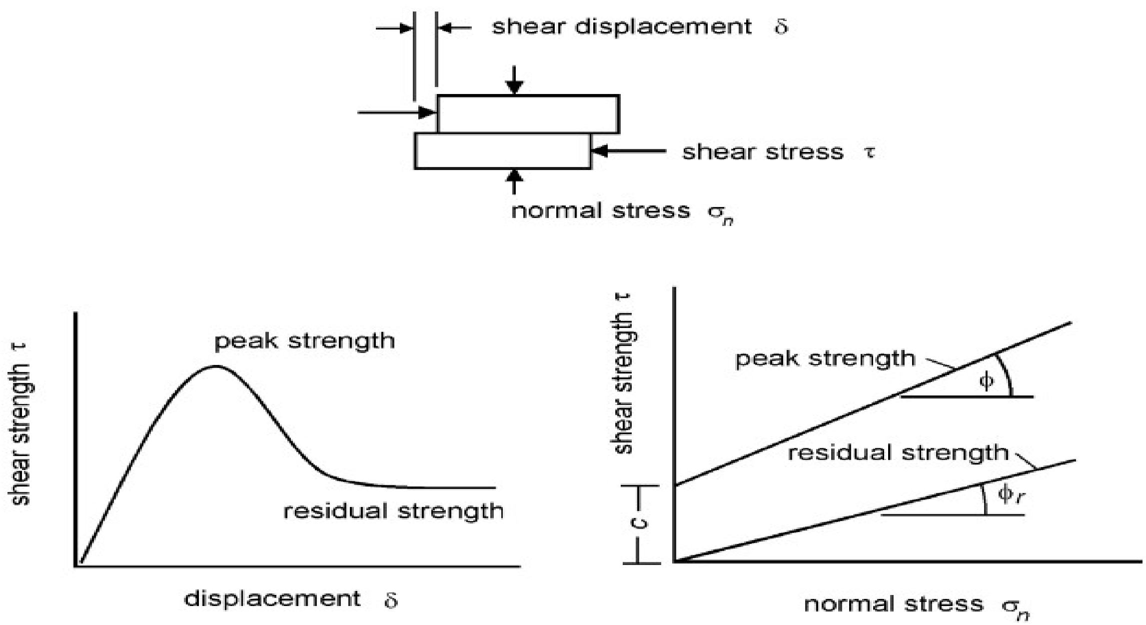


Figure 2. 6 Shear test and its Mohr coulomb shear strength envelope (Wyllie and Mah, 2017).

One of the important factors used to estimate the mechanical behavior of rock masses is the discontinuity of the rock. The variance in the strength and characteristic deformation of failure will cause a difference in the strength and deformation of the rock masses (Miller *et al.*, 2000). There are numerous empirical and theoretical constitutive models to approximate the shear strength of rock discontinuities.

1. Mohr Coulomb's Criterion for Estimating the Shear Strength of Discontinuities

The relationship between the shear strength and the normal stress can be categorized by the Mohr-Coulomb criterion as the following:

$$\tau = c + \sigma_n \tan\phi \tag{2.3}$$

Where, τ, c, σ_n and ϕ are shear strength, cohesion, normal stress, and friction angle respectively. The cohesion, peak, and residual friction angle for each shear test were determined from the Mohr-Coulomb failure criterion.

2. Jaeger's Criterion for Estimating the Shear Strength of Discontinuities

The behavior of a fracture under shear is extremely dependent on the normal stress acting through the crack. The peak shear stress and the residual shear strength increase with increasing normal stress. At higher normal stresses, the fracture has cohesion (c) resulting from the characteristic shear strength of the rough edges and has an effective angle of internal friction.

Jaeger (1971) suggested the following empirical criterion for approximating the shear strength of rock discontinuities, also roughly taking small and large normal stresses into account.

$$\tau = c[1-\exp(-b\sigma_n)] + \sigma_n \tan\phi_r \tag{2.4}$$

Where, τ, c, σ_n, and ϕ_r are shear strength, cohesion, normal stress, and residual friction angle respectively. The value of p and b are obtained from charts.

3. Barton's Criterion for Shear Strength of Discontinuities

Barton's empirical shear strength criterion was used to describe the shear strength of site discontinuities. The three input parameters of JRC, JCS, and residual friction discontinuity angle must be determined to use this principle(Barton's criterion). Barton and his co-workers (1971-1990) suggested the following empirical criterion for estimating the shear strength of rock joints:

$$\tau = \sigma_n \tan[\mathrm{JRC}\ \log_{10}\left(\frac{JCS}{\sigma n}\right) + \ \phi r] \tag{2.5}$$

Where, τ, σ_n, ϕ_r, JRS, and JRC are shear strength, normal stress, residual friction angle, joint compressive strength, and joint roughness coefficient, respectively.

For non-eroded(unweathered) rock fractures, the residual friction angle (ϕ_r) is equal to the base friction angle (ϕ_b), which can be obtained from shear tests on non-eroded smooth joint surfaces. Joint wall compressive strength (JCS) is the strength of a layer of rock adjacent to the joint and it controls the strength and deformation properties of the rock mass (Hoek, 2007). The joint roughness coefficient (JRC) is the coefficient of joint surface roughness having a value ranging from 0 to 20 (smoothest to roughest) as described by Barton.

2.5 Shear Strength for Rock Mass

For the geological conditions a cut has been made in fractured rock, there is no distinct discontinuity surface on which sliding can take place. The shear strength of rock mass can be determined from the generalized Hoek-Brown strength criterion which expresses the shear strength in terms of the major and minor principal stresses and equivalent linear Mohr-Coulomb failure criteria for rock mass.

1. Equivalent linear Mohr-Coulomb failure criteria for rock mass

Slope stability analysis involves examining the shear strength of the rock mass on the slip surface expressed by the Mohr-Coulomb failure criterion. Therefore, it is required to determine angles of friction and cohesion strengths that are comparable between the Hoek Brown and Mohr-Coulomb criteria. These strengths are required for each rock mass and stress range along the sliding surface.

$$\phi' = sin^{-1}\left[\frac{6am_b(s+mb\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+mb\sigma'_{3n})^{a-1}}\right] \tag{2.6}$$

$$c' = \frac{\sigma_{ci}[1+2a]s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)}}} \tag{2.7}$$

Where, $\sigma'_{3n} = \sigma'_{3max}/\sigma_{ci}$

Equation 2.6 and 2.7 is used to determine the Hoeks Brown equivalent Mohr-Coulomb parameters for practical applications. The Mohr-Coulomb shear strength τ, for a given normal stress σ, is found by substitution of these values of c′and ϕ' into the following equation:

$$\tau = c' + \sigma tan\phi' \tag{2.8}$$

2. Hoek-Brown Strength criterion for rock Mass

The Generalized Hoek-Brown criterion is an empirical failure criterion that establishes the strength of the rock in terms of major and minor stresses. It predicts strength envelopes that are in good agreement with values determined from laboratory triaxial tests of intact rock, and defects observed in articulated rock masses. Generalized Hoek-Brown criterion was recently updated and implemented by *RocData* (the 2002 edition). The edition fixes some previously tricky issues, including the applicability of the criterion to very weak rock masses and the calculation of equivalent Mohr-Coulomb parameters from the Hoek-Brown failure envelope. The Generalized Hoek-Brown criterion is non-linear and relates the major and minor effective principal stresses (sigma1 and sigma3) according to the following equation (Hoek, 2007):

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}(m_b\frac{\sigma'_3}{\sigma_{ci}} + s)^a \tag{2.9}$$

where: $\sigma'_1, \sigma'_3, \sigma_{ci}, m_b$ are the axial (major), confining (minor) effective stresses, the uniaxial compressive strength (UCS) of the intact rock material, and a reduced value of the material constant mi (for the intact rock) respectively. a and s are constants that depend on the properties of the rock mass.

In general, it is practically impossible to carry out triaxial tests on rock masses in the scale required to obtain direct values of the parameters in the Generalized Hoek-Brown equation. Therefore a practical way is needed to estimate the material constants mb, s, and a. According to the latest research, the parameters of the Generalized Hoek-Brown criterion [Hoek, Carranza-Torres & Corkum (2002)], are given by the following equations (Miller *et al.*, 2000):

$$m_b = m_i exp\left[\frac{GSI-100}{28-14}\right] \tag{2.10}$$

$$s = exp\left[\frac{GSI-100}{9-3D}\right] \tag{2.11}$$

$$a = \frac{1}{2} + \frac{1}{6}\left[e^{-GSI/1} \quad - e^{-20/3}\right] \tag{2.12}$$

Where: GSI relates the surface condition to the rock structure in the field, mi is intact rock material coefficient, D is a factor that depends on the degree of disturbance with which the rock was excavated. Its value ranges from 0 for undisturbed in situ rock masses to 1 for highly disturbed rock masses.

2.6 Methods of Slope Stability Analysis

Slope stability analysis is performed to evaluate the safe design of natural or artificial slopes and equilibrium conditions. Slope Stability analysis techniques are broadly classified as a conventional method (i.e. kinematic, empirical, deterministic/limit equilibrium, probability methods) and numerical method (continuum, discontinuum, and hybrid modeling) (Mulatu *et al.*, 2011).

2.6.1 Conventional method of slope stability analysis

Conventional slope stability analysis can be generally divided into kinematic, empirical, probability, and limit equilibrium methods. Kinematic methods concentrate on the possibility of translational disturbance because of the formation of wedges or planes of daylight. By themselves, these methods rely on the comprehensive evaluation of rock mass structure and the geometry of existing sets of discontinuities that can contribute to blocking instability. This evaluation may be performed using stereonet plots or specialized computer codes and focuses on the formation of planar and wedge.

A probabilistic approach of stability analysis determines the probability of soil and rock slope failure by considering uncertainty in the input parameter used for stability analyses (Eberhardt, 2003). The method reduces the uncertainty and variation of the input parameter as it takes a range of values of the parameters defined by its probability density function.

However, Limit equilibrium methods are normally used in the landslide analysis where translational or rotational movements arise on different fault surfaces. Limit equilibrium analyses can be very relevant for simple block failure along discontinuities or rock slopes that are severely fractured or eroded (i.e. behave as soil continuum). All LE techniques share a common approach based on a comparison of resistive forces/moments mobilized and the disturbing forces/moments. However methods may vary concerning the slope failure mechanism in question (e.g. translational or rotational slip), and the assumptions used to achieve a determinate solution. Each method available in conventional methods of analysis has certain advantages and limitations as shown in table 2.3 (Eberhardt, 2003).

Hence the selection of these methods must depend on their governing parameters and the condition of the site under consideration. Accordingly, the condition of the study area satisfies the condition of LEM so, the limit equilibrium method is selected for this study, and further description was made in the next paragraphs.

Table 2. 2 Conventional types of slope stability analysis methods(Eberhardt, 2003).

Kinematic method of slope stability analysis	
Critical parameters	Critical slope and discontinuity orientations; representative shear strength characteristics.
Advantages	Comparatively practical provides an initial hint of failure potential, can permit identification and analysis of critical key-blocks using block theory.
Limitations	Only suggest the potential failure (don't provide quantitative stability condition). Only suitable for preliminary design or non-critical slope design.
Empirical method of slope stability analysis	
Critical parameters	Discontiuniuty oreintaation, failure modes, excavation methods
Advantages	Able to investigate the large area in less time.
Limitations	Not suitable for complex geometric and geologic conditions.
Probabilistic method	
Critical parameters	Statistical uncertainity, testing device used, boundary conditions, soil disturbance, models, and correlations used
Advantages	Recognizes uncertainties on leading parameters for a realistic result.
Limitations	Requires a collection of detailed and well distribute data over the slope.
LEM	
Critical parameters	Illustrative geometry and material features; soil or rock mass shear strength parameters, unit weight of the material and ground water table.
Advantages	Simple to apply and gives quantitative stability conditions.
Limitations	Assumptions made on it may lead to nonrealistic results.

2.7.1.1 Limit Equilibrium Method

Limit equilibrium method is often used to perform deterministic slope stability analysis. In LEM of slope stability analysis, FOS is the factor by which the shear strength of the soil or rock mass would have to be divided to bring the slope to a nearly stable state of equilibrium(Duncan, 1996). All LEMs utilize the Mohr-Coulomb criterion to determine the shear strength (τf) along the sliding surface. The shear strength of the soil is the shear stress at which a soil fails in shear.

A state of limit equilibrium is obtained by expressing the mobilized shear stress (τ) as a fraction of the available shear strength (τf) in limit equilibrium analysis. The mobilized shear stress(τ) depends on the external and internal forces acting on the soil mass, however, the available shear strength depends on the type of soil and the effective normal stress. The shear strength is fully mobilized along with the surface of failure when the critical state conditions are reached, at the moment of failure. Mohr-Coulomb linear relationship is used to express the shear strength, where the τ_f and τ are defined by:

Available shear strength: $$\tau_f = c' + \sigma' tan\phi' \qquad (2.13)$$

Mobilized shear stress: $$\tau = \frac{\tau_f}{F} = \frac{c' + \sigma' tan\phi'}{F} \qquad (2.14)$$

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 stability analysis in the limit equilibrium method is through iterative aspect till the critical slip surface is found out, where the slip surface with the lowest FS is the critical slip surface.

There are two methods of slope stability analysis in the limit equilibrium method (i.e mass procedure and method of slices). If the soil that forming the slope is assumed to be homogeneous, taking the mass of the soil above the slip surface as a unit, the mass procedure is used for the stability analysis (Aryal *et al*., 2005). Whereas, in the method of slices the soil above the surface of sliding is divided into several vertical parallel slices and each slice is treated separately. According to the equilibrium conditions considered and the shape of the slipe surface many alternatives were proposed as shown in table 2.4.

Table 2. 3 Types of LEM, equilibrium satisfied, slip surface, and condition suited for.

	The ordinary method of a slice (Fellenius, 1927)
Equilibrium conditions satisfied	Moment equilibrium about the center of the circle.
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.
	Bishop's Modified Method (Bishop, 1955)
Equilibrium conditions satisfied	Vertical equilibrium and overall moment equilibrium.
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.
	Janbu's Generalized Procedure of Slices (Janbu, 1968)
Equilibrium conditions satisfied	Force equilibrium (vertical and horizontal).
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.
	Morgenstern & Price's Method (Morgenstern & Price's, 1965)
Equilibrium conditions satisfied	All conditions of 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.

Spencer's Method (Spencer, 1967)	
Equilibrium conditions satisfied	All conditions of 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.

The method of a slice is a versatile method that takes into account the water pressure of the pores, the inhomogeneity of the soils, and the variation of the normal stress along the potential failure surface. Figure 2.7 shows the method of slices and forces acting on each slice. The difference between different types of limit equilibrium methods stems from satisfied equilibrium (Table 2.3) and the assumptions that make the problem determinate (Table 2.4).

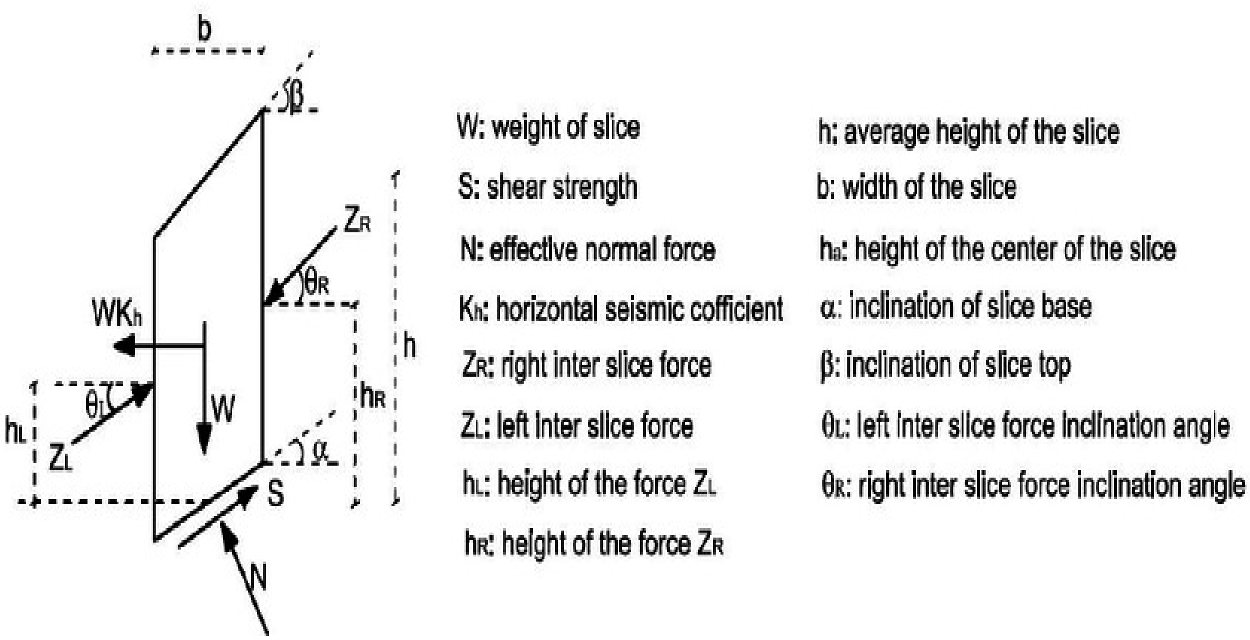


Figure 2. 7 Loads applied on each slice.

In general, limit equilibrium has its advantages and limitations over numerical slope stability analysis methods. The advantage of limit equilibrium over the numerical method of slope stability analysis is, it can provide quick answers and well-understood limit conditions.

Limitations of LEM includes: the use of trial predetermined failure mechanism, its incapability to analyze 3D problems, inability to define the stress, strain, and deformation behavior, its assumption of considering soil mass movement as a rigid block along the failure surface, and local mobilization of shear stresses along the failure surface uniformly are some of the limitations in limit equilibrium method (Burman *et al.*, 2015).

Table 2. 4 Different methods of LEM and their respective assumptions.

Methods	Assumptions
Ordinary or Fellenius	Interslice forces between slices are neglected.
Bishop's Simplified	Resultant forces between slices are horizontal (i.e., there is no interslice shear force).
Janbu's Generalized method	The position of the interslice normal force is defined by an assumed line of thrust.
Morgenstern-Price method	The direction of the resulting interslice is defined by an arbitrary function. The percentage of the functions required to satisfy moment and force equilibrium is computed.
Spencer method of LEM	Resultant interslice forces are of constant slope throughout the sliding mass.

2.7.2 Numerical method of slope stability analysis

Using the conventional method of slope stability analysis would be precisely difficult because of complex geometry, material anisotropy, and loading conditions (Digvijay *et al.*, 2017). These limitations of conventional methods are resolved by the numerical method of slope stability analysis (i.e continuum, discontinuum, and hybrid numerical methods). Continuum modeling (e.g. FEM, FDM) of numerical slope stability analysis is suitable for soil slopes, heavily disintegrated rock mass considering them continuous all over the slope.

In the continuum modeling approach of the whole mass is discretized to a finite number of elements with the help of generated mesh through the finite element method. In discontinuum approach (e.g. discrete element and distinct element) the discontinuity of the rock is the controlling factor and this method is used for the analysis of slopes whose failure mechanism is controlled by former(pre-existing) discontinuities. Whereas, the hybrid approach of numerical slope stability analysis considers the combined effect of both continuum and discontinuum approach of slope stability analysis (Digvijay *et al.*, 2017). Different types of numerical method have their advantages and limitation as described in the table 2.5 as shown below.

Table 2. 5 Advantages and limitations of numerical slope stability analysis.

Methods	Advantages	Limitations
Continuum method	Allows for material deformation and failure(incorporated FOS concepts); can model complex behavior and mechanisms; able to assess effects of parameter variations; computer hardware advances allow complex models to be solved with reasonable run times.	Users must be properly trained; be aware of model and software limitations (for example; contour effect, meshing errors and time constraints); generally poor input data availability; inability to model effects of highly jointed rock.
Discontinuum modeling	Allows for block deformation and movement of blocks relative to each other; can model complex behavior and mechanism; able to assess effects of parameter variation on instability.	In addition to the limitations listed above need to simulate representative discontinuity geometry; limited data on jointed properties.
Hybrid modeling	Coupled finite element models capable of simulating intact fracture propagation and fragmentation of jointed and bedded rock.	Complex problems require a large memory capacity; relatively little practical experience in use.

2.7.2.1 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). 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:

$$SRF = \frac{tan\phi'}{ta\quad {}'_{f}} = \frac{c'}{c'_f} \qquad (2.15)$$

Where ϕ'_f and c'_f are reduced shear strength parameters, $\phi' and$ c' are original shear strength parameters and SRF is the strength reduction factor. The value of SRF at failure is equal to the factor of safety of the slope. c and ϕ are reduced by the same factor as the shear strength reduction method generally uses the same SSR factor for all material and all strength parameters. The strength reduction method uses a constant stiffness modulus from the previous step during computations and it has the same behavior as the Mohr-Coulomb model where a constant stiffness modulus is used (Lamessa, 2019).

2.8 Seismic Slope Stability Analysis

Seismic loading is one of the important external causal factors triggering slope instability. Three methods developed to date to assess the stability or performance of slopes during earthquakes. These methods are; Pseudo-static analysis, stress-deformation analysis, and permanent displacement analysis. Each of these stability analysis methods has strengths and weaknesses, and three of them can be applied correctly in different situations. A pseudo-static method is the simplest method of seismic slope stability analysis to calculate the FS.

2.8.1 Pseudo static analysis

The pseudo-static method of analysis simply extends the static analysis and determines the FS of slope subjected to a static horizontal acceleration. Pseudo-static analysis models the seismic quaking as a permanent body force that is added to the force body diagram of conventional static limit equilibrium analysis. Since the effects of vertical forces tend to average out to near zero, only the horizontal component of earthquake shaking is modeled. In the analysis, the horizontal inertia force persuaded by an earthquake is characterized by a static horizontal force as a product of a seismic coefficient k and the weight of the potential sliding mass. Based on the main principle of the limit equilibrium method, the critical acceleration Ac is derived as that which produces an FoS of 1. The main advantages of this approach are that it uses the established limit equilibrium method and that is easy to use. However, the transient nature of seismic movement cannot be measured using the pseudo-static approach.

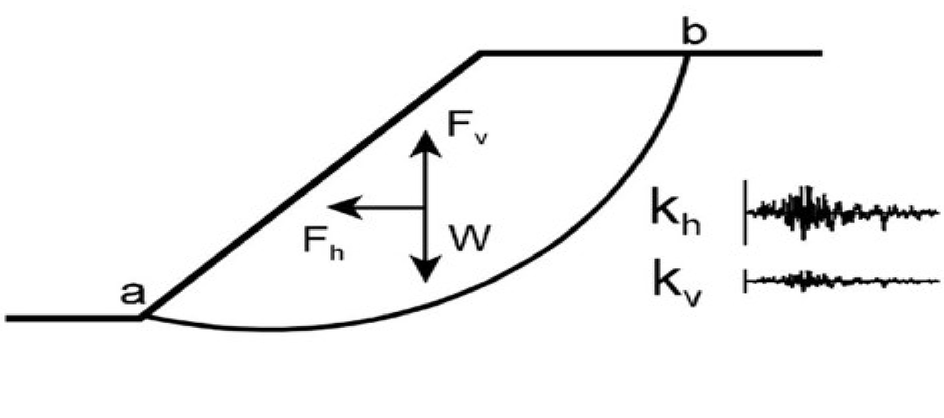


Figure 2. 8 The effect of seismic forces acting on on the slope section.

$$FS = \frac{Resisting\ Forces}{Driving\ Forces} = \frac{cl + [(W - f_v)cos\beta - f_h sin\beta]tan\phi}{(W - f_v)sin\beta - f_h cos\beta}$$

$$f_h = \frac{\alpha_h W}{g} = k_h W$$

$$f_v = \frac{\alpha_v W}{g} = k_v W$$

Where; c and $\emptyset$ are shear strength parameters l is the length of the failure plane, β is the angle of failure plane from the horizontal, f_v and f_h vertical and horizontal pseudo-static forces, k_v and k_h dimensionless vertical and horizontal pseudo-static coefficients, α_v and α_h vertical and horizontal pseudo-static accelerations and W is the weight of failure mass.

2.8.2 Stress deformation analysis

First established by Richard Courant (1943) and this method was founded on mathematical methods. Stress deformation analysis is a method where finite element modeling uses a mesh to model a deformable system, as the deformation at each node of the mesh is calculated in response to applied stress (Jibson, 2011). One of the benefits of this modeling method is that it gives the most precise picture of what occurs in the slope during an earthquake. Stress deformation modeling procedures have drawbacks due to (1) the need to select a set of input ground motions; (2) the required data acquisition, which typically includes undisturbed soil sampling and extensive laboratory testing of samples; (3) the need for an accurate, nonlinear, stress-dependent, cyclic model of soil behavior; and (4) computational requirements. The other challenge of this method is that most procedures model only distributed deformation, and the amount of deformation is limited.

2.8.3 Permanent displacement analysis

Introduced by Nathan Newmark (1965) to assess the performance of slopes during earthquakes that remove the gap between pseudo-static analysis and complex stress deformation analysis.

Newmark's method models a landslide as a rigid block that slides on an inclined plane; the block has a known yield or critical acceleration(i.e acceleration required to overcome basal resistance and initiate sliding).

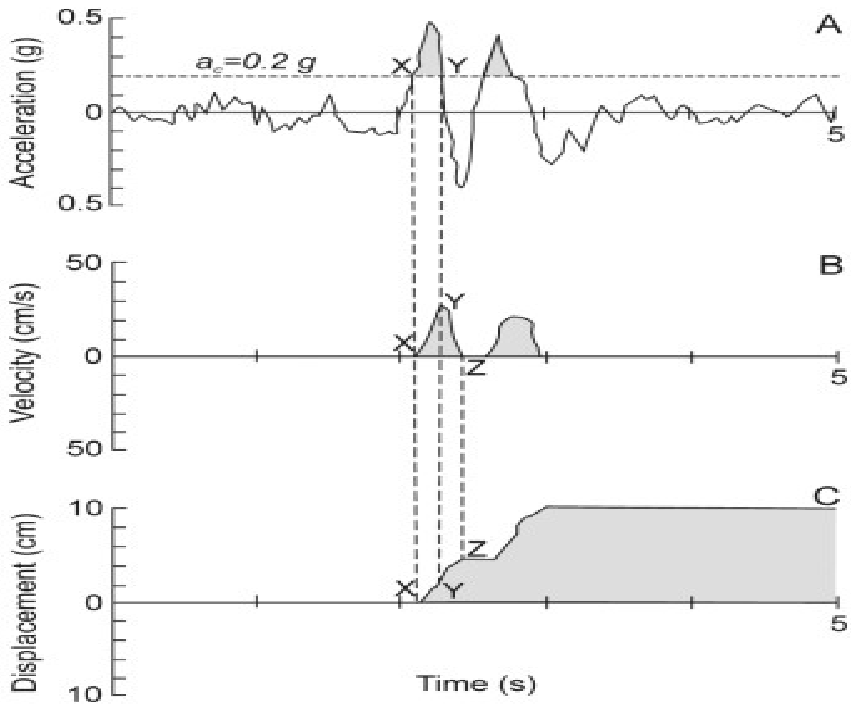


Figure 2. 9 Illustration of the Newmark integrations algorithm. A, B and C are Earthquake acceleration time history with critical acceleration (dashed line) of 0.2 g superimposed, the velocity of the landslide versus time, and displacement of landslide versus time respectively. Points X, Y, and Z are for reference between plots.

The main assumption of Newmark's method is that the mass does not undergo internal deformation, it regards a landslide as a rigid-plastic body, it does not experiences permanent displacement under accelerations below the critical level, and it deforms plastically at constant stress along a discrete base shear surface when the critical acceleration is exceeded.

2.9 Slope stabilization methods

Whenever the result from the stability analysis of the slope does not meet the minimum factor of safety requirement, then it may be necessary to use slope stabilization methods.

Slope stabilization improves the factor of safety either by reducing driving force or increasing resistive force (Popescu, 2002). The choice of an appropriate corrective measure depends on: technical feasibility, economic feasibility, legal/regulatory conformity, social suitability, and environmental acceptability.

Among the various remedial measures for slope instability problems, the common techniques include; slope geometry modification, drainage, support structures, and internal slope reinforcement. Modification of slope geometry includes removing material from the area driving the landslide (with possible substitution by light weight fill), then adding material to the area maintaining stability (counter weight berm or fill). This method also includes reducing general slope angles and using different slope angles (i.e benched slopes).

Drainage is used for controlling surface and subsurface water conditions which is the main cause of slope failures. This method is a crucial remedial measure due to its important role in reducing shear strength parameters by pore water pressure. Surface and groundwater drainage is the most widely used, due to its high stabilization efficiency, and cost stability, and is generally the most successful stabilization method (Norfush, 1992). By using ditches and pipes Surface water is diverted from unstable slopes to make the slope stable, whereas drainage of the shallow ground water is usually accomplished by networks of trench drains. In the case of deep landslides, the groundwater will be depressed by driving drainage tunnels into the intact material underneath the landslide.

Retaining structures such as gravity retaining walls, crib-block walls, gabion walls, passive piles, piers/caissons, cast-in-situ reinforced concrete walls, and reinforced earth retaining structures are used for slope stabilization. On the other hand, the reinforcement of slope increases the stability of steep slopes by providing tensile reinforcement, erosion protection, and strength improvement. Internal slope reinforcements such as rock bolts, micro piles, soil nailing, anchors, grouting, stone columns, freezing, electro-osmotic anchors, vegetation planting, and geotextile are used to improve cut or embankment slopes.

2.10 Previous related works

(Addisalem, 2017) conducts slope stability analysis of rainfall-induced landslides on Abay gorge. LEM slope stability analysis using SLOPE/W was made and the pore water distribution is conducted using seep/w. Consequently, loss of suction due to rainfall infiltration is critical for the instability of slopes. The result displays reduction of FS from 1.506 to 1.29 due to rainfall infiltration from its initial dry state.

(Atalla *et al.*, 2008) studied stability analysis of slopes using the finite element method and limiting equilibrium approach. The factor of safety is determined using the strength reduction approach and Mohr-Coulomb failure criteria are used to analyze the slope. Finally, the study shows that the differences in the values of safety factors achieved for different conditions were found to be small. The critical slip surfaces obtained using both methods were compared well. Finally, this study concluded that because of the differences in the values of safety factors obtained, it is recommended that the engineer should analyze critical slopes using both FEM and LEM.

(Bushira, 2018) studied cut soil slope stability analysis along national highway at Wozeka–Gidole road, Ethiopia. The comparative study of the LE methods shows that the Bishop, Janbu, Morgenstern-Price, GLE, and Spencer methods produces the same FS for circular sliding surfaces in most cases. However, the ordinary method can underestimate the FS. Except for the ordinary methods, the FS estimated by all LE methods are higher than FE analysis in PLAXIS. The numerical results of FEM and LEM confirm the instability of the slope. PLAXIS and GEO-SLOPE modeling results show that the slope falls during the rainy season when the soil is completely saturated. The results of FS also show that the slope is likely to fail with produced slip surface even without reaching the fully saturated condition. From these two results, it can be concluded that with the prevailing condition the cut slope was unstable which attested to the situation at the site, and the model provided the possible main causes of slope instability in the study area and their possible remedial measures.

(Burman *et al.*, 2015) conducts a comparative study LE and FEM in solving slope stability problems. Simple single-layered slope, a slope having multi layers, and homogenous single-layered slope are used to consider the effect of rapid draw down. FS was determined for both LE and FEM for different conditions and the comparison of LE and FEM shows that the FS values agree very well. The strength reduction method is used for the analysis in FEM. The study describes that in LEM, several slip surfaces should be analyzed to find the critical slip surface that is not considered in FEM. Additionally, FEM satisfies the equations of equilibrium and compatibility equations from the theory of elasticity. It gives displacements, stress, and strains at several nodes in the slope domain that show few of the further benefits of using the finite element method.

(Lamessa, 2019) studied the slope stability in the road from Gutane Migiru to Finca'a sugar factory. The study recognizes wedge and planar failure as the main modes of rock failures from kinematic analyses. The factor of safety and probability of failure was calculated using Rocplane and Swedge software for planar and wedge mode of rock failure respectively. In the same way, the stability analysis of soil slopes was carried using slide software. The deterministic and probabilistic stability analysis displays that both critical rock and soil slopes were unstable under dynamic saturated conditions as compared to slopes in the dry conditions. Moreover, the study identifies rainfall, groundwater, and unfavorable orientation of rock discontinuities as contributory factors for critical slope failures. Based on the result analyses, (Lamessa, 2019) suggested that it is better to use proper remedial measures such as rock bolt, Anchor, soil nail, and proper drainage system in the critical slope failure sections.

(Tran *et al.*, 2015) studied the effect of Extreme Rainfall on Cut Slope Stability: Case Study in Yen Bai City, Viet Nam. The FS was calculated using Bishop's simplified method. The impact of extreme precipitation, initial soil conditions, and soil permeability on the changes of pore water pressure within the typical slope is analyzed, subsequently determining the impact of these changes on the slope stability factor of safety. The results show that when the maximum infiltration capacity of the slope top soil is less than the rainfall intensity, slope failures may occur 14 hours after the rain starts. And when this happens, the rainfall duration is the decisive factor that affects the slope FS values.

(Jampani and Bhupathi, 2017) conduct the stability analysis of slope with different soil types and its stabilization techniques. A slope of 30 m height and 50^{o} inclination is analyzed with different soil types i.e., clay, silty clay, and gravel to find out the best slope stabilization technique for a particular soil type. The effect of surcharge, water table, and seismic load on the slope's stability is analyzed. Stability analysis is accomplished using SLOPE/W (Geostudio 2007). Stabilization techniques used in the study are berming, soil nailing, and a combination of both. The effect of berm angle, berm width, soil nail angle, length, and spacing along with surcharge and water table is analyzed. Seismic analysis is performed using the Pseudo-static method with horizontal and vertical seismic coefficients of 0.18 and 0.09 respectively. The results show that the effect of surcharge on slope stability is insignificant in comparison with a water table and seismic load. Effect of water table & seismic was more pronounced in gravel. From the results, it also concluded that soil nailing is an appropriate slope stabilization practice for gravelly soil and a combination of berming and soil nailing technique is sensible in clayey soil.

(You *et al.*, 2018) studied FE analysis incorporating the SSR method was applied to study the west slope stability of an open cut mine in Australia using Mohr-Coulomb and generalized Hoek Brown criteria. The pit of the mine has multiphase excavations and reached 180 m in depth. Three slope arrangements namely, Stage 1 inter ramp slope 43°, Stage 2 inter ramp slope 49°, and optimized Stage 2 slope 54° are examined. The equivalent factor of safety was 1.96, 1.87, and 1.40 under dry slope for the three configurations, respectively when implementing the generalized Hoek– Brown failure criterion. Nevertheless, under partially saturated conditions, the optimized slope would have a factor of safety 1.16. The difference is related to an overestimate of the shear strength parameters by the linear Mohr-Coulomb criterion under low confining stresses related with the non-linear Hoek Brown.

2.11 Summary of previous related works

Table 2. 6 Previous related works with their respective title, methods, and findings

	Author: (Addisalem, 2017)
Title	Slope stability analysis of rainfall-induced landslides on Abay gorge
Methods	LEM slope stability analysis using SLOPE/W and pore water distribution is conducted using seep/w.
Findings	The result shows a reduction of FS from 1.506 to 1.29 due to rainfall infiltration from its initial dry state.
	Authors: (Atalla *et al.*, 2008)
Title	Stability analysis of slopes using the finite element method and limiting equilibrium approach.
Methods	SSR approach and Mohr-Coulomb failure criteria is used to analyze the slope using LEM and FEM
Findings	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. Because of the differences in the values of safety factors obtained, it is recommended that the engineer should analyze critical slopes using both FEM and LEM.
	Author: (Bushira, 2018)
Title	Cut soil slope stability analysis along National Highway at Wozeka–Gidole Road, Ethiopia.
Method	LEM-based geo-slope and FEM-based plaxis software are used.
Findings	The Bishop, Janbu, Morgenstern-Price, GLE, and Spencer methods produce the same FS for circular slip surface in most cases. 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 main resaons for the possible slope instability in the study area and possible remedial measures are provided.

Authors: (Burman *et al.*, 2015)	
Title	Comparative study LE and FEM in solving slope stability problems.
Methods	LEM and FEM using SSR method and Mohr Columb failure criteria.
Findings	The FS values obtained using LEM compare very well with those obtained from FEM. FEM satisfies the equations of equilibrium and compatibility equations from the theory of elasticity and displacements, stress, and strains at various nodes in the slope domain are also obtainable that are some benefits from FEM.
Authors: (Tran *et al.*, 2015)	
Title	Effect of Extreme Rainfall on Cut Slope Stability: Case Study in Yen Bai City, Viet Nam.
Methods	The FS was calculated using Bishop's simplified method from LEM.
Findings	The results show that when the maximum infiltration capacity of the slope top soil is less than the rainfall intensity, slope failures may occur 14 hours after the rain starts. And finally concluded that cut slopes in Yen Bai may be stable in normal conditions after the excavation, but under the influence of tropical rain storms, their stability is always questionable.
Authors: (Jampani and Bhupathi, 2017)	
Title	The stability analysis of slope with different soil types and its stabilization techniques.
Method	Stability analysis is performed using SLOPE/W of LEM and Stabilization techniques used in the study are berming, soil nailing, and a combination of both. The pseudo-static approach is used for Seismic analysis.
Findings	The results indicate that the effect of surcharge on slope stability is negligible in comparison with a water table and seismic load. Effect of water table & seismic was more pronounced in gravel. Based on the results it can also be concluded that soil nailing is an appropriate slope stabilization technique for gravelly soil and a combination of berming and soil nailing technique is advisable in clayey soil.

CHAPTER THREE

MATERIALS AND METHODS

3.1 General

This section describes the description of study area and process to go through to achieve the general and specific objectives of the study. To achieve the objectives of the study the following processes are undertaken, first, a detailed literature review was made to understand: the deep knowledge of the area, to know about the parameters to be used, methods to be followed, and causes and mechanism of slope failure as well as possible remedial actions. Identifying the critical slope sections through visiting the study area, where critical slope sections are highly affected parts of the slope sections. Based on the field manifestations of instabilities, four potentially unstable slopes were identified in the study area. Information of the material from the selected critical slope sections such as critical slope location (Northing, easting, and elevation), strike, dip, and dip direction, the geology of the rock, and rock mass characteristics was identified. Finally, soil and rock samples are taken from the selected critical sections as per the rule and regulations of soil testing standards for laboratory investigation.

3.2 Description of the study area

Ambo-Gedo road is the main route that connects western Ethiopian cities with the capital of Addis Ababa. This road is found in central Ethiopia, Oromia region west Shoa zone and the area of this study is mainly around Guder. Guder is a town located in the West Shewa Zone of the Oromia region of Ethiopia. Located 12km west of Ambo and 131.3km from the capital of Addis Ababa, this town has a latitude and longitude of 8° 58' 14.07" N and 37° 45' 39.4884" E respectively, with an elevation of 2101 meters above sea level.

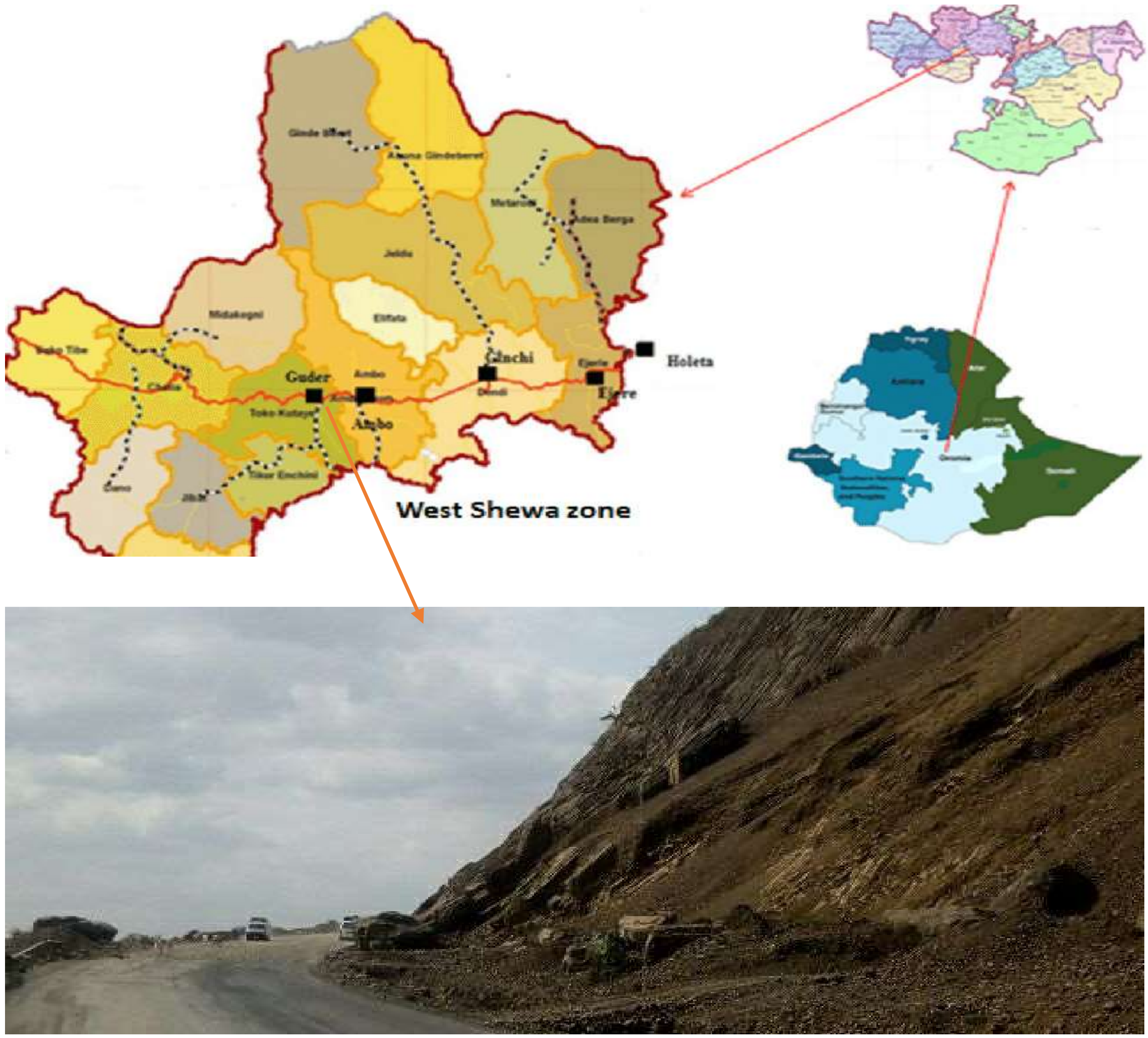


Figure 3. 1 Detail of the study area (Rock slope section one)

Table 3. 1 Location and elevation of selected critical slope sections

Section	Easting	Northing	Elevation(m)
SCSS1	36º22′51″	99º17′04″	2160
SCSS2	36º23′31″	99º16′86″	2169
RCSS3	36º24′32″	99º14′46″	2145
RCSS4	36º18′48″	99º21′32″	2182

3.3 Rainfall of the study area

Many soil and rock slope failures in Ethiopia are associated with rainfall. Rainfall increases the saturation of earth material which intern increases the pore water pressure. Rainfall and groundwater cause slope failure by: a) altering the cohesion and frictional parameters and b) reducing the normal effective stress. Groundwater causes increased up thrust and driving water forces and hurts the stability of the slopes.

Physical and chemical effects of pore water pressure in joints filling material can thus alter the cohesion and friction of the discontinuity surface. The study area has high rainfall intensity during July and august which reduces the shear strength of slope material leading seepage related instability problems. The records of the Ambo meteorological service shows that the rainy period of the year lasts for 9.1 months, from February 5 to November 8, with a sliding 31-day rainfall of at least 0.5 inches. The most rain falls during the 31 days centered around August 4, with an average total accumulation of 11.5 inches. The rainless period of the year lasts for 2.9 months, from November 8 to February 5. The least rain falls around December 17, with an average total accumulation of 0.2 inches.

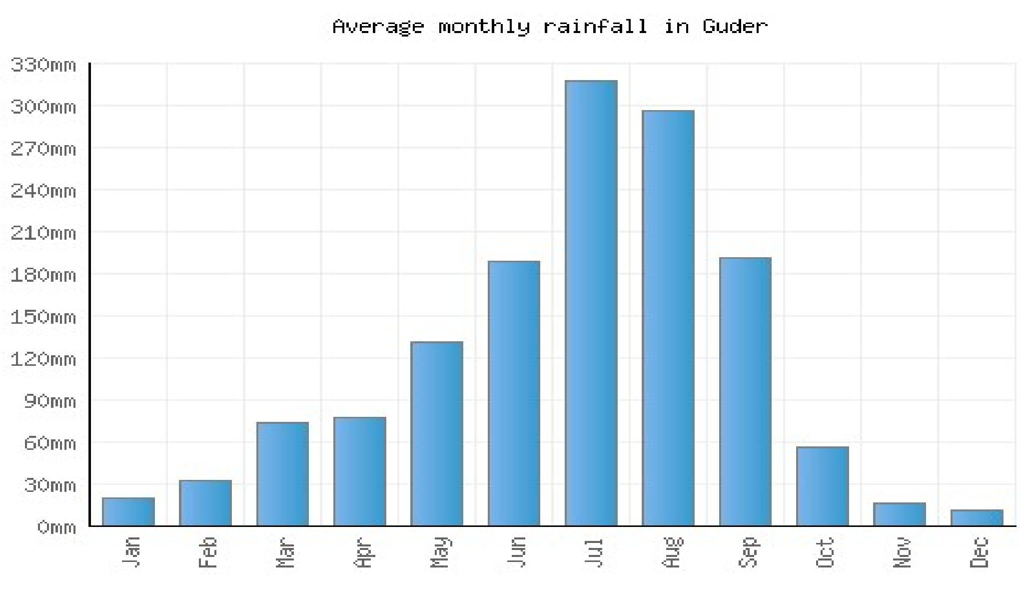


Figure 3. 2 Average monthly rainfall distribution of study area

3.4 Seismicity of the study area

Earthquake-induced ground shaking is the cause of slope failures in marginally to moderately stable slopes before the earthquake. According to (Jibson, 2011) seismically triggered slope failures may occur due to an increase in driving movement or the reduction of shear strength. During earthquake shocks, particularly in a shorter duration of larger acceleration the horizontal dynamic component of the force increases which increases driving forces.

According to the Ethiopia Building Code of Standard (EBCS - 2015) divides the country into six major earthquake zones which range from the no-risk zone (zone 0) to zones of major damages (zone 4 and 5). The study area falls in zones of low risk (zone 1) in earthquake with an acceleration of 0.03 to 0.06g (EBCS - 2015).

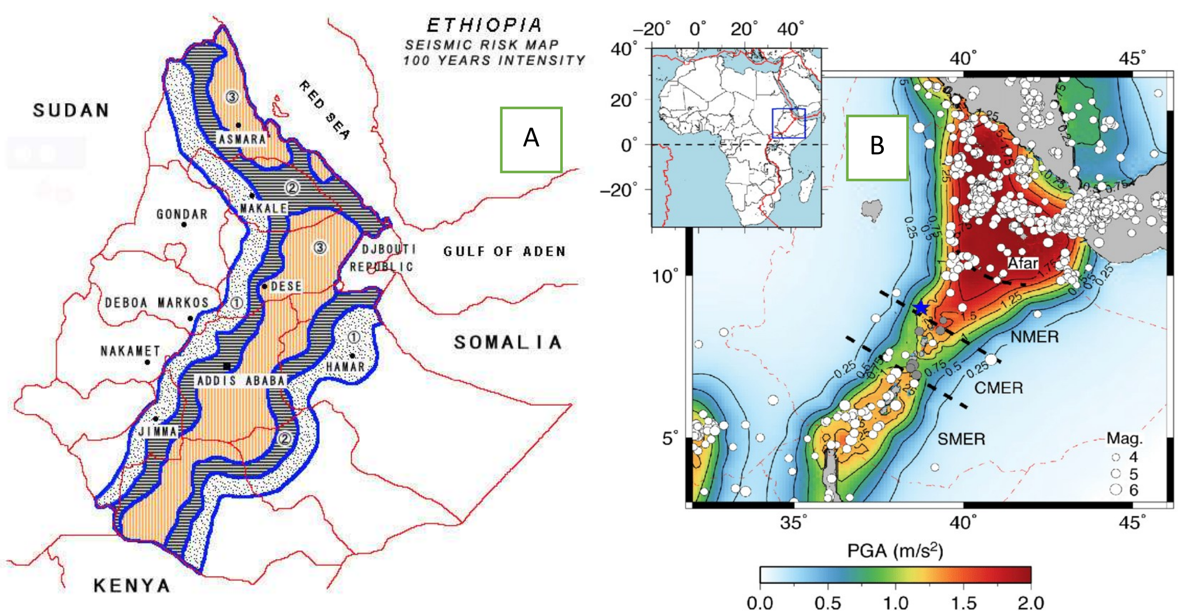


Figure 3. 3 Seismic hazard map of Ethiopia: A) for100-year return period as per EBCS 8: 1995, B) based on the GSHAP data for a return period of 475 years (Worku, 2011)

3.5 Material and Methodology

This section emphasizes the research methodology followed to achieve the general and specific objectives of the study. First, a systematic literature review was made on existing literature and theories to get a better understanding of the cause and mechanism of slope failure, methods of stability assessments, and possible remedies as described in chapter 2. Then various data needed for the stability analysis of the cut slope were collected from different sources such as from organizations, through literature review, field investigation, and laboratory work.

3.5.1 Data collection

Both primary and secondary data collected in different ways are used in this study. The data collected for this study consists of both qualitative data and quantitative data. Successive field investigations are carried out to obtain information about the slope sections. During the field investigation information on location, geological structures of the site, discontinuity, and geometry of the critical slope sections were identified. Data collection was made through observation, interview, and measuring (location, geometry, and discontinuity data). Then after the representative soil(disturbed) and rock (irregular shape) samples are taken from selected critical slope sections to the laboratory investigation to get the index and strength parameters of the materials. The laboratory test results are used to: classify the soil, know the specific gravity of the soil, the natural water content of the soil, rock density to calculate rock unit weight and shear strength parameter of the soil and rock.

Secondary data such as groundwater table of the site, geological data, earthquake data, stiffness parameters of the soil, and rocks are obtained from correlations and pieces of literature. Rock data software is used to determine strength and stiffness parameters. The zone and acceleration coefficient of earthquake for the study area was adopted from the seismic risk map of Ethiopia and Ethiopian building codes of standard (EBCS-2015) as well as the rainfall data from Ambo meteorological agency.

3.5.2 Method of slope stability Analysis

Slope stability analysis susceptible to different types of faults can be performed with different techniques. The choice of a suitable technique is, therefore, a very important factor in the slope stability evaluation process. In this research work, slope stability analysis and comparative study of slope angles are carried out by LEM and FEM, from conventional and numerical slope stability analysis methods respectively using data collected from different sources.

Method of slices is used in LEM that, the soil above the surface of sliding is divided into several vertical parallel slices and each slice is treated separately. Also, the SSR method is used in FEM in which 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. Mohr-coloumb model is used for slope stability analysis. Detail of the LEM and FEM is discussed in chapter two.

3.5.3 Conditions for which stability analysis was made

In this research work, the factor of safety and deformation of the cut slope was determined for dry and wet conditions. Both conditions are discussed in the next paragraph.

Dry condition: Dry condition is the condition by considering that the slope section is completely dry, stability analysis for this condition is performed under both static and pseudo-static state. This condition defines that the water within the slope, if any, will not contribute to any kind of destructive water forces within the slope and may not contribute towards the instability of the slope.

Wet condition: This condition is the condition that represents when the slope is saturated to some degree with water. This situation may occur during very heavy sustained rain in the area. The groundwater will be fully recharged and the slope material will be saturated. Under such a situation the water forces developed within the slope material will be destructive and will induce total instability condition in the slope. For this condition stability analysis is performed in both static and pseudo-static situations.

3.5.4 Software application

Finite element-based plaxis 2d and limit equilibrium-based slide software's are used for this research work.

Plaxis 2D: Plaxis is a finite element program for geotechnical applications, where soil models are used to simulate soil behavior. The software can simulate soil and rock layers, structures, construction phases, loads, and boundary conditions.

Plaxis is a finite element software package designed for the 2D or 3D analysis of deformation, stability, dynamics, and groundwater flow in geotechnical engineering. Geotechnical engineering applications require advanced constitutive models to simulate the non-linear, time-dependent, and anisotropic behavior of soils and/or rock. In addition, since the soil is a multi-phase material, special procedures are required to deal with pore pressures and (partial) saturation in the soil. Although soil is an important issue in itself, many geotechnical projects involve the structural modelling, and the interaction between the structures and the soil. Plaxis has the ability to handle all aspects of complex geotechnical structures (Plaxis, 2013). Considering the slope material Mohr Coulomb's model was selected for this analysis using a "phi-c reduction" approach.

Slide: Slide is a powerful, easy-to-use 2D limit equilibrium based slope stability analysis program suitable for all types of soil and rock slopes, embankments, earth dams, and retaining walls. Slide includes built-in finite element groundwater seepage analysis, probabilistic analysis, multi-scenario modeling, and supports design. The slide can analyze a single user-defined non-circular slip surface and find for the minimum non-circular failure surface. It is also possible to analyze composite surfaces that contain circular and non-circular components. It uses a variety of widely used limit equilibrium analysis methods (such as the Bishop, Janbu, Spencer, and GLE methods) to calculate the safety factors for circular and non-circular slope failure surfaces (Rocscience, 2002)

General work flow phase one

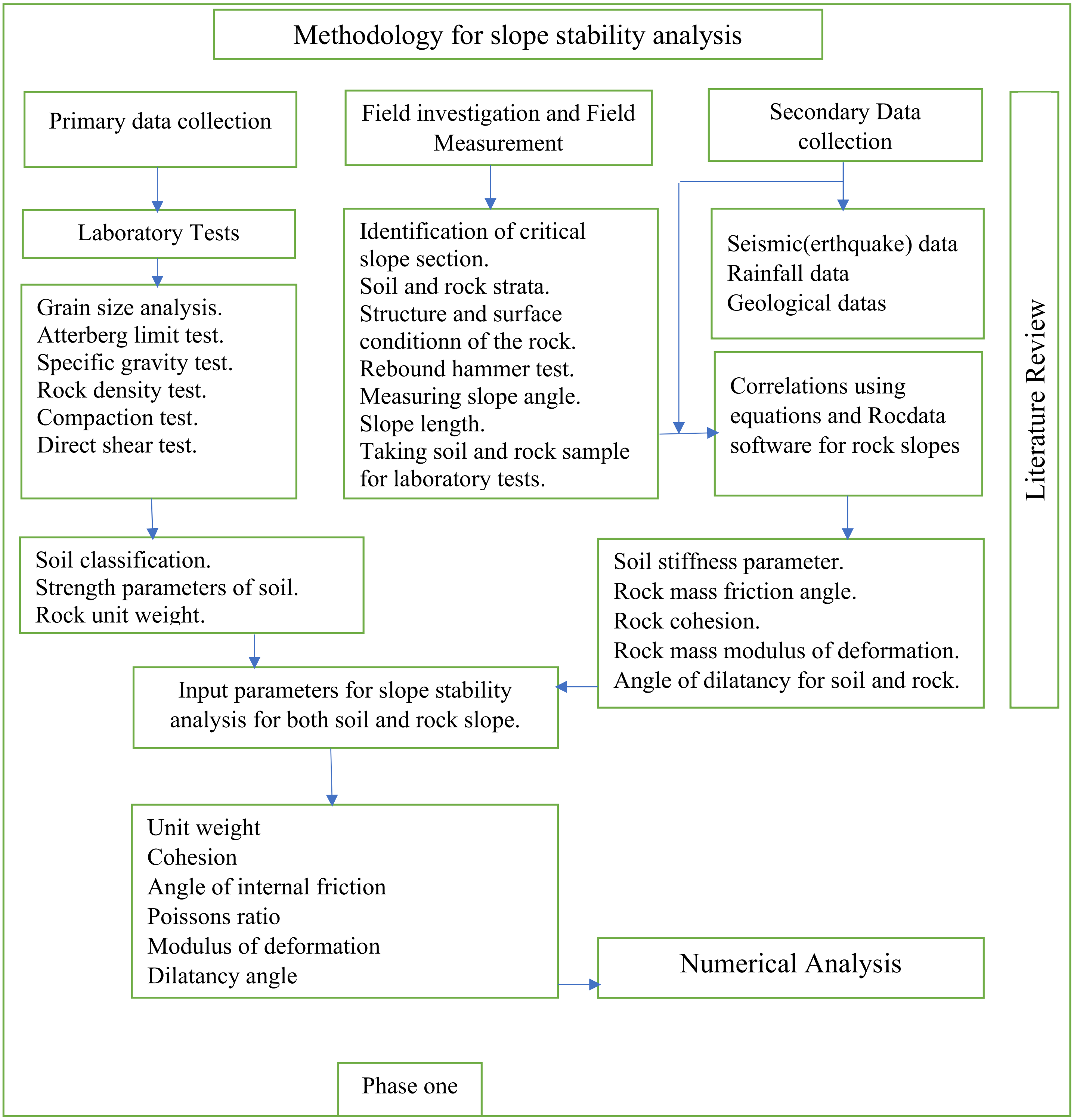


Figure 3. 4 General workflow of the study phase one

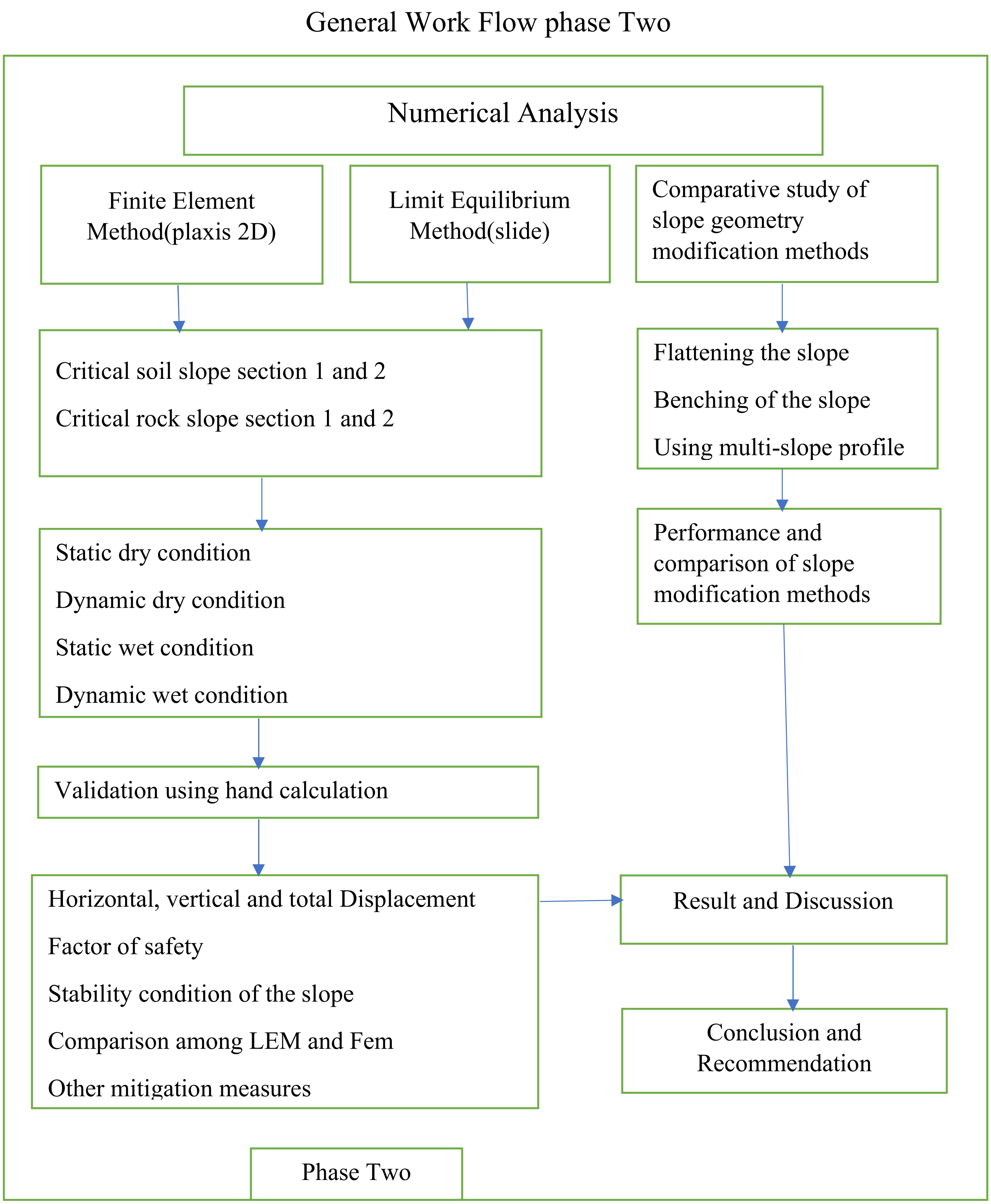


Figure 3. 5 General workflow of the study phase two

CHAPTER FOUR

FIELD INVESTIGATIONS AND LABORATORY TESTS

4.1 General

The first and foremost requirement for the analysis and design of slope is to guarantee their safety and reliability during their service life. To perform slope stability analysis both field investigation and laboratory tests must be carried out to generate field characteristics and shear strength parameters. Generally, data used for numerical modeling is obtained from field surveys, laboratory work, and correlation of field and laboratory data.

4.2 Field investigation

After the detailed literature review was made on types, causes, and mechanisms of slope failures a detailed field investigation was made to identify the critical slope section and to collect data for the stability analysis. During the field investigation, the inclination of slope face, slope angle, location of the critical sections, possible failure manifestations like debris flows, drainage condition, types of slope material, and weathering conditions were identified. surface mapping using the information on rock exposure and discontinuity characteristics gives more reliable data than mapping using information from core data (Wyllie and Mah, 2017). This is because it provides orientations of discontinuities that are not easily accessible from core data and accessibility of large-scale features compared to small-scale drill core. Hence, classification and characterization of rock mass in this study were made using the measured and observed structural, weathering, and discontinuity conditions.

During field investigation weathering conditions and discontinuity, characteristics were identified in the critical slope section. Using data collected from field investigation parameters such as textural classification, geological strength index (GSI), intact rock coefficient (mi), and disturbance factor (D) were determined through correlation and slope material conditions. The uniaxial compressive strength of rock is determined from the Schmidt rebound hammer test at the site during field investigation. The rebound number is taken from the Schmidt rebound hammer test and converted to UCS using correlation formula and correlation chart.

At the end of field investigation, soil samples are taken for laboratory investigation to determine the index and strength parameters of slope material depending on the material variation of the critical slope section. Four soil samples are taken from two different critical sections for the determination of index and strength parameters. Rock samples are taken from different four critical sections to determine the rock density in the laboratory. The sampling of soils was taken 1m horizontally from the face of the slope. The amount, as well as sample location, is decided based on the standards (ASTM, 2014 and EBCS, 2015).

4.2.1 Uniaxial compressive strength of the rock

The uniaxial compressive strength of the rock can be determined in the laboratory using a core sample or can be calculated using the Schmidt rebound hammer test on the field (Barton and Choubey, 1977)(Wyllie and Mah, 2017). For this research work, the Schmidt rebound hammer test is used to determine the uniaxial compressive strength of the rock. First, the surface of the rock is cleaned and then divided into ten equal parts of rectangles then after the Schmidt rebound hammer test is applied normal to the surface of the rock and the reading is taken. Three trials are done for each critical slope section each trail having ten rebound numbers. The average values of the rebound numbers are used for the correlation to determine the uniaxial compressive strength of the rock. The test was conducted based on ASTM D5873 standard.

Figure 4. 1 Schmidt hammer rebound rock strength test on field.

The test performed for 10 trials using 5cm equal spacing of the boxes and the direction of the the test hammer is normal to the surface of the rock surface. The recorded rebound number is converted to UCS using both equation 4.1 and the correlation chart shown in figure 4.2.

$$\log_{10}(UCS) = 0.00088\gamma R + 1.01 \tag{4.1}$$

Table 4. 1 Sample result of UCS of the rock from Schmidt rebound hammer test

Slope section	Trial	Rebound number (R)	Average Rebound No.	UCS from Equ. (Mpa)	UCS from curve (Mpa)
RCSS1	1	14,16,20,24,19,15,20,14,19,18	17.9	26.8	28
	2	20,22,26,25,27,23,25,29,24,21	24.2	37.5	39
	3	25,30,32,38,36,35,29,32,28,37	32.2	57.7	54
	Average			40.7	40.3

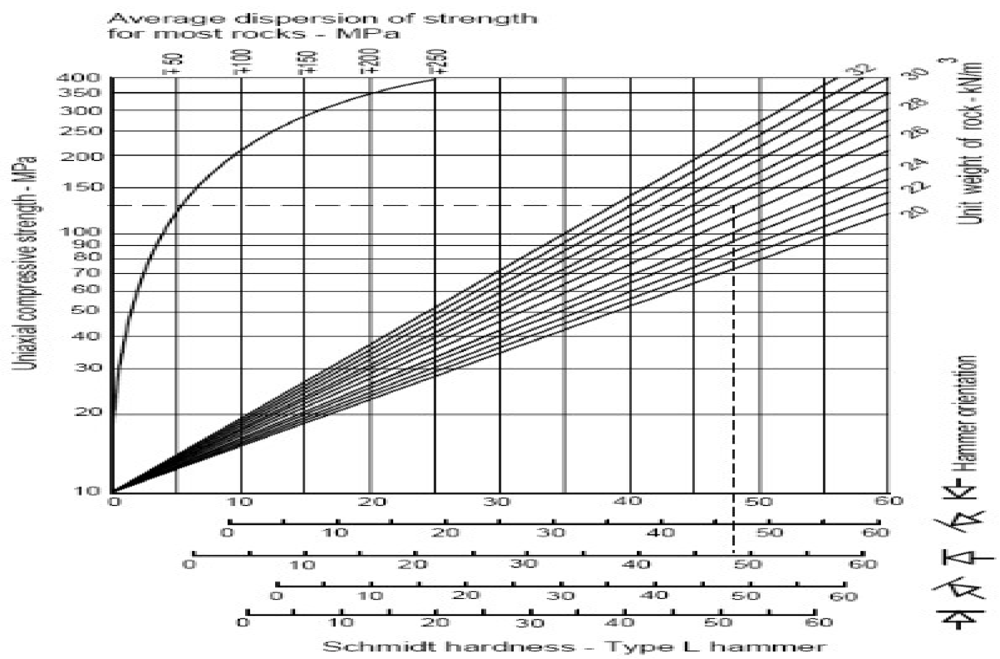


Figure 4.2 Correlation chart for Schmidt hammer relating rock density, compressive strength, and rebound number (Barton and Choubey, 1977).

4.3 Laboratory tests on soil samples

Shear strength parameters and unit weight of the soil which composes the slope soil mass is the most important input parameter to be determined to perform the stability analysis of the slope under consideration. In addition to that, atterberg limit test and grain size distribution test of the soils were performed to give a general classification of the soil.

4.3.1 Natural Moisture Content

Natural water content is the in-situ water content of soils conducted on afresh excavated soil samples without exposing it for air. The test was conducted on fresh soil samples collected from four different locations based on ASTM D 2216 Standard test procedure. Detailed results are summarized in appendix I.

Table 4. 2 Natural moisture content.

sample code	SCSS11	SCSS12	SCSS21	SCSS22
Moisture content(%)	18.4	20.3	17.5	16.6

4.3.2 Specific gravity of the soil

The specific gravity (G_S) of soil refers to the ratio of the solid particles unit weight to the unit weight of water. The test was determined by a pycnometer using ASTM D 854-00 standard. Results are summarized in appendix I.

Table 4. 3 Specific gravity of the soil.

Sample code	SCSS11	SCSS12	SCSS21	SCSS22
Specific gravity	2.58	2.65	2.63	2.65

4.3.3 Grain size distribution test

Grain size distribution is the method used to separate soil particles in different sizes based on the diameter of soil particles. Mechanical analysis is used for the coarse-sized soils(particle size > 75µm) by using the nest of sieve and hydrometer analysis is used for fine-grained soils(particle size < 75µm). In this research work, test on first sample R1 conducted using both wet and dry sieve and when the results are compared it is already the same value since the soil is sandy soil so, the remaining samples are carried out using dry mechanical sieve analysis on soil samples collected from the critical section to determine the grain size distribution as dry mechanical sieve analysis is recommended for sandy soil. The test was conducted based on ASTM D422-63 standard procedure. The results of the test are summarized as shown in figure 4.2 and others in appendix I.

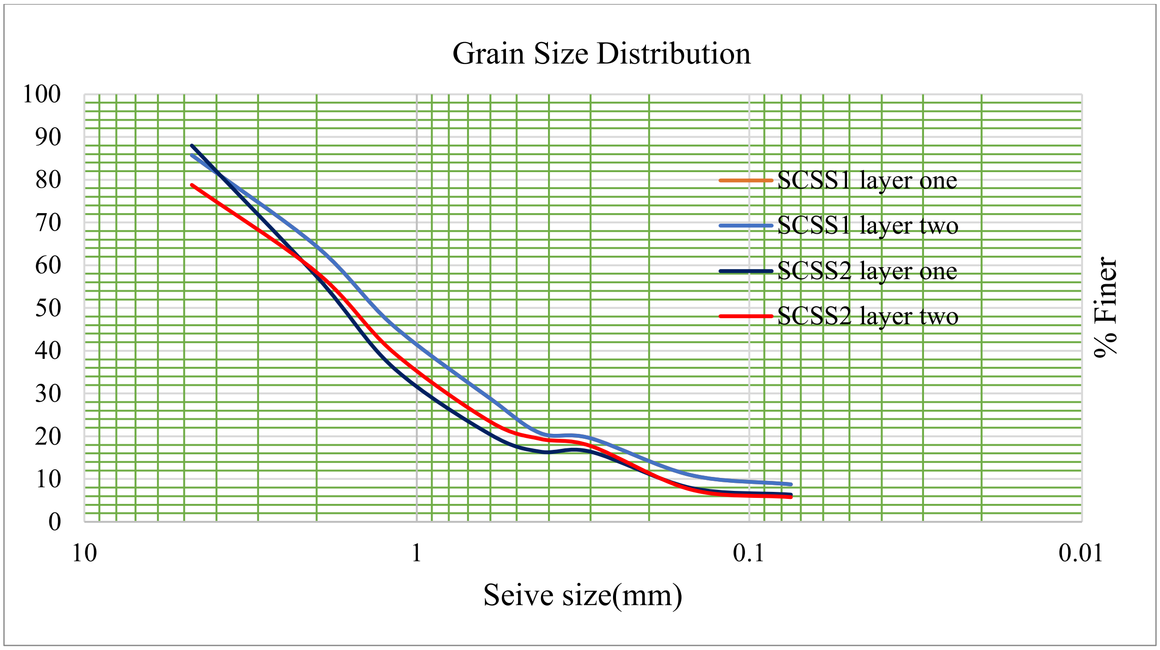


Figure 4. 2 Grain size distribution curve for soil critical slope sections.

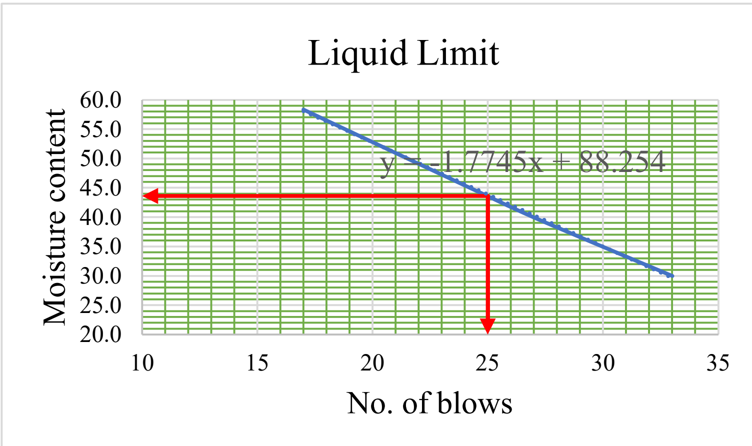
4.3.4 Atterberg limit test

The Atterberg limits and related indices have proved to be very useful for soil identification and classification. The test is used to determine the liquid and plastic limits of fine-grained soil. The liquid limit (LL) and plastic limit (PL) are the moisture content that defines the point of soil phase changes from liquid to plastic state and plastic to semi-solid state respectively. Liquid Limit is measured by spreading a portion of the soil sample in the brass cup of a liquid limit machine (Casagrande apparatus) and dividing it using a grooving tool.

The test standard used is ASTM D4318. Plastic limit is determined by repeatedly remolding a small mass of wet plastic soil and manually rolling it out into a 0.125in the thread. The Plastic Limit is the water content at which the thread crumbles before being completely rolled out. The standard test method used is ASTM D4318. The result is summarized in the appendix.

Table 4. 4 Summary for atterberg limit test

Slope section	Liquid limit	Plastic limit	Plasticity index
SCSS11	46.3	31.2	15.1
SCSS12	43.6	34.5	9.1
SCSS21	42.4	30.1	12.3
SCSS22	51.7	38.3	13.4



Liquid limit = 43.64

Plastic limit = 34.52

Plastic Index(PI) = LL - PL

= 43.64-34.52 = 9.12

Figure 4. 3 Liquid limit chart, plastic limit, and plasticity index

4.3.5 Compaction test

Compaction places soils in a dense state and hence decreases further settlement, increases shear strength and decreases permeability. Mechanical compaction is one of the most common and cost-effective means of stabilizing soils. During compaction process, air is expelled from the void spaces. Therefore, compaction increases the density of the soil. Optimum water content is the water content that produces the maximum density under specific pressure.

Compacting at water contents higher than the optimum water content will result in a relatively sparse soil structure (parallel particle orientations) which is weaker, more ductile, less previous, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. Compacting soils below the (dry) optimum moisture content usually results in a flocculent soil structure (random particle orientations) which has the opposite characteristics of the wet compacted soil with optimum water content at the same density.

Two types of compaction tests routinely performed are the standard proctor test and the modified proctor test. In this work, the test is conducted using a standard proctor test, on four soil samples. The test result was used to determine the maximum dry density and optimum moisture content of the soil used to remolded the soil in the shear strength test. MDD and OMC for slope sections are summarized in figure 4.5 and table 4.5 as shown below.

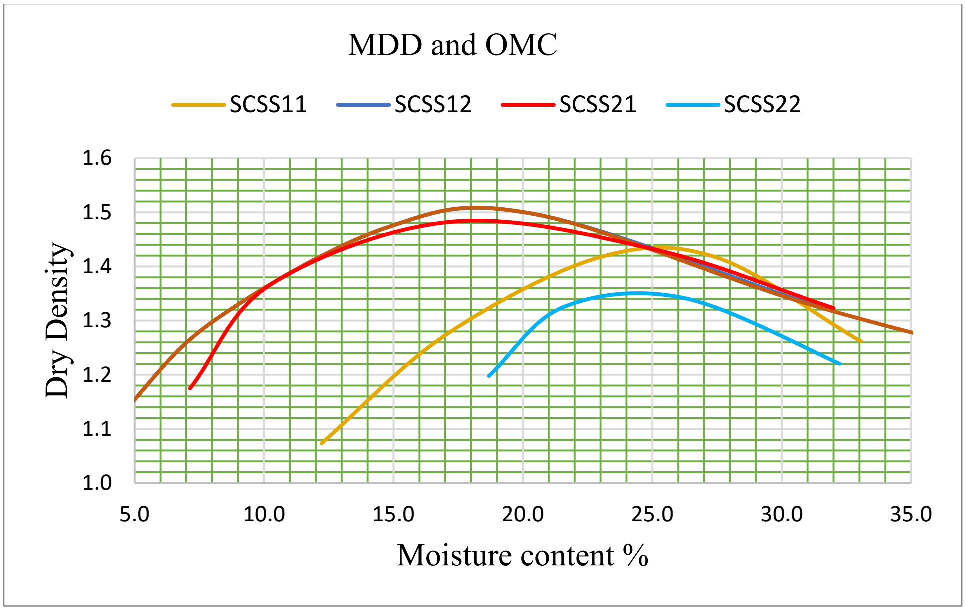


Figure 4.5 Compaction curve for soil critical slope sections.

Table 4. 5 Summarized values of MDD and OMC for all critical slope sections.

	SCSS11	SCSS12	SCSS21	SCSS22
MDD(g/cc)	1.467	1.51	1.485	1.35
OMC(%)	25	18	18	24

4.3.6 Direct shear test

The direct shear test is one of the shear strength tests used to determine the strength parameters of samples of undisturbed or remolded soil materials. To define the Mohr-Coulomb failure criteria shear strength parameter internal angle of friction (φ) and cohesion(c) were determined from the direct shear test. The sample test result is shown in figure 4.4 and the summarized value of the test result are given in table 4.5 the remaining is in the appendix.

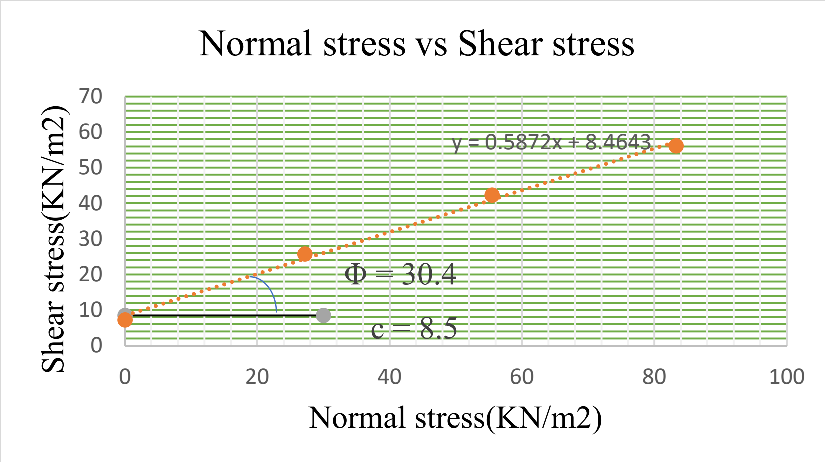


Figure 4. 4 Shear stress vs normal stress curve.

Table 4.6 Normal stress vs shear stress .

	SCSS11			SCSS12			SCSS21			SCSS23		
Normal force(N)	100	200	300	100	200	300	100	200	300	100	200	300
Normal stress(KN/m2)	27.2	55.5	83.3	27	56	83.3	27.2	55.5	83.3	27.2	55.5	83.3
Shear stress(KN/m2)	25.7	42.3	56.2	31	49	65.8	28	46	72	30.5	42	70.2

For this research work, the test was performed on four disturbed soil samples in a predefined failure surface using ASTM D 3080. For the test readings of shear load, shear displacement and normal displacement are recorded for each normal load applied. Then from the Mohr-Coulomb envelope the strength parameters c and φ were determined.

4.4 Laboratory tests on rock samples

4.4.1 Unit weight of the rock

Unit weight of the rock is one of the parameters used in the stability analysis of the slope under consideration so, its determination is necessary for a complete definition of material properties. In this research work, the unit weight of the rock is determined using the buoyancy technique in the laboratory. Two samples are taken from each critical slope section and a buoyancy technique is done for the samples from each CSS and finally, the average of two values was taken for the stability analysis.

Table 4. 7 Rock unit weight from buoyancy technique.

Section	Sampl e No.	Dry mass(kg)	Vol. Of water(m3)	Vol. Water+rock	Vol.o f rock	Density (Kg/m3)	Unit Weight (KN/m3)
SCSS13	1	0.103	0.5	0.849	0.049	2510.2	24.5
	2	0.118	0.5	0.848	0.048	2458.3	24.1
Averega							24.2
SSS23	1	0.115	0.8	0.84	0.04	2875.0	28.2
	2	0.08	0.8	0.838	0.038	2105.3	20.6
Averega							24.4
RCSS1	1	0.115	0.8	0.848	0.048	2395.8	23.5
	2	0.13	0.8	0.85	0.05	2600.0	25.5
Averega							24.5
RCSS2	1	0.099	0.8	0.838	0.038	2605.3	25.5
	2	0.13	0.8	0.85	0.05	2600.0	25.5
Average							25.5

4.5 Correlation for rock mass properties and soil stiffness parameters

4.5.1 Correlation for soil stiffness parameters

Modulus of elasticity and poisons ratio is the important soil stiffness parameters used for deformation analysis in numerical modeling of the slope section. Suitable values of these parameters were obtained from numerical correlations and Literature.

Corrected SPT N value to an average energy ratio of 60% is calculated using equation 4.2 as recommended by Wolff (1989). Modulus of elasticity of the soil is correlated with SPT N value as proposed by Begemann (1974) as cited in (Das, 2002). The drained poisons ratio of granular soils is obtained from equation 4.6 as suggested by Kulhawy and Mayne (1990)as cited in (Das, 2002).

$$\phi(deg) = 27.1 + 0.3N_{60} - 0.00054N_{60}^2 \qquad (4.2)$$

$$E = 1200(N+6)\frac{KN}{m^2} \quad for\ N < 15 \text{ and sandy gravel.} \qquad (4.3)$$

$$E = 4000 + 1200(N-6)\frac{KN}{m^2} \quad for\ N > 15\ and\ sandy\ gravel\ soil. \qquad (4.4)$$

$$\frac{E}{P_a} = 5N_{60} \quad \text{for sand with fines.} \qquad (4.5)$$

$$v_d = 0.1 + 0.3\left(\frac{\phi-25}{20}\right) \quad \text{for } \phi < 45^{\circ} \tag{4.6}$$

Values of stiffness parameters of the soil slope sections are summarized in the table below.

Table 4. 8 Summary of stiffness parameters of soil

Slope section	Poissons ratio(v)	Modulus of deformation, E (Mpa)
SCSS11	0.181	20400
SCSS12	0.208	8500
SCSS21	0.301	44800
SCSS22	0.253	26000

4.5.2 Determination of rock mass strength

Slope stability analysis involves an examination of the shear strength of the rock mass on the sliding surface which is expressed in terms of the Mohr-Coulomb failure criterion (c and ϕ). The shear strength parameters of rock mass are determined from rocdata software using the generalized Hoeke-Brown strength criterion in terms of the major and minor principal stresses. The Hoek-Brown failure criterion is an empirical formulation for estimating the confinement strength relationship of a rock mass (Wyllie and Mah, 2017). Uniaxial compressive strength (UCS), the geological strength index (GSI), intact rock coefficient (mi), disturbance factor(D), slope height, and unit weight of the rock are used as input for the software as discussed below.

1. Source of input data used in Rocdata software:

The uniaxial compressive strength of intact rock (UCS) was determined from the Schmidt rebound hammer test carried out on the field. Geological strength index (GSI) was obtained by considering weathering conditions of rock surface and rock mass structure as proposed by (Wyllie and Mah, 2017). The intact rock coefficient value (mi) was taken from the tabulated data for different types of rock. The intact rock cofficient for basaltic rock is 25 ± 5 from the tabulated data.The tabulated data is already built in the rocdata software. Disturbance factor (D) was obtained based on the degree of disturbance of rock mass subjected to mechanical excavation (Wyllie and Mah, 2017).

Table 4. 9 Table summary of input data used in Rocdata software.

Sample code	Rock types	Structure	Surface condition	UCS (Mpa)	D	mi	GSI
SCSS13	Highly weathered basalt	blocky disturbed	poor	38	0.7	25	30
SCSS23	Highly weathered basalt	very blocky	Fair	57.8	0.7	25	45
RCSS1	Highly weathered and fructured basalt	Blocky	Good	40.7	1	25	45
RCSS2	Highly weathered and fructured basalt	Blocky	fair	36.3	1	25	40

2. Deformation parameters for rock mass

In addition to the estimate of the strength of intact rock and rock masses, the analysis of the behavior of a slope, also requires an estimate of the deformation modulus of the rock mass in which the slope is excavated.

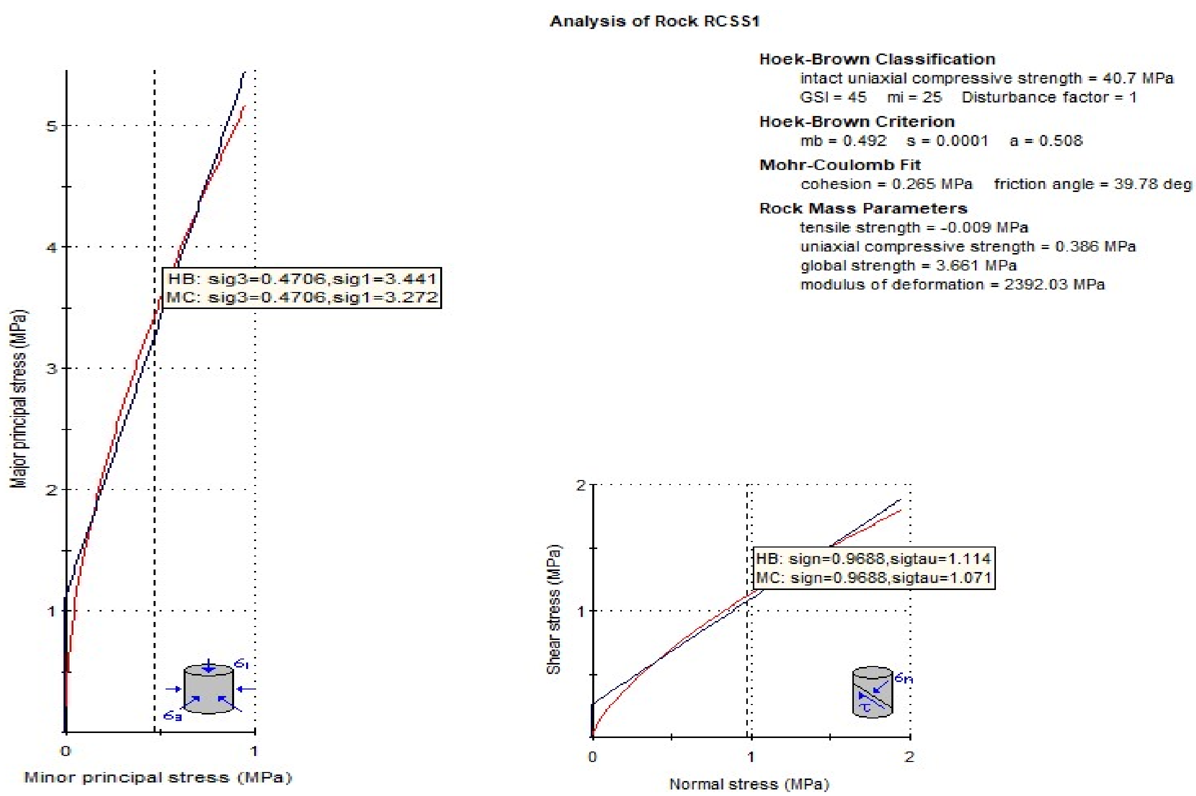


Figure 4. 5 Equivalent rock mass properties as calculated from MC and HB criterion

Hoek–Brown failure criterion can allow determination of rock mass modulus of deformation used for numerical analysis of slope stability using equation 4.7 (Wyllie and Mah, 2017). The Poisson's ratio of the rock mass is one of the most important rock parameters used for calculating the deformation of rock slopes. The value of poisons ratio for rock mass is estimated by correlating it with internal friction angle as shown in Equation 4.8 (Vásárhelyi, 2009).

$$E = (1 - \frac{D}{2})^2 \sqrt{\frac{UCS}{100}}\, 10^{(\frac{GSI-10}{40})} \tag{4.7}$$

Where UCS < 100Mpa and E is in Gpa

$$v = -0.002GSI - 0.003mi + 0.457 \tag{4.8}$$

$$\psi(dilatancy\ angle) = \phi(degree) - 5 \tag{4.9}$$

Table 4. 10 Summary of rock mass parameters.

sample code	UCS(Mpa)	Disturbance factor	mi	GSI	Modulus of deformation E(Mpa)	Poissons ratio(v)
RCSS1	40.7	1	25	45	2392.03	0.31
RCSS2	36.3	1	25	40	1694.04	0.302
SCSS13	38	0.7	25	30	1267.08	0.322
SCSS23	57.8	0.7	25	45	3705.76	0.292

CHAPTER FIVE

SLOPE MODELING AND NUMERICAL VALIDATION

With a numerical model, geotechnical engineers can simulate the real and actual situation of the geotechnical problems such as construction phases, different loading conditions, excavation conditions, and boundary conditions. The finite element method is a fully advanced and powerful numerical modeling tool widely used in geotechnical engineering that is why the method was selected for this research work to predict the stability of the slope and to investigate the comparative study of the slope profiles. Plaxis 2D was used to determine the deformation and factor of safety under static and pseudo-static loading conditions and the limit equilibrium method also included in this evaluation using LE-based slide software.

5.1 Modeling Using Plaxis 2D

Finite element modeling is the first and the most crucial step in any analysis. According to (Plaxis, 2013) the continuum must be discretized into small pieces or elements to describe the behavior or actions of individual pieces and reconnecting them to represent the behavior of the continuum as a whole. In the FEM, a continuum is divided into several(volume) elements. Each element consists of several nodes. Each node has several degrees of freedom that correspond to the discrete value of the unknowns in the boundary value problem to be solved. The first step in FEM is defining the problem domain which includes defining the geometrical model, material model, and also applying boundary conditions. Triangular area elements were used to discretize soils in plaxis 2d to form a mesh that connects with nodes to transfer forces and displacements between them. The element stiffness functions are formed after the problem domain is defined and discretized into finite elements.

$$[K]\{d\} = \{f\} \tag{5.1}$$

$$K = \int B^T D^e B dV \tag{5.2}$$

Where; K is the element stiffness matrix, d is a vector of an unknown degree of freedom or nodal displacement, and f is a resultant vector of element nodal forces. The elastic element stiffness matrix is also expressed as a function of strain interpolation matrix B and an elastic material matrix D^e.

The element stiffness matrix is gathered and assembled to form a global continuum equation containing the material and geometric data. The unknown nodal displacements are solved from the global equations by applying known boundary conditions. The values between nodes are obtained from interpolated shape functions. The remaining parameter forces, stresses, and strains are obtained by post-processing using determined nodal displacements in the element and global equations. In numerical modeling proper selection of parameters such as element type, mesh size, boundary conditions, constitutive models increase the accuracy of results. Hence, emphasis was given to the selection of these parameters.

The procedures required to model and analyze the slope using FEM (plaxis 2D) and LEM (slide) are included in this section. Finally, the model was validated using previous case studies obtained from the literature. The workflow is shown in figure 5.1.

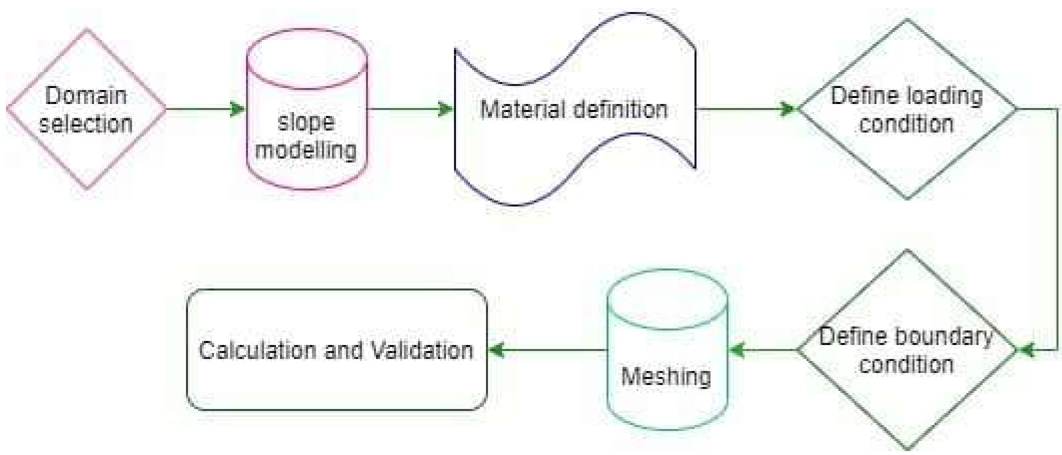


Figure 5. 1 Steps to carry out finite element method simulation.

1. Domain selection

The selection of the domain for the slope stability analysis was done based on basic and mandatory criteria such as the availability of soil and rock data. The other significant selection criteria are the risks for road and hazard values of critical slope locations in the case study. The selection of the domain will influence the accuracy of the result. To minimize this effect the selection of domain extent was made by performing sensitivity analysis for vertical and lateral boundaries under constant loading conditions.

2. Geometric Modelling

The first and the most critical step of any analysis. The part is modeled omitting complicated geometric features. This will provide the insight to remove insignificant features from the geometry eventually saving some computational time as well as unwanted complexity. During FE geometric modeling the selection of model extent influences the accuracy of the result. In this research work, four critical slope sections of the slope were used for plaxis 2d modeling.

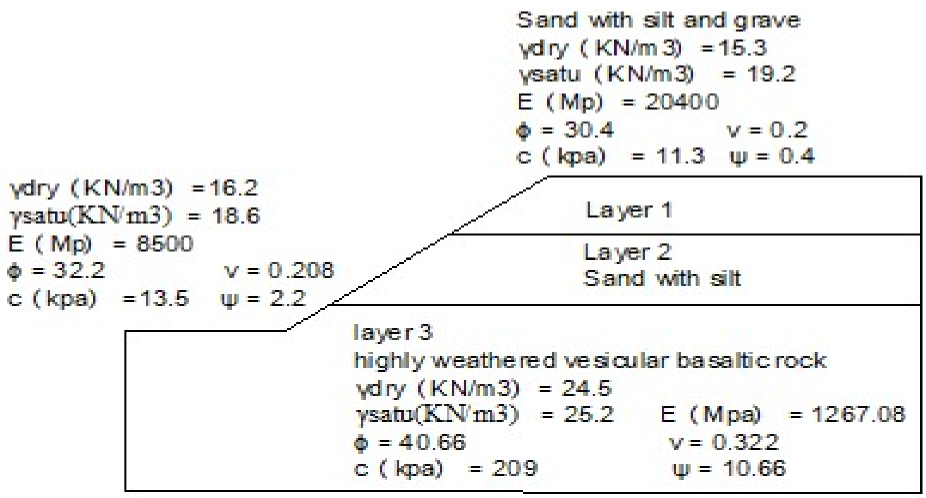


Figure 5. 2 Geometric modeling of soil critical slope section one.

The soil critical slope sections one and two are modeled for slope angles of 35 and 40 respectively. The slope length for both slope sections is 50m. The extent of the horizontal boundary and vertical boundary of the slope is taken as recommended from previous related works and using recommended formulas. The slope model for slope section one is given in figure 5.2 and the others are discussed in the material modeling section.

3. Material definition

The strength parameters and stiffness parameters for soil and rock are determined from a field test, laboratory tests as well as correlations as discussed in chapter three. For this research work, Mohr-coulomb elastic perfectly plastic model is used and rock layers were modeled using Hoek's -Brown equivalent Mohr-Coulomb model. Parameters used as input in plaxis 2D and slide software are arranged as follows.

Table 5.1 Soil and rock parameters used as input in numerical analysis.

Slope section	Layer	Classification	γsatu (KN/m3)	γdry (KN/m3)	c(kpa)	ϕ°	V	E(Mpa)	ψ
Soil slope section 1	layer 1	Sand with silt and gravel(SW-SM)	19.2	15.3	11.3	30.4	0.181	20400	0.4
	layer 2	Sand with silt(SW-SM)	18.6	16.2	13.5	32.2	0.208	8500	2.2
	layer 3	Highly weathered vesicular basalt	25.2	24.5	209	40.6	0.322	1267.08	10.7
Soil slope section 2	layer 1	Well graded sand(SW)	18.1	15.8	5.3	38.4	0.301	44800	8.4
	layer 2	Sand with Gravel(SW)	17.2	14.8	8.5	35.2	0.253	26000	5.2
	layer 3	Highly weathered vesicular basalt	26	25.5	377	50.8	0.487	3705.76	20.8
Rock slope section 1	layer 1	Highly weathered and fructured vesicular basaltic rock	25.8	24.2	265	39.78	0.31	2392.00	9.78
Rock slope section 2	layer 1	Highly weathered and fructured vesicular basaltic rock	26	24.4	206	36.36	0.292	3706.00	6.36

4. Loading condition

The road cut slope is analyzed under both static and pseudo-static loading conditions. Pseudo static loading condition is used to analyze the activity of the seismic earthquake on the stability of the slope as the use of dynamic analysis is difficult to use here. The horizontal earthquake coefficient taken from EBCS 1995 is used for the pseudo-static analysis of the slope sections.

5. Boundary condition

Slope stability problems in geomechanics are boundary value problems and boundary conditions play an important role in the development of internal stresses in the medium and hence influence the calculated factor of safety (Chugh, 2003). For the solution based on continuum mechanics to be meaningful, the slope model must be extended to the location where slope failure can occur.The commonly used boundary conditions are (Chugh, 2003):

(a) There is no out-of-plane displacement in the x direction at the end of the model (u = 0 in the yz-plane at the end of the x direction). These limits are far enough away from areas where slope failure can occur.

(b) No displacement at the base of the slope model (u = v = w = 0 for the xy plane at z = 0). This boundary is placed far enough from the region where slope failure is likely to occur.

(c) Plane strain in the y-direction. This implies the out-of-plane displacement, v and shear stresses, τyx and τyz on the y-faces are zero. However, the normal stress, σyy, on the y-faces is not zero.

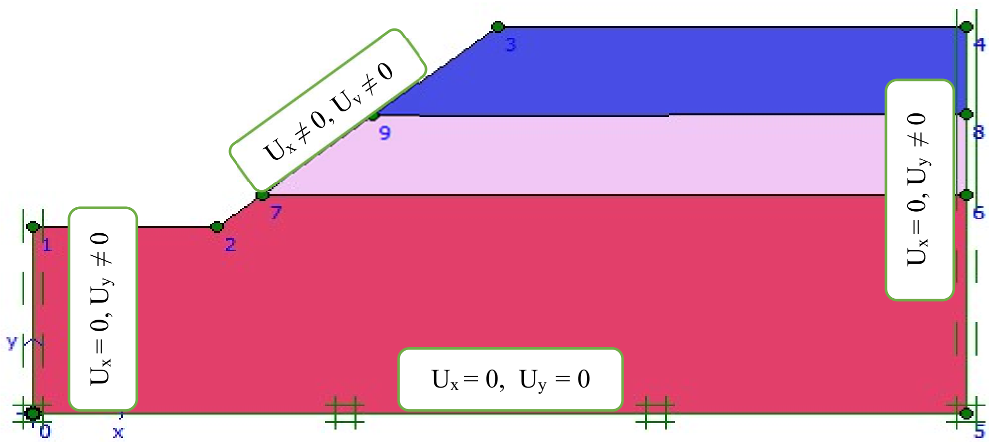


Figure 5.3 Boundary conditions for Plaxis 2d modeling.

6. Discretization of the slope model

Discretization is subdividing a complex structure into an ensemble of smaller components called finite elements. In finite element method numerical analysis, the continuum is discretized into smaller finite elements, the discretized finite elements are treated separately and finally reconnected to represent the continuum. Infinite element-based analysis, the selection of the proper shape of the element, element number, and element size are crucial to minimizing their effect in the calculation of the numerical results. There are different shapes of the element, element size, and many numbers of element numbers that can be used during the meshing of the model in the finite element method. For this research work, sensitivity analysis is carried out to select the size of the mesh. Sensitivity analysis is done by evaluating the effect of different mesh sizes on total with constant loading conditions. Accordingly, the effect of reducing element, size increases horizontal displacement becomes negligible beyond element number 250 (medium size).

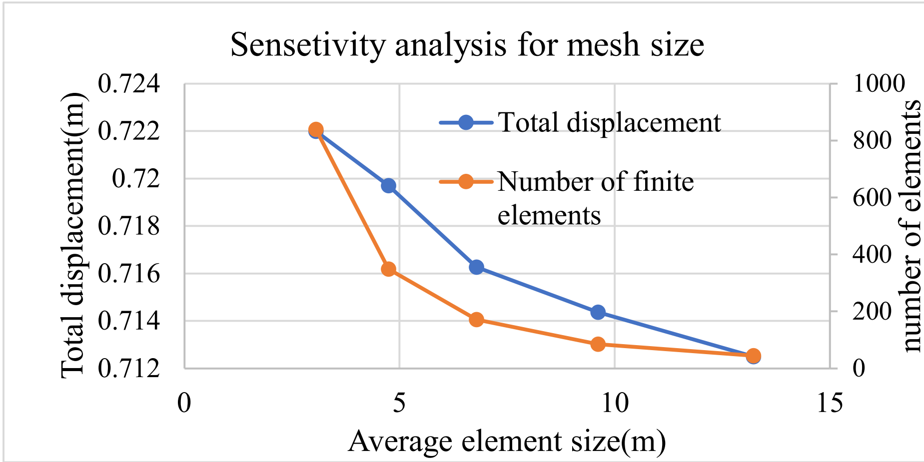


Figure 5.4 Sensitivity analysis to select mesh size.

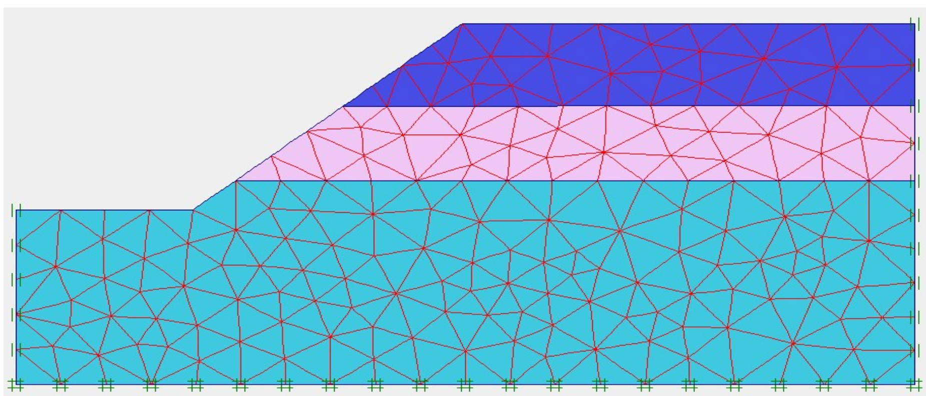


Figure 5. 5 Discretized slope model.

Table 5. 2 Mesh information and sources of input data for slope modelling.

Mesh Size	Type	Number of element
Medium	15 node element type	210

Input Data	Source
γ	Calculating density from direct shear test sample then unit weight is calculated from density.
ϕ	Obtained from the Plot of shear stress Vs normal stress from direct shear test.
c	Plotting shear stress Vs normal stress from direct shear test.
V, E, ψ	Calculated using correlation for both soil and rock. Formulas are discussed in chapter 4.

5.2 Calculation Using Plaxis 2D

Plaxis allows calculation for deformation and factor of safety of the modeled slope. First, the general setting of the software is adjusted on the plain strain model and 15 node element type. Then after the units of parameters going to be used are adjusted and the dimension of the geometry is corrected as required for the road cut slope dimension. The slope model is drawn, the material property is set for the modeled slope, applying standard fixities, generating the mesh, generating pore water pressure and initial stresses, then the next step is the calculation part. The groundwater table is applied to the slope section and water pressure is generated using a phreatic level. The last step is a calculation of deformation and factor of safety.

5.2.1 Deformation analysis

Deformation analysis is a method where finite element modeling uses a mesh to model a deformable system, as the deformation at each node of the mesh is calculated in response to applied stress. The deformation analysis was carried out with plastic calculation type and staged construction loading input. Two soil and rock critical slope sections were analyzed in this method using static and dynamic loading conditions. In the static condition for both dry and wet states, the evaluation was made for its material weight using $\sum Mweight$ as a multiplier. Whereas the pseudo-static condition was carried by introducing the earthquake forces in terms of horizontal acceleration coefficient (α=0.06) which uses multiplier (ΣMaccel) in the calculation routine.

5.2.2 Calculation for factor of safety

The safety factor in plaxis is computed using phi/c reduction method. Plaxis reduces the soil parameters(friction angle and cohesion) until the soil collapses. After running the model, the FS (Sum-Msf in plaxis) is found when the defined calculation phase is highlighted. The global safety factor is equal to the total multipliers $\sum Msf$ at the point of failure which is expressed as the ratio of initial and reduced strength parameters It is possible to get the value of FS using the curve of Sum-Msf in the function of the displacement.

$$\sum Msf = \frac{\tan(\phi'_{input})}{\tan(\phi'_{reduced})} = \frac{c'_{input}}{c'_{reduced}} \tag{5.3}$$

Sample deformation and factor of safety calculation from plaxis for both static and pseudo-static loading conditions under both dry and wet slope conditions. Staged construction loading input and the total multiplier is used for calculation of deformation for static and pseudo-static condition respectively. Phi/c reduction method is used for the calculation of factors of safety for both loading conditions. Sample calculation for critical soil slope section 1 is given in figure 5.6.

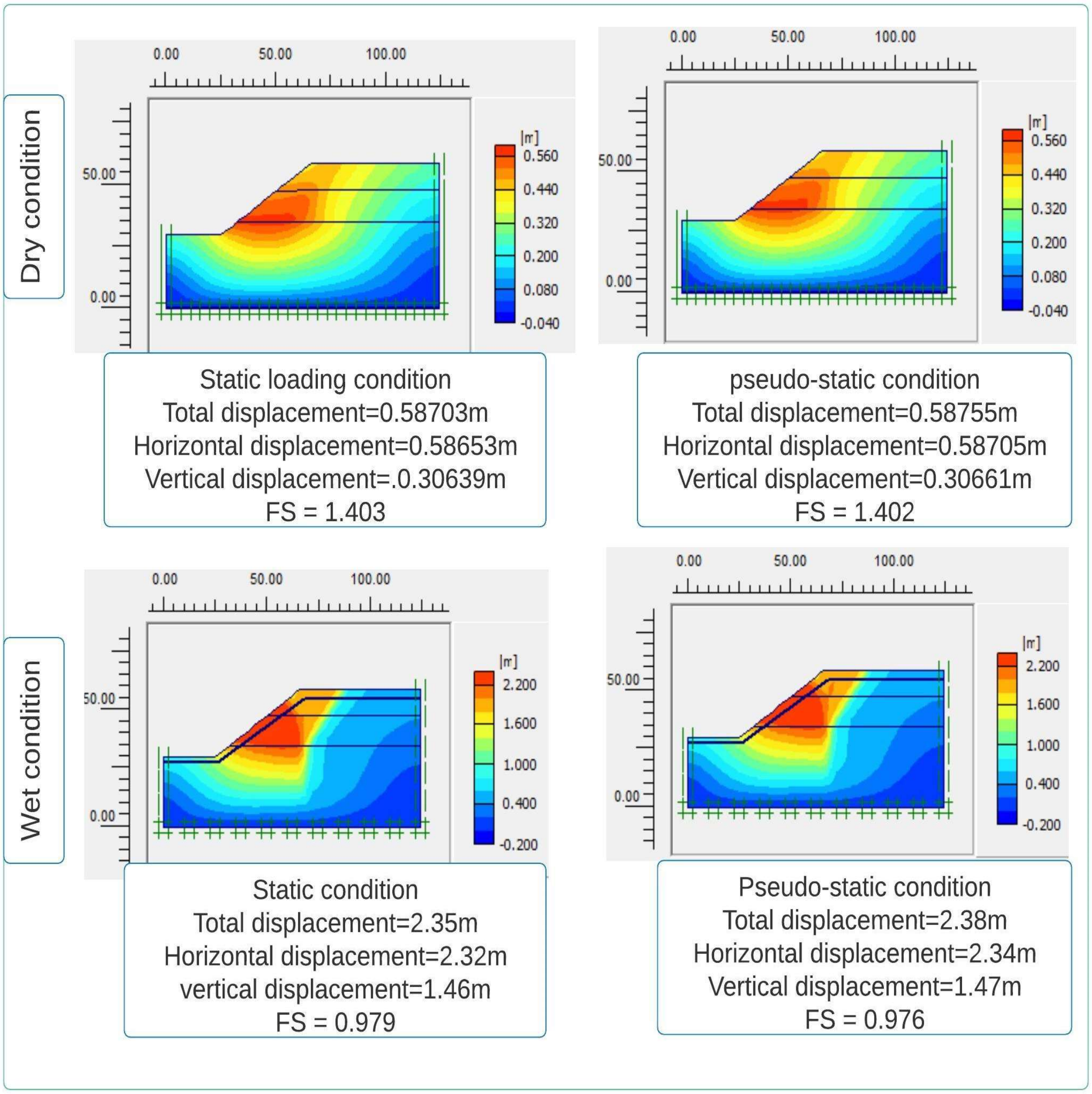


Figure 5.6 Displacements and factor of safety for critical soil slope section one.

5.3 Slope modeling using slide

The slide is a 2D slope stability program used to evaluate safety factor of circular or non-circular failure surfaces in soil and rock slopes. In this research work, both soil and rock critical slope sections are analyzed under static and pseudo-static loading conditions during a dry and wet state of the slope sections. Bishop, Janbu, Spencer, and GLE/Monstegern methods are used for the analysis. Mohr-Coulomb strength criterion was used to define the material properties of the slope. The criterion uses unit weight, cohesion, and angle of internal friction as the input parameter. Input parametes and slope model for soil critical slope section one is given in table 5.3 and figure 5.7 respectively. Iput parameters for all critical slope sections are given in material modelling section in section 5.1.

Table 5. 3 Input parameters for critical soil slope section one

Slope section	Layer	Classification	γsatu (KN/m3)	γdry (KN/m3)	c(kpa)	ϕ˚	V	E(Mpa)	ψ
Soil slope section 1	layer 1	Sand with silt and gravel(SW-SM)	19.2	15.3	11.3	30.4	0.181	20400	0.4
	layer 2	Sand with silt(SW-SM)	18.6	16.2	13.5	32.2	0.208	8500	2.2
	layer 3	Highly weathered vesicular basalt	25.2	24.5	209	40.6	0.322	1267.08	10.7

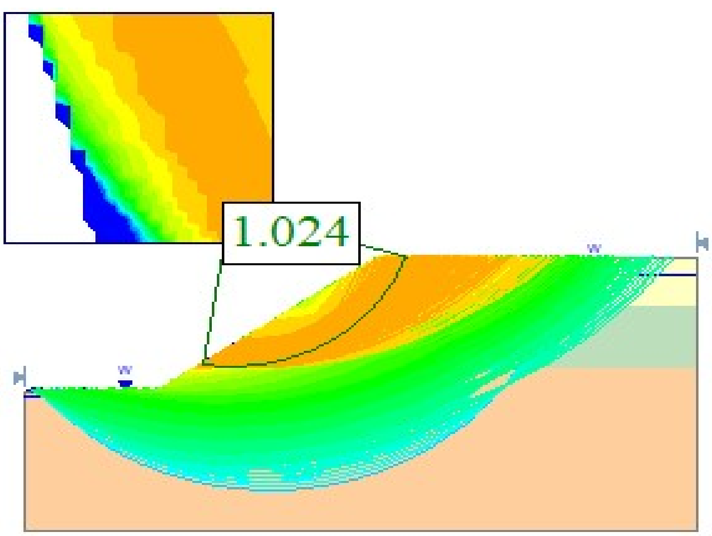


Figure 5.7 Determination of FS using slide software (SCSS1).

5.4 Comparative study on the performance of different slope profiles

The geometry of the slope is one of the most critical factors controlling the stability of the slope. Hence, here investigation based on performance is carried out for the unstable slope section to select appropriate slope profiles considering stability performance. Unstable slope section is modeled for the following slope geometry and finally, their performance is evaluated.

1. Single slope profile and the effect of different slope angle
2. Benching slope with different bench width and bench number.
3. Different combinations of multi-slope profiles.
4. Effect of using soil nail on slope stability

The performance of the single-slope profile was evaluated by decreasing the slope angle from the actual site condition(i.e. 40 degrees) to 35º, 30º, 25º. The respective values of the factor of safety of each slope angle are described in chapter six. The performance of benching the slope is discussed using the failing slope section of critical slope section two. The use of 2m, 3m, 4m, and 5m bench width by keeping the slope angle constant and slope height of 10m is used for the evaluation by using two benches.

In a multi-slope profile, the lower slope angle is kept constant as the actual condition found on the site. The lower layer is composed of rock material starting from the toe of the slope to 5m along the face of the rock. As the lower layer is composed of soil and rock material the layer is assumed to be a stiff layer and the angle of the layer is kept constant on the angle measured from the site which is 40 degrees. The angle of the upper layer is reduced until the slope comes to a state of stability.

Installing soil nails along the slope face increases the stability of the slope by increasing the resisting force against the failure driving force of the soil mass in the failure zone. The important concept in soil nailing is the development of tensile force in the soil mass and renders the soil mass stable. (Quansah *et al.*, 2018) recommended that nail spacing in both directions generally ranges from 1.2m to 2m and soil nail inclination ranges from 10 to 20 degrees from the horizontal, and most commonly at 15 degrees. Soil nail length required to be 0.7H, where H is wall height and nail length less than 0.5H will not likely satisfy sliding stability requirements.

The factor of safety of the soil slope increases with increasing of nail length to wall height ratio L/H, the factor of safety of the slope decreases with increasing nail spacing, but the inclination of the nail has a small effect on stability (Prashant and Mukherjee, 2010). The nail inclination increases the factor of safety until 15 degrees because inclined nails make it more difficult for the failure surface to avoid intersecting rows of nails. For this study, nail spacing of 2m, and nail inclination of 15 degrees were used. The length of the nail starts from 0.7H and is increased to check the effect of the length of the nail. The study of the soil nail effect in this research work depends on the effect of the nail length on the stability of the slope. The bond strength of the soil nail is 100-180kpa for sand/gravel soil and the corresponding construction method is rotary drilled. The average value of 100kpa and 180kpa were used for the analysis which is 140kpa. The tensile capacity of the nail is calculated using equation 5.4.

$$T = \Phi_k \Phi_n \Phi_t f_p A_p \ (KN) \tag{5.4}$$

Where

Φ_k = importance category reduction factor =0.8 (Table B1: AS4678).

Φ_n = structure classification category factor (0.9-1.1) 1 is used (Table 5.2: AS4678).

Φ_t = Material reduction factor = 0.9 (Table B2).

f_p = Tensile strength of nail(kN/m2)

A_p = Cross-sectional area of the nail.

AS4678 is the Australian standard for soil retaining structures.

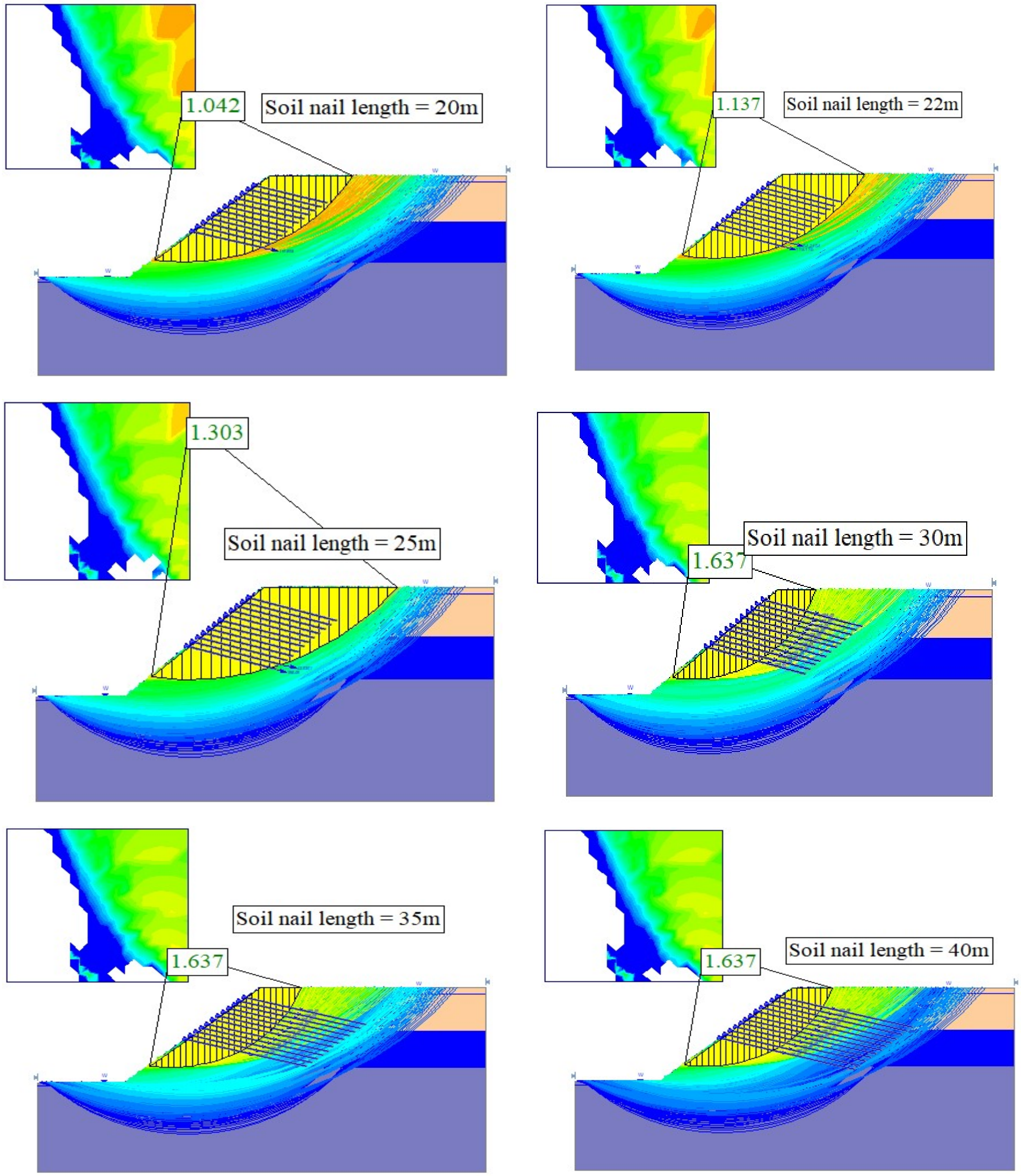


Figure 5. 8 Slope modelling for different soil nail length and corresponding factor of safety.

5.5 Numerical validation

1. Validating using hand calculation

A sample hand calculation is carried out using fellonius method to verify whether the result calculated from slide software is correct or not. Critical soil slope section one under static dry conditions is used for validation using sample calculation. There are 25 slices with in the slip surface and each slice having different parametrs as shown in table 5.3. The detail of each slice used in the hand calculation are taken from slide software. A result from the Janbu method is used for sample calculation.

Table 5.4 Detail information for each slice to calculate FS using the Janbu method.

slice no.	W	α_n	α_n in radian	c′	ϕ	ϕ in radian	ln	$\cos\alpha_n$	$\sin\alpha_n$	$w\cos\alpha_n$	$w\sin\alpha_n$	c′*ln	$w\cos\alpha_n$* tanϕ′
1	32.78	62.48	1.09	11.3	30.4	0.53	3.23	0.46	0.89	15.15	29.07	36.50	8.89
2	90.4	58.3	1.02	11.3	30.4	0.53	2.84	0.53	0.85	47.50	76.91	32.09	27.87
3	121.2	54.58	0.95	11.3	30.4	0.53	2.58	0.58	0.81	70.22	98.74	29.15	41.20
4	142.2	51.17	0.89	11.3	30.4	0.53	2.38	0.63	0.78	89.18	110.80	26.89	52.32
5	158.3	47.99	0.84	11.3	30.4	0.53	2.23	0.67	0.74	105.91	117.58	25.20	62.14
6	163	45.08	0.79	13.5	32.2	0.56	2.02	0.71	0.71	115.12	115.44	27.27	72.49
7	172.6	42.36	0.74	13.5	32.2	0.56	1.93	0.74	0.67	127.50	116.26	26.06	80.29
8	179.3	39.76	0.69	13.5	32.2	0.56	1.85	0.77	0.64	137.80	114.65	24.98	86.78
9	183.5	37.26	0.65	13.5	32.2	0.56	1.79	0.80	0.61	146.03	111.08	24.17	91.96
10	185.5	34.84	0.61	13.5	32.2	0.56	1.74	0.82	0.57	152.22	105.96	23.49	95.86
11	185.4	32.48	0.57	13.5	32.2	0.56	1.69	0.84	0.54	156.41	99.57	22.82	98.50
12	183.5	30.19	0.53	13.5	32.2	0.56	1.65	0.86	0.50	158.58	92.26	22.28	99.87
13	179.3	27.95	0.49	13.5	32.2	0.56	1.61	0.88	0.47	158.42	84.06	21.74	99.76
14	172.8	25.75	0.45	13.5	32.2	0.56	1.58	0.90	0.43	155.60	75.05	21.33	97.98
15	164.6	23.6	0.41	13.5	32.2	0.56	1.56	0.92	0.40	150.82	65.89	21.06	94.98
16	155	21.47	0.37	13.5	32.2	0.56	1.53	0.93	0.37	144.23	56.72	20.66	90.82
17	144	19.38	0.34	13.5	32.2	0.56	1.51	0.94	0.33	135.80	47.77	20.39	85.52
18	131.6	17.32	0.30	13.5	32.2	0.56	1.49	0.95	0.30	125.63	39.18	20.12	79.12
19	118	15.28	0.27	13.5	32.2	0.56	1.48	0.96	0.26	113.78	31.08	19.98	71.65
20	103.1	13.26	0.23	13.5	32.2	0.56	1.46	0.97	0.23	100.30	23.64	19.71	63.16
21	86.92	11.25	0.20	13.5	32.2	0.56	1.45	0.98	0.20	85.25	16.96	19.58	53.68
22	69.6	9.26	0.16	13.5	32.2	0.56	1.44	0.99	0.16	68.69	11.20	19.44	43.26
23	51.11	7.28	0.13	13.5	32.2	0.56	1.44	0.99	0.13	50.70	6.48	19.44	31.93
24	31.46	5.31	0.09	13.5	32.2	0.56	1.43	1.00	0.09	31.32	2.91	19.31	19.73
25	10.68	3.35	0.06	13.5	32.2	0.56	1.43	1.00	0.06	10.66	0.62	19.31	6.71
										Summation	1649.89	582.92	1656.47

$$FS = \sum_{p=1}^{p=n} \frac{(c' * ln + \mathrm{W}\cos\alpha\mathrm{n}\,\tan\phi')}{\sum_{p=1}^{p=n} \mathrm{W}\sin\,\alpha\mathrm{n}} = \frac{582.92 + 1656.47}{1649.89}$$

$$= 1.35$$

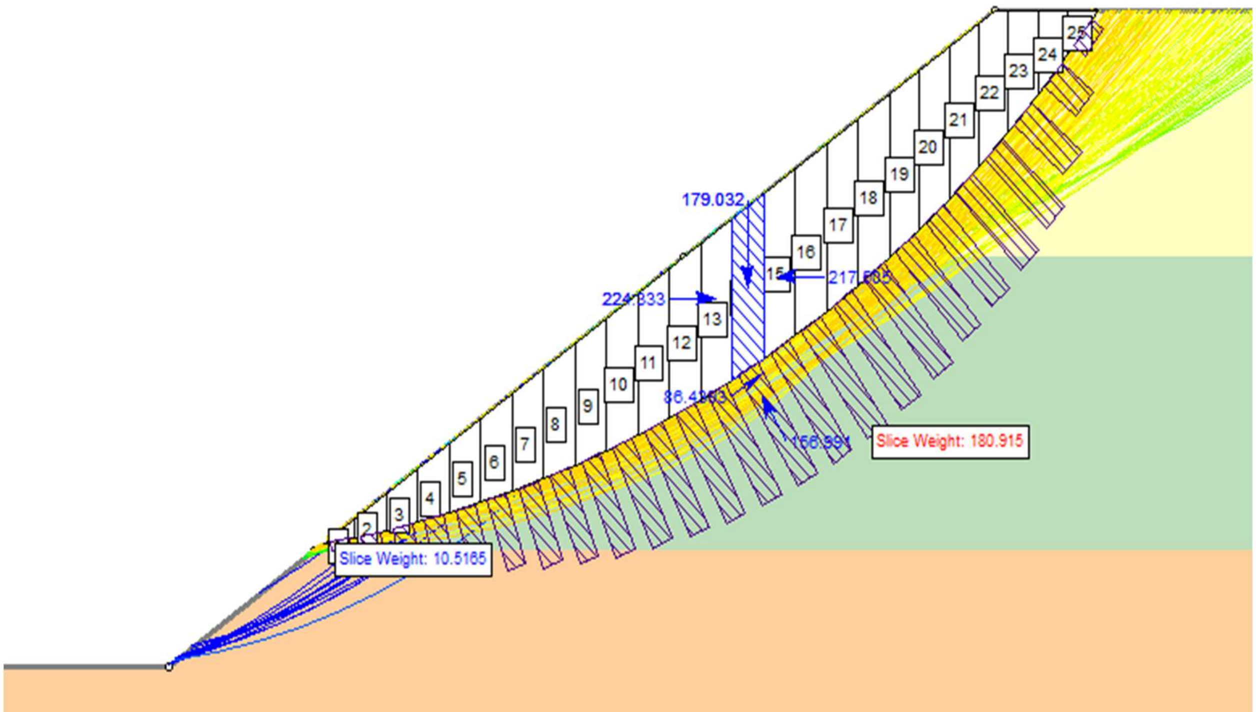


Figure 5.9 Model from slide software on which hand calculation is carried out.

The result from hand calculation using fellanious method and the result of FS from slide using the Janbu method is nearly the same which is 1.35 and 1.34 respectively. Since the result from hand calculation and using slide shows a good agreement it is reasonable to accept the results.

CHAPTER SIX

RESULT AND DISCUSSION

6.1 General

The stability condition and deformation were analyzed for different slope models with both static and dynamic loading conditions for dry and wet soil slope states. The comparative study of the slope geometry modification techniques was also discussed in this chapter. In this section, a discussion was made on the results of stability analysis using both finite element method and limit equilibrium method, as well as the performance of different slope modification techniques, are evaluated.

6.2 Result from deformation and factor of safety analysis

6.2.1 Result from deformation analysis

The deformation analysis was made for two soil critical slope sections and two rock critical slope sections under both static and dynamic loading conditions in the dry and wet state of the slope. During deformation analysis staged construction was used for static loading conditions and a total multiplier is used for dynamic analysis for all critical slope sections. The result shows that the slope material is highly deformed during the wet conditions of the slope or when the groundwater table is high. The displacement of the slope is increased slowly from static dry condition to dynamic dry condition and increased rapidly from static dry condition to static and dynamic wet condition since the factor highly affecting the displacement of the slope is the rise of the groundwater table or high amount of moisture content within the slope material. The total displacement of soil critical slope section one is 0.58703m, 0.58755m, 2.35m, and 2.38m during static dry, dynamic dry, static wet, and dynamic wet conditions respectively. From the result, it is understood that a large amount of displacement occurs during the wet state of the slope. It also shows the instability of the slope section during the wet state of the critical slope section.

The dynamic analysis was carried out using a pseudo-static method by representing the earthquake in the horizontal acceleration coefficient α = 0.06 since the study area falls under zone 1 from the seismic map of Ethiopia. Earthquake activity is not the main factor of the slope instability here but since it is about a factor of safety or stability of the slope the condition must be analyzed.

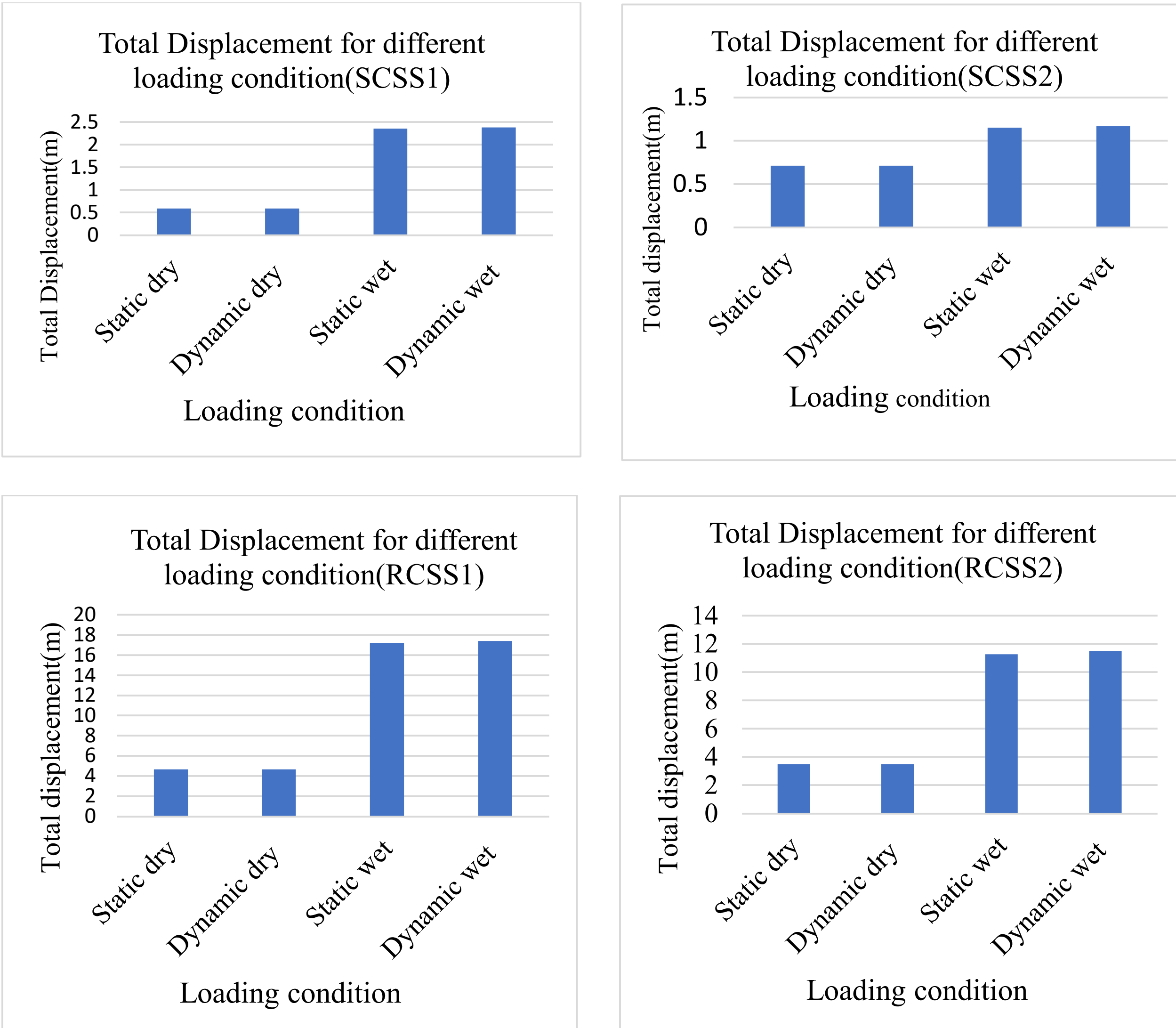


Figure 6.1 Total displacement for different loading conditions for all critical slope sections.

The total displacement of the slope sections increases from both static dry and dynamic dry to static wet and dynamic wet loading conditions rapidly due to the action of the groundwater table. A large amount of displacement is recorded on the critical rock slope sections during both static wet and dynamic wet conditions. It must be recognized that the deformation is only static (i.e. not resulted from dynamic vibration) in this loading condition.

As we can see from the result, the simulations showed that the groundwater table position with the application of seismic activity affects a large amount of displacement. But, the effect of the rise of the groundwater table was a more dominant factor affecting a large amount of displacement than seismic earthquake activity when they were applied to the slope section separately.

6.2.2 Result from the factor of safety analysis

A factor of safety of the slope sections is evaluated using both the finite element method(Plaxis 2d) and limit equilibrium method(slide software) to identify the level of stability of the critical slope sections. Shear strength reduction method and method of slice with Mohr-Columb model was used for calculation of factor of safety. The analysis was made for both static and dynamic conditions(pseudo-static analysis) under a dry and wet state of the slope.

6.2.2.1 Factor of safety using FEM(plaxis2d)

Two critical soil slope sections and two critical rock slope sections are evaluated in plaxis 2d using the shear strength reduction method as discussed in chapter four. The analysis was carried out for static dry, static wet, dynamic dry, and dynamic wet conditions. Accordingly, a factor of safety of the slope sections are decreases in the order of static dry, dynamic dry, static wet, and dynamic wet respectively, as the highly affecting factor of instability of the slope is the rise of the ground water table.

Soil critical slope section one is stable in static dry and dynamic dry conditions by a factor of safety of 1.403 and 1.402 respectively. But the slope is unstable under static wet and dynamic wet conditions by a factor of safety of 0.979 and 0.976 respectively. The slope is stable by a factor of safety of 1.403 at its initial condition and the stability of the slope reduced by 0.071%, 30.2%, and 30.4% during dynamic dry, static wet, and dynamic wet state respectively. The deformation of the slope also increases in the order of static dry, dynamic dry, static wet, and dynamic wet state of the slope respectively.

Soil critical slope section two is the same as critical soil slope section one, that the slope is stable in a dry state(static and dynamic) and unstable in a wet state(static and dynamic) by a factor of safety of 1.205, 1.204, 0.975, and 0.974 respectively. The stability of the slope is decreased from its initial condition by 0.083%, 19.1%, 19.2% during dynamic dry, static wet, and dynamic wet respectively. The deformation of the slope section is increased under the wet state of the slope during the high groundwater table.

The two rock critical slope sections are stable during both static and dynamic loading conditions under both the dry and wet states of the slope. The value of factor of safety during static dry and dynamic dry loading conditions are 2.526, 2.524 and 2.07, 2.069 for critical rock slope sections one and two respectively. The factor of safety in static wet and dynamic wet loading conditions is 1.861, 1.857 and 1.512, 1.467 for rock critical slope sections one and two respectively. The detailed values of the factor of safety for all loading conditions are given in table 6.1.

Table 6.1 Calculated values of factor of safety using plaxis 2d

	Loading condition			
Critical slope sections	Static dry	Dynamic dry	Static wet	Dynamic wet
Soil critical slope section one	1.403	1.402	0.979	0.976
Soil critical slope section two	1.205	1.204	0.975	0.974
Rock critical slope section one	2.526	2.524	1.861	1.857
Rock critical slope section two	2.07	2.069	1.512	1.467

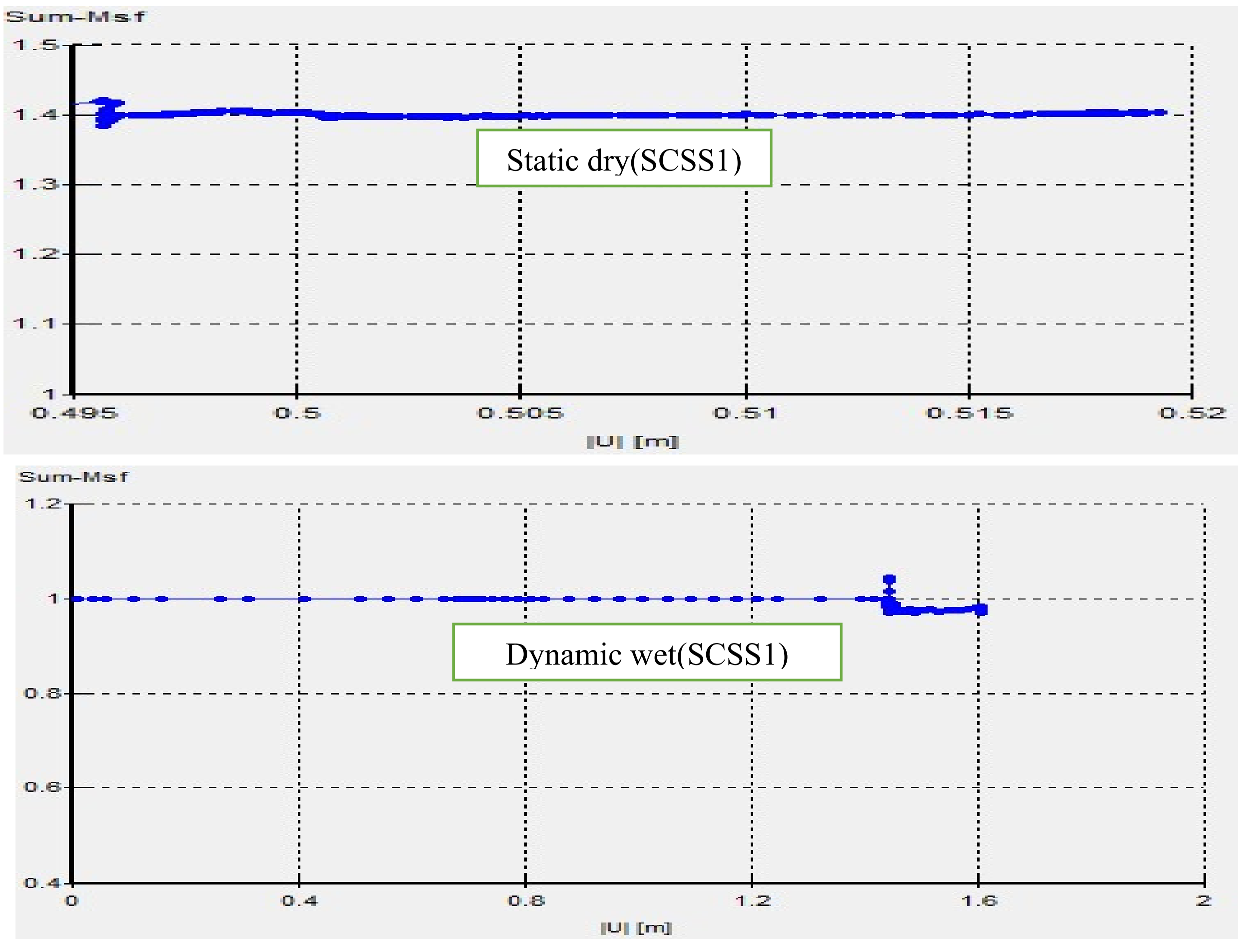


Figure 6. 2 Factor of safety versus displacement from plaxis 2d for soil critical slope section one.

6.2.2.2 Factor of safety values from LEM(slide software)

All soil critical slope sections and rock critical slope sections that are analyzed using the finite element method(using plaxis 2d) are also evaluated using the limit equilibrium method(using a slide). All slopes were evaluated for the same loading condition and slope state as analyzed in plaxis 2d. The stability of slope was evaluated using LEM (i.e.Bishop, and Janbu, Spencer, and GLE) using slide software. Limit equilibrium method(slide software) only evaluates the depth of slip surface and the factor of safety of the slope sections. The value of factor of safety from Spencer and GLE are some more than the values from Bishop and Janbu because of the condition of equilibrium they can satisfy and the way of their assumption. The analysis using slide software shows that soil critical slope sections are stable during dry conditions and unstable during the wet state of the slope sections. The rock critical slope sections are stable during both the dry and wet states of the slope. The effect of earthquake activity is very less when compared with the activity of the rise of the groundwater table during high rainfall infiltration. A factor of safety of the soil slope sections is reduced from its initial dry condition during a wet state of the slope. The highly affecting factor of the instability of the slope is the rise of the groundwater table during high rainfall infiltration.

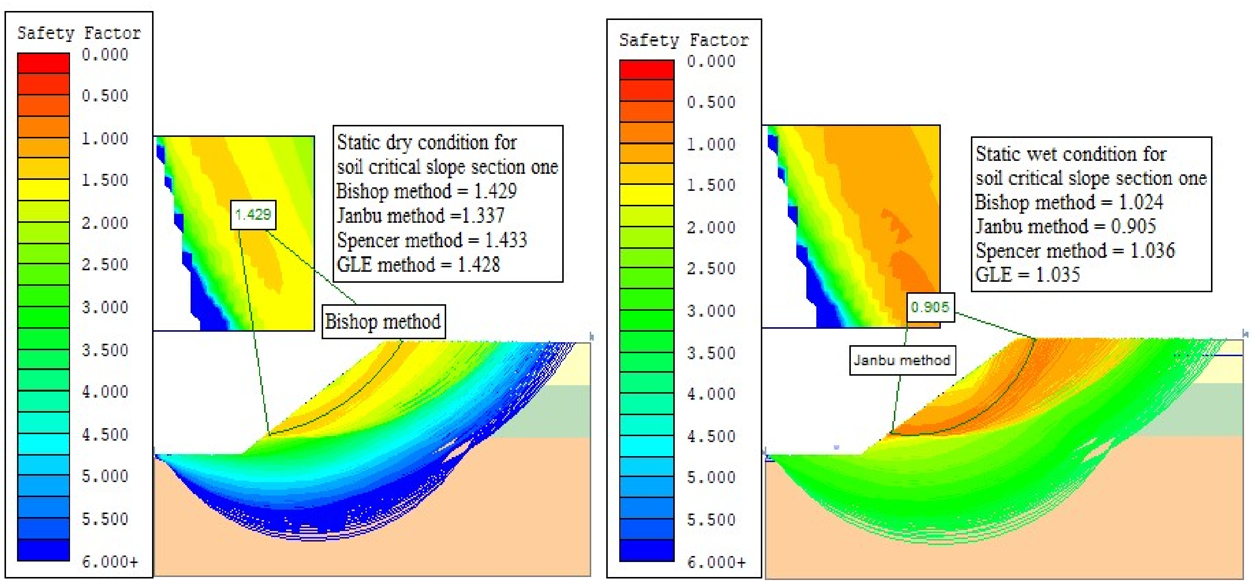


Figure 6. 3 Sample model for a calculated factor of safety using different methods of LEM.

The detailed values of factors of safety calculated using slide software are summarized in table 6.2 as shown below.

Table 6. 2 Factor of safety calculated using slide software.

Slope section	Critical slope section	Loading condition	State of the slope section	Calculated FS using different methods			
				Bishop	Janbu	Spencer	GLE
Soil slope section	SCSS1	Static	Dry	1.429	1.337	1.433	1.428
			Wet	1.024	0.905	1.036	1.035
		Dynamic	Dry	1.278	1.195	1.277	1.279
			Wet	0.987	0.888	1.001	0.996
	SCSS2	Static	Dry	1.216	1.147	1.204	1.205
			Wet	0.832	0.722	0.847	0.847
		Dynamic	Dry	1.093	1.027	1.086	1.083
			Wet	0.742	0.631	0.762	0.758
Rock slope section	RCSS1	Static	Dry	2.466	2.475	2.471	2.472
			Wet	1.961	1.970	1.983	1.966
		Dynamic	Dry	2.361	2.325	2.364	2.364
			Wet	1.820	1.788	1.802	1.823
	RCSS2	Static	Dry	1.797	1.943	1.972	2.001
			Wet	1.326	1.407	1.450	1.468
		Dynamic	Dry	1.703	1.775	1.760	1.775
			Wet	1.250	1.282	1.268	1.300

6.3 Comparison of LEM(slide based) and FEM(Plaxis 2d)

Two main concepts that should be considered when comparing LEM and FEM are, first LEM is based on plasticity, we directly solve the factor of safety of a trial slip surface, and then we need to check other trial surfaces to find the slip surface with the lowest factor of safety. Second, FEM is indirectly solving for a factor of safety by the non-convergence of the model through the SSR process which will lead to large strain and the contours of this strain identify the slip surface (no trial slip surface to assume and no searching).

The LEM and FEM are compared using results from the slide and plaxis software respectively. Outputs show that a slightly higher FS is obtained in LEM than FEM. It has been seen that results from FEM (plaxis software) give more information about the failure surface i.e. the stress, strain, deformation, and displacements at any point in the slope section. Whereas the LEM (slide) gives only uniform FS along the failure surface. Its possible to know the total displacements of the slope at each finite element node and the relative shear stress (i.e. the proximity of the stress points to the Mohr-Coulomb failure envelop) which is not accessible in LEM. Statistical analyses indicate that except for some cases none of the LEMs matched with FEM for the factor of safety values. In particular cases, both methods may over or underestimate the real case in the field. From this study, the value of factor of safety from slide software and plaxis software are agreed well in the static and dynamic dry state of the slope than in static and dynamic wet condition because the consistency between LEMs and FEMs in terms of factor of safety value decreases as the water saturation increases and also seismic coefficient, but not affecting the slope stability more as horizontal earthquake coefficient is low in zone 1. FEM depicts deeper localization of slip surface than LEM for both soil slope section and rock slope section. The result also shows that the value of the factor of safety varies within LEM depending on the method used.

6.4 Discussion on the stability of the slope sections

Slope stability analysis is very important in engineering practice to guarantee the stability of the structure and avoid accidents and monetary losses. In this research work, two soil critical slope sections and two highly weathered and fractured rock slopes are analyzed using the finite element method and limit equilibrium method. The analysis involves both static and pseudo-static analysis.

Theoretically, all slopes with a factor of safety greater than unity are safe. However, many design standards recommend the use of FS greater than one. Because, with time there are certain processes or events (rainfall, weathering, earthquake, etc) that tends to destabilize the slope reducing temporary or permanently the factor of safety of the slope. At some point, one of those factors will result in reducing the factor of safety to less one and subsequently, the slope fails or landslide occurs. The two soil slope sections are marginally stable during the static dry and dynamic dry conditions and unstable during static wet and dynamic wet loading conditions. The factor of safety of the slope is decreased from its initial static dry condition by 0.077%, 24.65%, and 24.8% on average during dynamic dry, static wet, and dynamic wet conditions respectively.

The factor of safety of the soil critical slope section one is 1.403 during the static dry condition and reduced to 1.402, 0.979, and 0.976 during dynamic dry, static wet, and dynamic wet conditions respectively as calculated from plaxis 2d. The factor of safety of soil critical slope section two is also decreased from 1.205 to1.204, 0.975, and 0.974 respectively during dynamic dry, static wet, and dynamic wet. A high amount of deformation of the slope occurs during the wet state of the slope for both static and dynamic loading conditions.

The rock critical slope sections are stable for all loading conditions and slope states. A factor of safety of both rock critical slope sections is reduced from its initial static dry condition during dynamic dry, static wet, and dynamic wet analysis. The factor of safety of the rock slope sections is reduced from the initial static condition by 0.15%, 18.4%, and 19.635 on average during dynamic dry, static wet, and dynamic wet conditions respectively. The value of factor of safety during static dry and dynamic dry loading conditions are 2.526, 2.524 and 2.07, 2.069 for critical rock slope sections one and two respectively. The factor safety during static wet and dynamic wet loading conditions is 1.861, 1.857 and 1.512, 1.467 for critical rock slope sections one and two respectively.

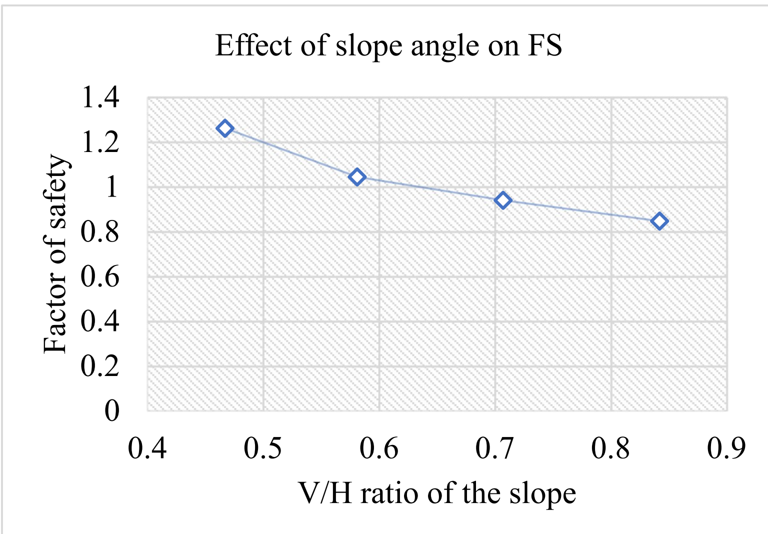
Generally, the two soil critical slope sections are marginally stable during the dry condition for both static and dynamic loading conditions. Soil slope sections one two are unstable under static wet conditions and dynamic wet conditions. Accordingly, rainfall activity is the parameter that highly affecting the instability of the slope sections. Under these rainfall conditions, as rainwater infiltrate into the slope the matric suction in the soil decrease and as the result, the apparent shear strength associated with matric suction can decrease, causing the slope to instability. The rock slope sections are stable during dry and wet states of the slope under both static and dynamic loading conditions.

6.5 Effect of different slope profiles on slope stability

6.5.1 Single slope profile and the effect of different slope angle

Increasing slope angle increases the shear stress while decreasing shear strength and decreasing of slope angle decrease shear stress and increase shear strength on the potential failure plane depending on the nature of the slope material. The effect of slope angle is examined using an unstable soil slope section under wet conditions.

The slope is modeled and calculated for 40, 35, 30, and 25 to examine the performance of a single slope profile. The value from the spencer method is used to draw the factor of safety versus the V/H ratio of the slope. The result shows a factor of safety is increased as the V/H ratio of the slope decreases. From the comparative study of slope modification methods flattening the slope(from real site condition i.e. 40º) to 35º, 30º, 25º increases the factor of safety of the slope by 9.89%, 18.87%, and 32.88% respectively as shown in figure 6.4.

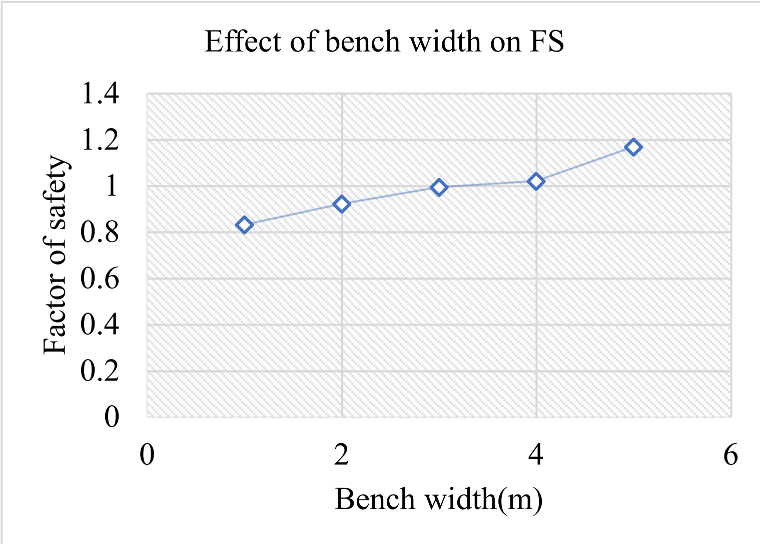


V/H ratio	Factor of safety
0.842	0.847
0.707	0.94
0.581	1.044
0.467	1.262

Figure 6. 4 Effect of slope angle on the FS of the slope.

6.5.2 Effect of benching and bench width on Factor of safety

Benching of the slope is a technique used to avoid the design of very high and gentle slopes on a single slope profile. It is a method in which the overall slope is divided into multiple small slopes. The method of benching will allow the forces of cohesion and internal friction to hold the soil together and keep it from flowing downs the face of the slope. For this study, soil critical slope section two under static wet conditions was used to carry out the performance evaluation of the benching technique. The effect of benching is investigated using 10m slope height by two bench numbers for 40º slope angle which is the actual condition of the critical slope section under consideration from the site.

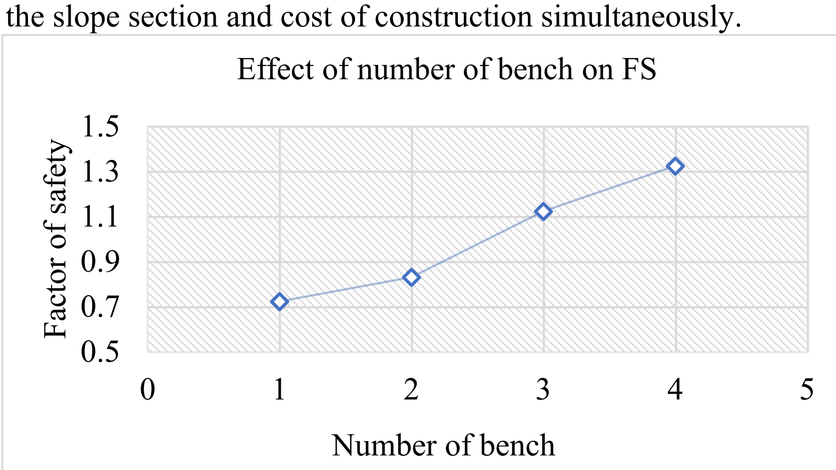


Bench width(m)	factor of safety
2	0.923
3	0.995
4	1.021
5	1.169

Figure 6. 5 Effect of bench width on the factor of safety.

The use of 2m, 3m, 4m, and 5m bench width by keeping the slope angle constant and slope height of 10m increases the factor of safety by 8.23%, 14.87%, 17.04%, and 27.54% respectively when two numbers of the bench are used.

The other is the effect of the number of the bench on the stability of the slope section. The evaluation is made for 1 to 5 number of benching. As shown in figure 6.6 a factor of safety increases as the number of benches increases but the problem is the cost of constructing the number of benches on the slope face. So, 2 number of the bench is selected by considering the stability of the slope section and cost of construction simultaneously.



No. of bench	Factor of safety
1	0.725
2	0.832
3	1.125
4	1.326

Figure 6. 6 Effect of the number of benches on slope stability.

6.5.3 Effect of multi-slope profile on slope stability

The multi slope is a type of slope geometry in which the angle of the slope section varies according to slope section material. In these methods, the steep and gentle slope is used in stiff and loses materials respectively when a single slope comprises stratified soil.

To analyze the performance of multi-slope geometry different combination of slope angles was used for this research work.

Table 6 3Effect of multi-slope geometry on the stability of the slope.

1.Lower thick layer	40	40	40	40	40	40
2.Upper layer	40	37.8	35.3	32.4	29	25
1/2 ratio	1.0	1.05	1.13	1.2	1.4	1.6
FS	0.847	0.934	0.97	1.027	1.124	1.451

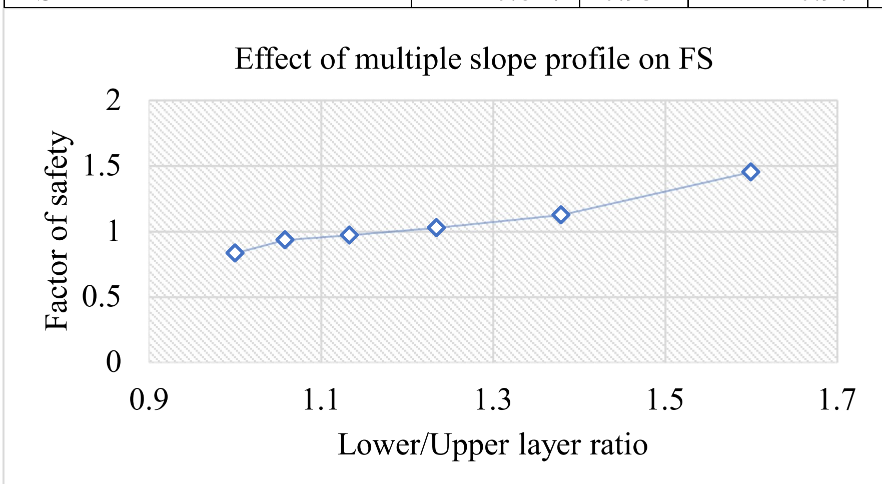


Figure 6. 7 Effect of using a multi-slope profile on the stability of the slope

The lower slope angle is kept constant as the real condition found on the site since the lower layer is composed of rock material starting from the toe of the slope to 5m along the face of the rock. As the lower layer is composed of soil and rock material the layer is assumed to be a stiff layer and the angle of the layer is kept constant on the angle measured from the site which is 40 degrees. The angle of the upper layer is reduced until the slope comes to a state of stability. The factor of safety is increased from 0.847 to 1.451 from the current real condition on the site to reduced upper layer slope angle to 25 degrees respectively.

6.5.4 Effect of soil nail on slope stability

The application of the soil nail increases the stability of the slope until 30m of soil nail length and the factor of safety of the slope remains constant for soil nail length of more than 30m. The soil nail length of 20m, 22m, 25m, 30m, 35m, and 40m with 2m of nail spacing are applied along the slope face and the factor of safety are 1.042, 1.137, 1.303, 1.637, 1.637, and 1.637 respectively. The use of 20m, 22m, 25m, 30m, 35m, and 40m increases the factor of safety of the slope by 18.7%, 25.5%, 35%, 48.3%, 48.3%, and 48.3% from the initial slope condition (slope without a nail) respectively. The result indicates that the factor of safety of the slope increases with increasing of the soil nail length. But, if the length of the soil nail passes an effective value of L (20m – 30m) the factor of safety of the slope does not change significantly. The length of the nail doesn't affect the stability of the slope if it is with in the slip surface without touching it because if the nail is not crossing the slip surface it cannot improve the slope. If the length of the soil nail is very long it doesn't affect the stability of the slope beyond the effective length of the soil nail as the length of the soil nail far away from the slip surface donnot participate in slope improvement.

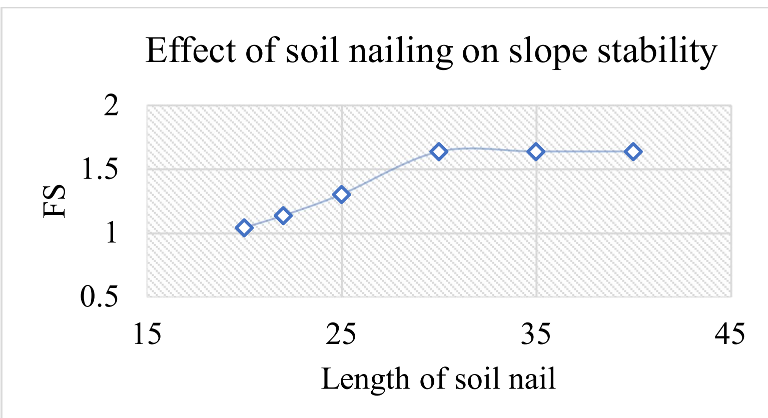


Figure 6. 8 Effect of soil nail on slope stability.

6.6 Comparative evaluation of slope profiles

Modification of the slope geometry is a very economical alternative to slope stability improvement. From many slope improvement techniques using slope geometry modification three of them (i.e flattening the slope, benching slope, and using multi-profile slope geometry) are discussed in the previous section of this paper. The selection of measures always depends upon the site, the topography, and the required result (i.e. susceptibility of the slope to erosion and infiltration, the variability of slope material, the height of slope, and adjacent area of the slope) as well as the cost of construction.

Proper selection and design of any measures play a very important role in slope stabilization and the control of erosion and measures should only be undertaken as the result of an integrated planning process.

The use of flattening the slope(from real site condition i.e. 40º) to 35º, 30º, 25º increases the factor of safety of the slope by 9.89%, 18.87%, and 32.88% respectively. The use of 2m,3m, 4m, and 5m bench width by keeping the slope angle constant and slope height of 10m increases the factor of safety by 8.23%, 14.87%, 17.04%, and 27.54% respectively when two numbers of the bench are used. The number of benches has its effect on the performance of this method of slope modification but increasing the number of benches may be uneconomical. Using multi-slope profile by keeping the lower stiff material with a constant slope and changing the slope of upper slope material. The use of upper layer/lower layer slope angles of 1.05, 1.13, 1.2, 1.4, 1.6 increase the factor of safety of the slope by 9.31%, 12.68%, 17.53%, 24.64%, and 41.63% respectively. The use of 20m, 22m, 25m, 30m, 35m, and 40m increases the factor of safety of the slope by 18.7%, 25.5%, 35%, 48.3%, 48.3%, and 48.3% respectively.

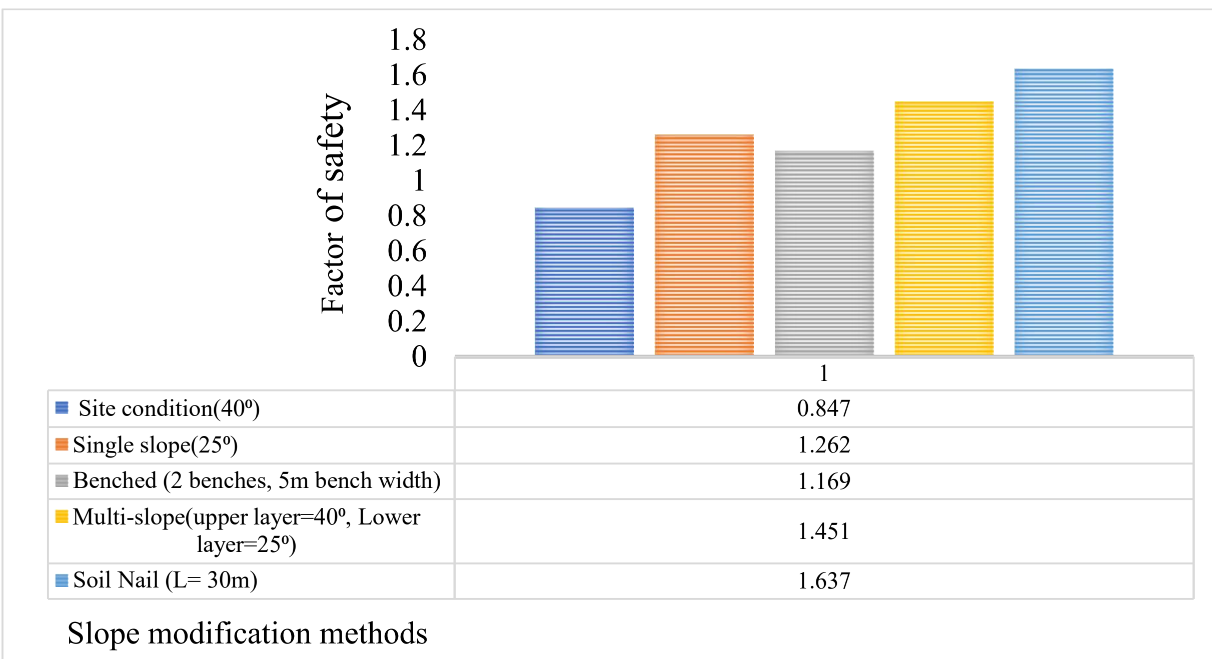


Figure 6. 9 Comparison of different slope geometry modification methods.

The use of a single slope profile is effective in homogenous stiff slope material when the height of the slope is low. Otherwise, the method may not be a safe and economical choice as weak and high slope sections need very gentle slopes. Multi slope profiles are suitable in slopes where there is material strength variability. It provides a very economical slope design without extra excavation by adjusting only within the slope section. Especially the method is ideal in slopes that comprise both rock and soil.

The use of a bench is an effective geometric measure of instability improvement when the height of the slope is large. It increases FS by reducing the driving force above the failure surface. In addition to its direct impact on the FS of the slope, these slope profiles have advantages and limitations. Therefore, the selection of slope profiles should be based on site-specific considerations of drainage, need for maintenance, aesthetic value, and need for extra excavation.

The use of the benching method of slope geometry stabilization method is recommended for the unstable slope section investigated under this research work. Because the highly affecting factor which causes the slope instability is rainfall infiltration or the rise of the groundwater table. Accordingly, the benching method of slope stabilization is suitable for steep slope and it reduces the area of the slope exposed to rainfall infiltration as it allows the use of steep slope between every benching also it provides effective drainage control by collecting the rainwater from each slope profile and draining it laterally to ditches.

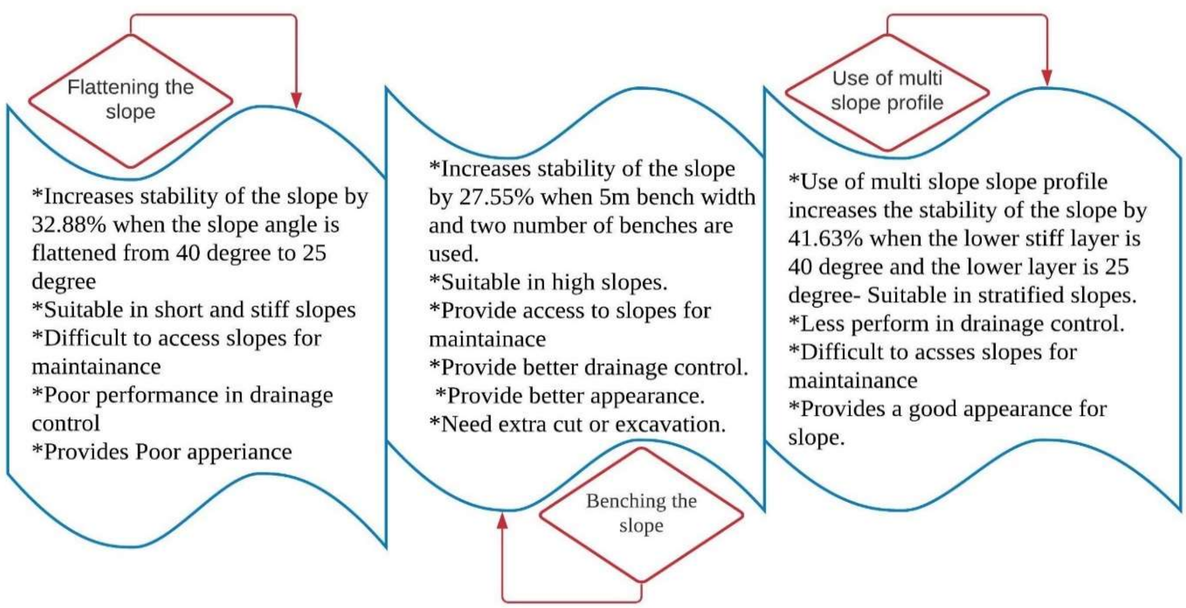


Figure 6. 10 Geometry modification techniques for slope stabilization methods.

CHAPTER SEVEN

CONCLUSION AND RECCOMENDATION

7.1 Conclusion

The primary requirement of slope analysis and design is to ensure their safety and reliability throughout their service life. The stability analysis and comparative study of the slope modification methods are evaluated in this study. The study was carried out on road cut slope from Ambo to Gedo. The study area of this study falls under zone 1 of seismic earthquake activity from the Ethiopian earthquake map. The study area has high rainfall intensity during July and august which reduces the shear strength of slope material leading seepage related instability problems.

Critical slope sections are identified during field investigation by considering some failure manifestations that may be responsible for slope failure. Two soil critical slope sections and two rock critical slope sections are analyzed in this research work. All data required for the study was taken and a Schmidt rebound hammer test was done on rock sections of the slope during field investigation. Then soil sample was taken from the slope section for laboratory investigation to determine index properties and strength parameters of the slope. The strength parameter of the rock mass slope section was determined using rocdata software. The stiffness parameters for the soil and rock materials were determined from correlations.

The analysis was carried out using the finite element method (plaxis 2d) and limit equilibrium method (slide) for dry and wet slope states under both static and dynamic (pseudo-static) loading conditions. Mohr-Columb material model and strength reduction method were used in finite element analysis. The method of slice was used in the limit equilibrium method. The following conclusions are made from the stability analysis:

1. The two soil critical slope sections are stable during dry state(static dry and dynamic dry) and unstable during wet state(static wet and dynamic wet) when there is high rainfall infiltration. The slope sections are stable during the dry state by a factor of safety of between 1.204 to 1.403. The slope sections are unstable during the wet state of the slope by a factor of safety of between 0.974 to 0.979 under both static and dynamic loading conditions.

2. The two rock critical slope section is stable in dry and wet state of the slope for both static and dynamic loading condition. The value of the factor of safety of the rock slope sections in a dry state is between 2.069 and 2.526 and between 1.467 and 1.861 in a wet state for static and dynamic loading conditions.

Comparative study of slope geometry modification methods is also carried out on critical soil slope section one in static wet conditions. The use of flattening the slope, benching slope, using multi slope profile, and the use of soin nailing method was studied and their performance is evaluated.

1. From the comparative study of slope modification methods flattening the slope(from real site condition i.e. 40^{o}) to 35^{o}, 30^{o}, 25^{o} increases the factor of safety of the slope by 9.89%, 18.87%, and 32.88% respectively.
2. The use of 2m,3m, 4m, and 5m bench width by keeping the slope angle constant and slope height of 10m increases the factor of safety by 8.23%, 14.87%, 17.04%, and 27.54% respectively when two numbers of the bench are used. The number of benches has its effect on the performance of this method of slope modification but increasing the number of benches may be uneconomical.
3. Using multi-slope profile by keeping the lower stiff material with a constant slope and changing the slope of upper slope material. The use of upper layer/lower layer slope angles of 1.05, 1.13, 1.2, 1.4, 1.6 increase the factor of safety of the slope by 9.31%, 12.68%, 17.53%, 24.64%, and 41.63% respectively.
4. The soil nail length of 20m, 22m, 25m, 30m, 35m, and 40m with 2m of nail spacing are applied along the slope face and the factor of safety is 1.042, 1.137, 1.303, 1.637, 1.637, and 1.637 respectively. The use of 20m, 22m, 25m, 30m, 35m, and 40m increases the factor of safety of the slope by 18.7%, 25.5%, 35%, 48.3%, 48.3%, and 48.3% from the initial slope condition (slope without a nail) respectively. The result indicates that the factor of safety of the slope increases with increasing of the soil nail length. But, when the length of the soil nail passes an effective value of L (20m – 30m) the factor of safety of the slope does not change significantly.

7.2 Recommendation

Two soil slope sections, as well as two rock slope sections, are studied in this research work. Comparative studies of slope modification methods and some other mitigation measures are also analyzed in this study. The selection of appropriate mitigation measures should be based on an assessment of risk, uncertainty, possible consequences, constructability, environmental impacts, and costs of construction. The use of a combination of surface drainage with benching is recommended which means lateral drainage (bench drain) to collect rainwater from each bench slope and drain it laterally to spillways for the study area, because, benching is suitable for high slopes and provide better drainage control.

In this study, cost analysis study don not carried out during a comparative study of the slope modification methods but it is highly recommended to do their respective cost analysis since construction cost is the second most important thing next to the safety performance of the slope construction methods.

Even if FEM provides better information about failure mechanisms, still LEM is commonly used in the evaluation of slopes. Therefore, it is better to use deformation-based FEM for safe and economical designs of slopes.

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APPENDIX

Appendix I: Strength parameters and index properties of soil from laboratory tests

1. Grain size distribution of the soil.

The distribution of different particle sizes affects the engineering properties of the soil. Particle size analysis provides a particle size distribution, which is necessary for soil classification. Particle size determination is a method of determining the size characteristics of target particles. The dry sieve analysis method is used in this research work and the results of the tests are discussed as shown below. Percentage of gravel, sand, and fine soils are determined from the sieve analysis test and the results are discussed below. Also, the value of the coefficient of curvature and coefficient of uniformity of the soil is determined for purpose of classification of the soil.

Formulas used in the grain size analysis are:

$$\text{Mass of the soil retained (gm)} = (\text{mass of sieve} + \text{retained soil}) - (\text{mass of sieve})$$

$$\text{Percentage of soil on each sieve}(\%) = \frac{\text{Mass of soil retained on the sieve}}{500} * 100$$

$$\text{Cumulative retained on the sieve}(\%) = \sum \text{Percentage of the soil on each sieve}$$

$$\%\text{Finer} = 100 - \text{Cumulative \% retained on the sieve}(\%)$$

$$\text{Coefficient of curvature} = \frac{D_{30}^2}{D_{60} * D_{10}}$$

$$\text{Coefficient of uniformity} = \frac{D_{60}}{D_{10}}$$

Where; D_{10}, D_{30}, D_{60} are grain diameters known as effective size corresponding to 10% finer, 30% finer, and 60% finer respectively. Grain diameters are in mm and these parameters are calculated using interpolation from the %finer of grain size analysis chart. Detail for each critical slope section is discussed on the next pages.

Table A.1 Grain size distribution table for soil critical slope section one layer one

seive #	seive size (mm)	Mass of each seive(g)	M. seive + retained soil(g)	Mass of soil retained(g)	% on each seive	Com. % retained	% finer	Soil classificatio	
4	4.75	441	518	77	15.4	15.4	84.6	15.4	Gravel
8	2	468	528	60	12	27.4	72.6		
16	1.18	434	498	64	12.8	40.2	59.8		
30	0.6	406	481	75	15	55.2	44.8		
40	0.425	367	437	70	14	69.2	30.8		
50	0.3	373	377	4	0.8	70	30		
100	0.15	341	407	66	13.2	83.2	16.8		
200	0.075	335	370	35	7	90.2	9.8	74.8	Sand
pan		252	301	49	9.8	100	0	9.8	Fine soil
Total				500	100				

D60 = 1.19 D30 = 0.42 D10 = 0.08 CC = 1.93 Cu = 15.84

Table A.2 Grain size distribution table for soil critical slope section layer two.

seive #	seive size(mm)	Mass of each seive(g)	M. seive + retained soil(g)	Mass of soil retained(g)	% on each seive	Com. % retained	% finer	Soil classification	
4	4.75	441	512	71	14.2	14.2	85.8	14.2	Gravel
8	2	468	575	107	21.4	35.6	64.4		
16	1.18	434	526	92	18.4	54	46		
30	0.6	406	492	86	17.2	71.2	28.8		
40	0.425	367	407	40	8	79.2	20.8		
50	0.3	373	379	6	1.2	80.4	19.6		
100	0.15	341	384	43	8.6	89	11		
200	0.075	335	346	11	2.2	91.2	8.8	77	Sand
pan		252	296	44	8.8	100	0	8.8	Fine soil
Total				500	100				

D60 = 1.8 D30 = 0.64 D10 = 0.12 Cc = 1.96 Cu = 15.56

Table A.3 Grain size distribution table for soil slope section two layer one.

seive #	seive size(mm)	Mass of each seive(g)	M. seive + retained soil(g)	Mass of soil retained(g)	% on each seive	Com. % retained	% finer	Soil classification	
4	4.75	439	499	60	12	12	88	12%	Gravel
8	2	467	620	153	30.6	42.6	57.4		
16	1.18	434	540	106	21.2	63.8	36.2		
30	0.6	406	485	79	15.8	79.6	20.4		
40	0.425	367	387	20	4	83.6	16.4		
50	0.3	373	373	0	0	83.6	16.4		
100	0.15	341	383	42	8.4	92	8		
200	0.075	334	342	8	1.6	93.6	6.4	81.6%	Sand
pan		251	275	24	4.8	98.4	1.6	4.8%	Fine soil
Total				492	98.4				

D60 = 2.22 Cu = 12.16 D30 = 0.95 D10 = 0.18 Cc = 2.21

Table A.4 Grain size distribution table for soil critical slope section two-layer two.

sieve #	sieve size (mm)	Mass of each sieve (g)	M. sieve + retained soil(g)	Mass of soil retained (g)	% on each sieve	Com. % retained	% finer	Soil classification	
4	4.75	439	545	106	21.2	21.2	78.8	21.20%	Gravel
8	2	467	569	102	20.4	41.6	58.4		
16	1.18	434	527	93	18.6	60.2	39.8		
30	0.6	406	488	82	16.4	76.6	23.4		
40	0.425	367	387	20	4	80.6	19.4		
50	0.3	373	381	8	1.6	82.2	17.8		
100	0.15	341	392	51	10.2	92.4	7.6		
200	0.075	334	343	9	1.8	94.2	5.8	73%	Sand
pan		251	280	29	5.8	100	0	5.80%	Fine soil
Total				500	100				

D60 = 2.21 D30 = 0.83 D10 = 0.19 Cc = 1.69 Cu = 11.96

Atterberg limit test

Atterberg limit test is performed to determine the plastic and liquid limits of fine-grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, where a part of the soil in the standard cup and cut by a groove of a typical size will flow together 13 mm at the bottom of the groove. When the cup falls 10 mm below the standard liquid limit and receives 25 shock, Casa Grande's instrument runs at a rate of two shocks per second. The plastic limit (PL) is the percentage of water content at which the soil will no longer be deformed by rolling into a 3.2 mm diameter wire without breaking. Both the LL test and PL test are used to classify fine-grained soil. The detail of atterberg limit tests is discussed below.

Soil critical slope section one layer one: Liquid limit test

Table A.5 Determination of liquid limit for soil critical slope section one layer one.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan + dry soil(gm)	Weight of dry soil(gm)	Weight of water(gm)	Water content(%)	No. of blows
1	19	32	29	10	3	30.0	32
2	20	33	29	9	4	44.4	26
3	19	34	29	10	5	50.0	23
4	19	36	30	11	6	54.5	20

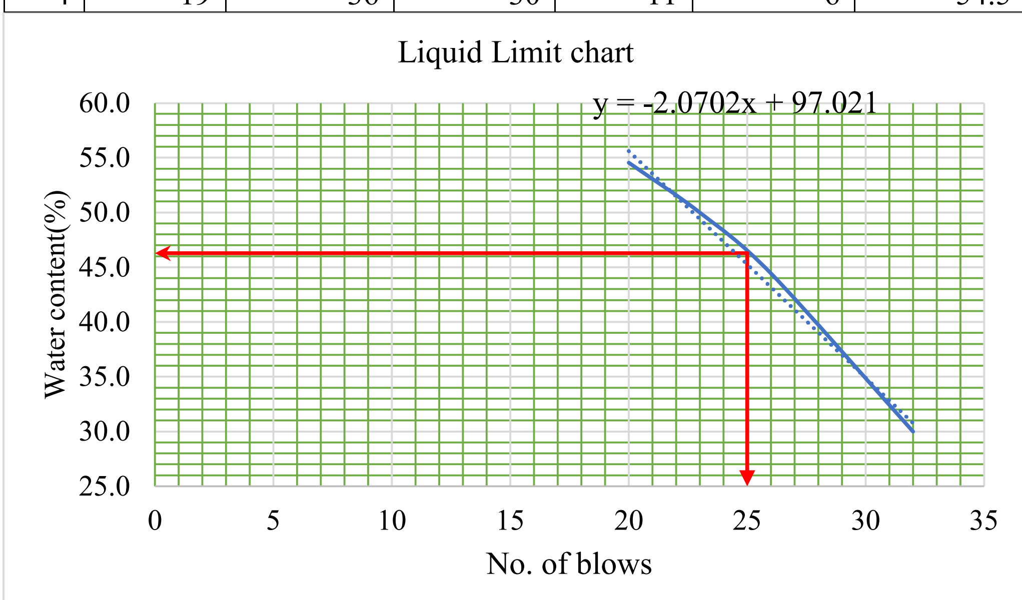


Figure A. 1 Liquid limit chart for soil critical slope section one layer one.

Soil critical slope section one layer one: Plastic limit test

Table A.6 Determination of plastic limit for soil critical slope section one layer one.

No. of can	Weight of can(gm)	Wcan+wet soil	Wcan+dry soil	weight of dry soil	Mass of moisture	Water content %
1	19	29	27	8	2	25.0
2	19	26	24	5	2	40.0
3	19	28	26	7	2	28.6

Liquid limit(LL) = 46.3

Plastic limit(PL) = 31.2

$$\text{Plasticity index(PI)} = \text{LL} - \text{PL} = 46.3 - 31.2$$

$$= 15.1$$

Soil critical slope section one layer two: Liquid limit

Table A.7 Determination of liquid limit for soil critical slope section one layer two.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan + dry soil(gm)	Weight of dry soil(gm)	Weight of water	Water content(%)	No. of blows
1	19	32	29	10	3	30.0	33
2	18	32	28	10	4	40.0	27
3	18	34	29	11	5	45.5	24
4	19	38	31	12	7	58.3	17

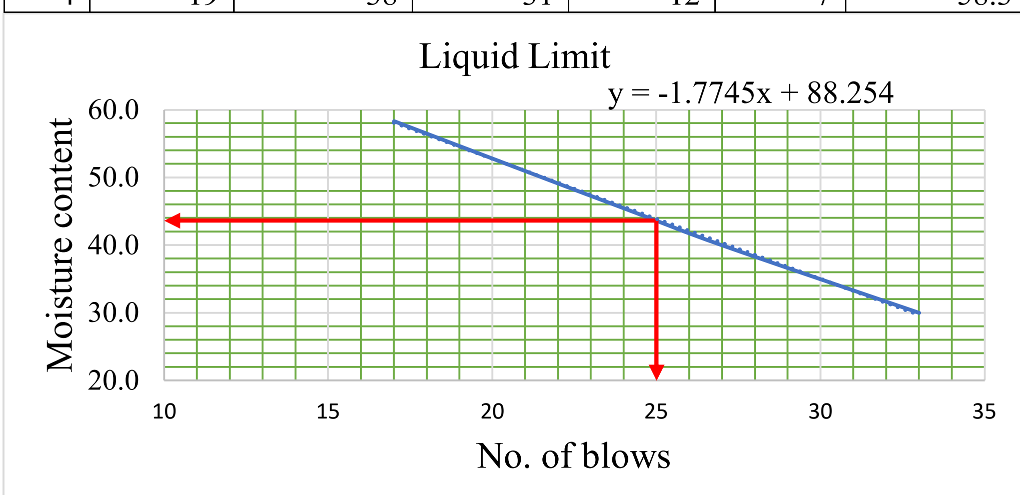


Figure A. 2 Liquid limit chart for soil critical slope section one layer two.

Soil critical slope section one layer-two: Plastic limit test

Table A.8 Determination of plastic limit for soil critical slope section one layer two.

Can No.	Weight of can(gm)	Wcan+wet soil	Wcan+dry soil	weight of dry soil	Mass of moisture	Water content %
1	18	27	25	7	2	28.6
2	18	34	30	12	4	33.3
3	19	36	31	12	5	41.7

Liquid limit(LL) = 43.6

Plastic limit(PL) = 34.5

$$\text{Plasticity index(PI)} = \text{LL} - \text{PL} = 43.6 - 34.5$$

$$= 9.1$$

Soil critical slope section two-layer one: Liquid limit test

Table A.9 Determination of liquid limit for soil critical slope section two-layer one.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan + dry soil(gm)	Weight of dry soil(gm)	Weight of water(gm)	Water content(%)	No. of blows
1	18	34	31	13	3	23.1	33
2	18	34	30	12	4	33.3	29
3	18	37	30	12	7	58.3	18

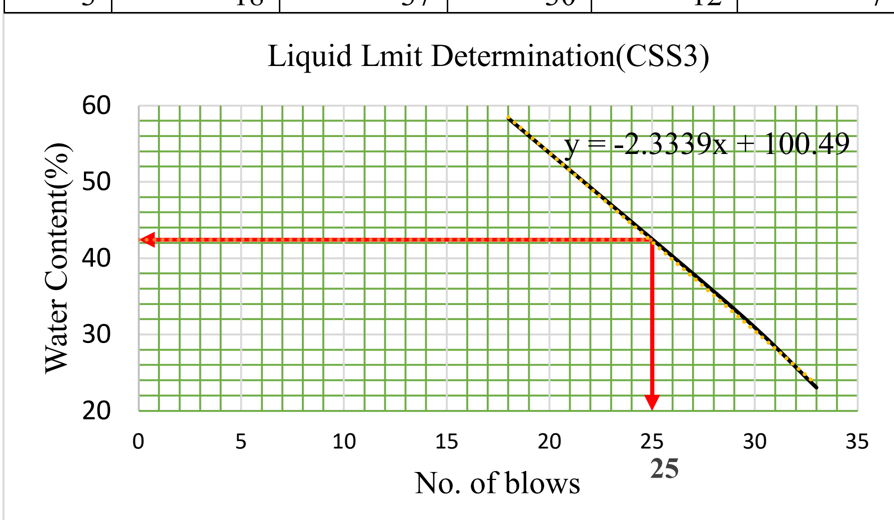


Figure A. 3 Liquid limit chart for soil critical slope section two-layer one.

Soil critical slope section two-layer one: Plastic limit test

Table A.10 Determination of plastic limit for soil slope section two-layer one.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan+dry soil(gm)	weight of dry soil(gm)	Mass of moisture (gm)	Water content %
1	19	26	24	5	2	40.0
2	19	33	30	11	3	27.3
3	19	35	32	13	3	23.1

Liquid limit(LL) = 42.42

Plastic limit(PL) = 30.12

$$\text{Plasticity index(PI)} = \text{LL} - \text{PL} = 42.42 - 30.12$$

$$= 12.31$$

Soil critical slope section two-layer two: Liquid limit test

Table A.11 Determination of liquid limit for soil critical slope section two-layer two.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan + dry soil(gm)	Weight of dry soil(gm)	Weight of water(gm)	Water content(%)	No. of blows
1	61	75	71	10	4	40	31
2	61	75	70	9	5	55.6	23
3	61	76	70	9	6	66.7	18

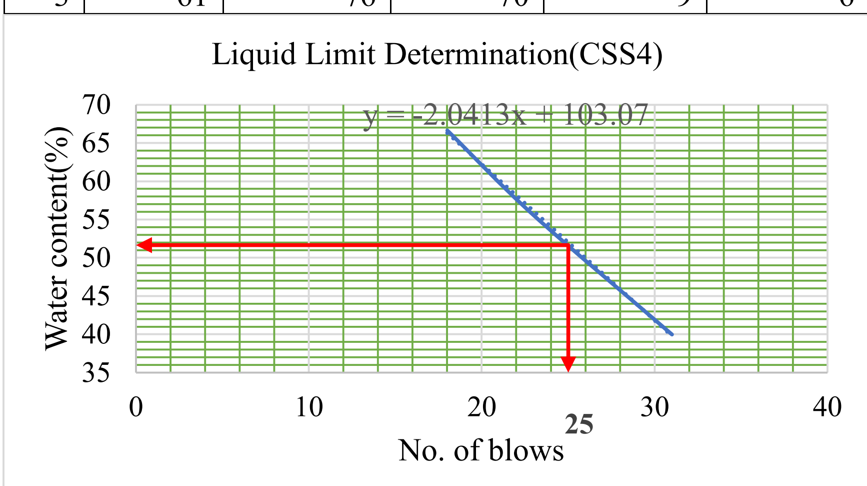


Figure A. 4 Liquid limit chart for soil critical slope section two-layer two.

Soil critical slope section two-layer two: Plastic limit test

Table A.12 Determination of plastic limit for soil critical slope section two-layer two.

Can No.	Weight of can(gm)	Wcan+wet soil(gm)	Wcan+dry soil(gm)	weight of dry soil(gm)	Mass of moisture(gm)	Water content %
1	61	68	66	5	2	40
2	61	64	63	2	1	50
3	60	65	64	4	1	25

Liquid limit(LL) = 51.7

Plastic limit(PL) = 38.3

$$\text{Plasticity index(PI)} = \text{LL} - \text{PL} = 51.7 - 38.3$$

$$= 13.3$$

2. Compaction test

It is used to determine the MDD and OMC of the soil used to remolded the soil in the shear strength test. In this work, the test is conducted using a standard proctor test, on four soil samples. The results of the tests are discussed below.

Soil critical slope section one layer one

Table A.13 Determination of compaction test for soil critical slope section one layer one.

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5
Weight of mould+wet soil(gm)	2930	3216	3465	3531	3404
Weight of mould(gm)	1725	1725	1725	1725	1725
Weight of wet soil(gm)	1205	1491	1740	1806	1679
Wet density	1.205	1.491	1.74	1.806	1.679
Can No.	1	2	3	4	5
Weight of can	20	19	19	19	18
Weight of can +wet soil	50.67	45.33	47.67	62.33	69
Weight of can +dry soil	47.33	41.5	42.33	53	56.33
Weight of water	3.34	3.83	5.34	9.33	12.67
Weight of dry soil	27.33	22.5	23.33	34	38.33
Water content(w)	12.2	17.0	22.9	27.4	33.1
Dry density(γ_{dry})	1.1	1.3	1.4	1.4	1.3

Soil critical slope section one layer two

Table A.14 Determination of compaction test for soil critical slope section one layer two.

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6
Weight of mould+wet soil(gm)	2900	3127	3412	3528	3465	3407
Weight of mould(gm)	1725	1725	1725	1725	1725	1725
Weight of wet soil(gm)	1175	1402	1687	1803	1740	1682
Wet density	1.175	1.402	1.687	1.803	1.74	1.682
Can No.	1	2	3	4	5	6
Weight of can(gm)	18	18	18	18	29	29
Weight of can +wet soil(gm)	49.33	45	46.67	55.33	81.67	74
Weight of can +dry soil(gm)	48	43	43	49	69	60
Weight of water(gm)	1.33	2	3.67	6.33	12.67	14
Weight of dry soil(gm)	30	25	25	31	40	31
Water content(*w)*	4.4	8.0	14.7	20.4	31.7	45.2
Dry density(γdry)	1.1	1.3	1.5	1.5	1.3	1.2

Soil critical slope section two-layer one

Table A.15 Determination of compaction test for soil critical slope section two-layer one.

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 4
Weight of mould+wet soil(gm)	2984	3233	3464	3516	3472
Weight of mould(gm)	1725	1725	1725	1725	1725
Weight of wet soil(gm)	1259	1508	1739	1791	1747
Wet density	1.259	1.508	1.739	1.791	1.747
Can No.	1	2	3	4	5
Weight of can(gm)	20	20	29.3	29.3	30
Weight of can +wet soil(gm)	50	45	65.3	79	81
Weight of can +dry soil(gm)	48	42.67	60	68.3	66
Weight of water(gm)	2	2.33	5.3	10.7	15
Weight of dry soil(gm)	28	22.67	30.7	39	36
Water content(*w)*	7.1	10.3	17.3	25.1	32.0
Dry density(γdry)	1.2	1.4	1.5	1.4	1.3

Soil critical slope section two-layer two

Table A.16 Determination of compaction test for soil critical slope section two-layer two.

	Trial 1	Trial 2	Trial 3	Trial 4
Weight of mould+wet soil(gm)	3147	3337	3419	3339
Weight of mould(gm)	1725	1725	1725	1725
Weight of wet soil(gm)	1422	1612	1694	1614
Wet density	1.422	1.612	1.694	1.614
Can No.	1	2	3	4
Weight of can(gm)	24	19	19	19
Weight of can +wet soil(gm)	53.67	39.67	49.3	58.67
Weight of can +dry soil(gm)	49	36	43	49
Weight of water(gm)	4.67	3.67	6.3	9.67
Weight of dry soil(gm)	25	17	24	30
Water content(*w)*	18.7	21.6	26.3	32.2
Dry density(γdry)	1.2	1.3	1.3	1.2

Formulas used in compaction test

$$\text{Wet density}(\gamma\text{bulk}) = \frac{\text{Weight of mould} + \text{wet soil(gm)} - \text{Weight of mould(gm)}}{1000}$$

$$\text{Water content} = \left[\frac{\text{Weight of water(gm)}}{\text{Weight of dry soil(gm)}}\right]*100$$

$$Dry\ density(\gamma dry) = \frac{\text{Wet density}(\gamma\text{bulk})}{1 + \frac{\text{Water content}(w)}{100}}$$

Summary of compaction test for all critical slope sections

Table A.17 Summarized values of MDD and OMC for all critical slope sections.

	CSS1	CSS2	CSS3	CSS4
MDD(g/cc)	1.467	1.51	1.485	1.35
OMC(%)	25	18	18	24

Direct shear test

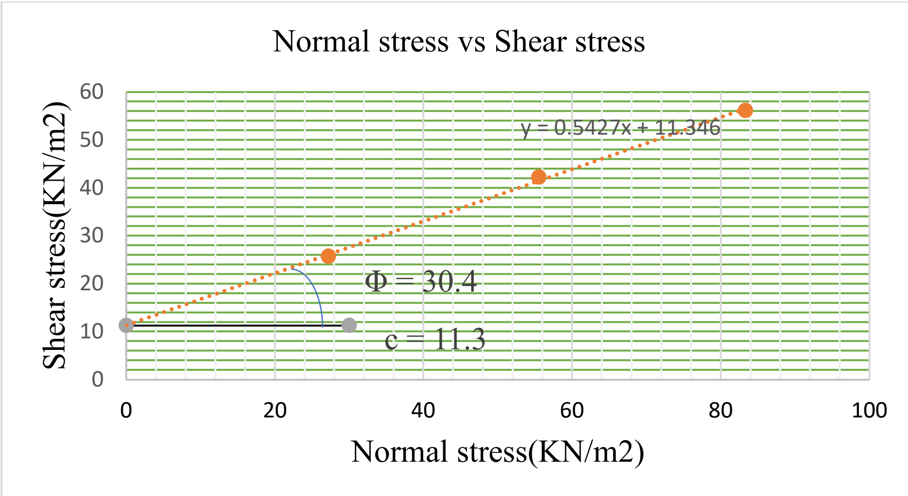


Figure A. 5 Normal stress versus shear stess for critical slope section one layer one.

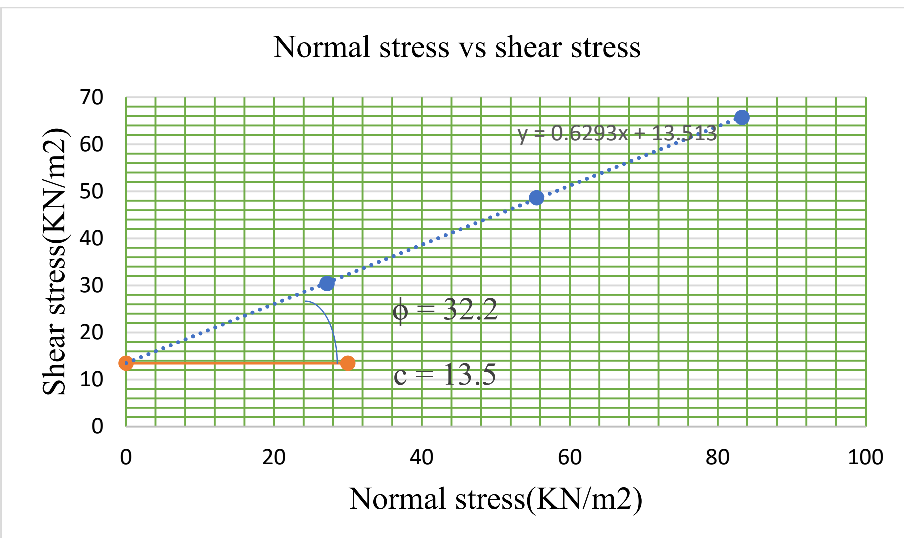


Figure A. 6 Norma stress versus shear stress for critical slope section one layer two

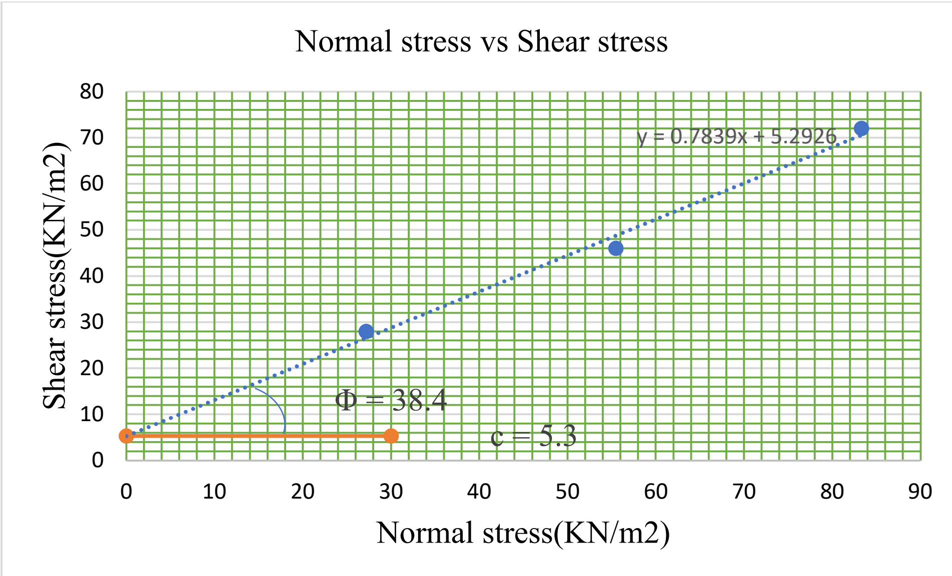


Figure A. 7 Normal stress versus shear stress for critical slope section two layer one

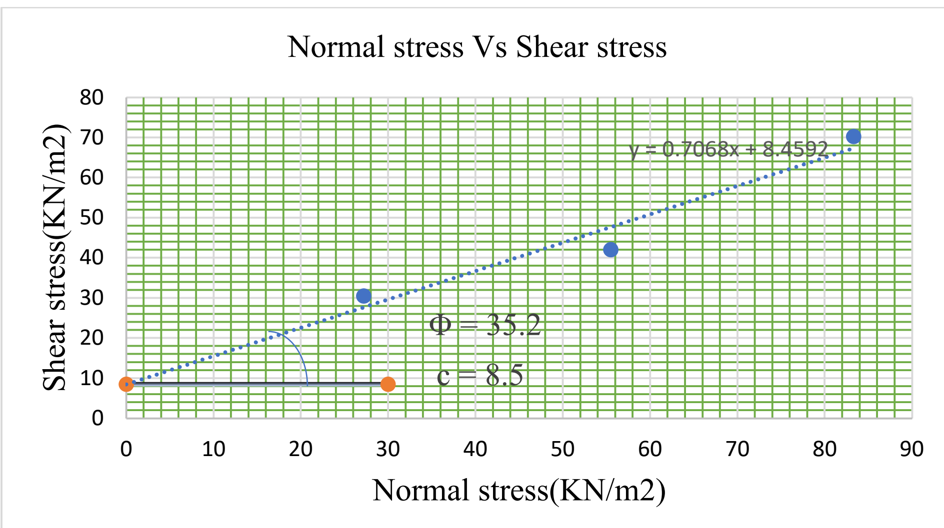


Figure A. 8 Normal stress versus shear stress for critical slope section two layer two.

Appendix II: UCS of rock and rock strength parameters

1. Uniaxial copressive strength of rock from rebound hammer test

The uniaxial compressive strength of the rock slope sections underneath the soil slope sections(i.e. SCSS13 and SCSS23) and rock critical slope section one and two is determined using schimdth rebound hammer test and values of the test are summarized as shown in table A.18.

Table A.18 UCS of the rock critical slope sections from schimdth rebound hammer test.

Slope section	Trial	Rebound number (R)	Average Rebound No.	UCS from Equ.	UCS from curve
	1	14,16,20,24,19,15,20,14,19,18	17.9	26.765155	28
	2	20,22,26,25,27,23,25,29,24,21	24.2	37.543578	39
	3	25,30,32,38,36,35,29,32,28,37	32.2	57.697769	54
RCSS1	Average			40.668834	40.3333333
	1	28,21,22,28,29,21,22,27,28,30	25.6	36.287521	37
	2	23,26,27,25,24,29,28,30,22,23	25.7	34.754003	38
	3	21,26,26,20,29,33,32,20,22,24	25.3	37.782155	37
RCSS2	Average			36.27456	37.3333333
	1	28,34,29,31,29,23,22,24,28,31	27.9	40.834076	38
	2	27,25,24,26,23,21,29,26,28,31	26	37.161468	36
	3	28,28,26,24,23,29,22,24,22,28	25.4	36.071786	35
SCSS13	Average			38.022443	36.3333333
	1	32,36,35,37,28,35,30,32,36,36	33.7	58.398231	56
	2	34,38,33,32,36,35,37,34,33,39	35.1	62.780219	61
	3	30,27,36,31,30,29,32,29,34,37	31.5	52.121929	51
SCSS23	Average			57.766793	56

2. Modulus of deformation and poisons ratio of the rock mass underneath soil critical slope section one and two (A) and rock critical slope section one and two (B).

Table A.19 Deformation modulus and poisons ratio of the rock mass.

A.

Sample code	UCS(Mpa)	D	mi	GSI	E(Mpa)	v
SCSS13	38	0.7	25 ± 5	30	1267.08	0.322
SCSS23	57.8	0.7	25 ± 5	45	3705.76	0.292

B.

Sample code	UCS(Mpa)	D	mi	GSI	E(Mpa)	v
RCSS1	40.7	1	25 ± 5	45	2392.03	0.31
RCSS2	36.3	1	25 ± 5	40	1694.04	0.302

Table A.20 Mohr coulomb equivalent shear strength parameters from Hoke's-brown generalized criterion for rock mass structure for rock underneath soil critical slope section one and two (Rocdata 3.0 Rocscience, 2004).

sample code	structure	surface condition	ϕ	C(kpa)	mb	s	a
SCSS13	Blocky disturbed	Poor	40.66	209	0.534	3.93E-05	0.522
SCSS23	Very blocky	Fair	50.8	377	1.218	0.0003	0.508

Table A.21 Mohr coulomb equivalent shear strength parameters from Hoke's-brown generalized criterion for rock mass structure for rock critical slope section one and two (Rocdata 3.0 Rocscience, 2004).

Sample code	Structure	Surface condition	ϕ	C(kpa)	mb	s	a
RCSS1	Blocky	Good	39.78	265	0.492	0.0001	0.508
RCSS2	Blocky	fair	36.37	206	0.344	4.54E-05	0.511

Appendix III: Output from plaxis 2d and slide software.

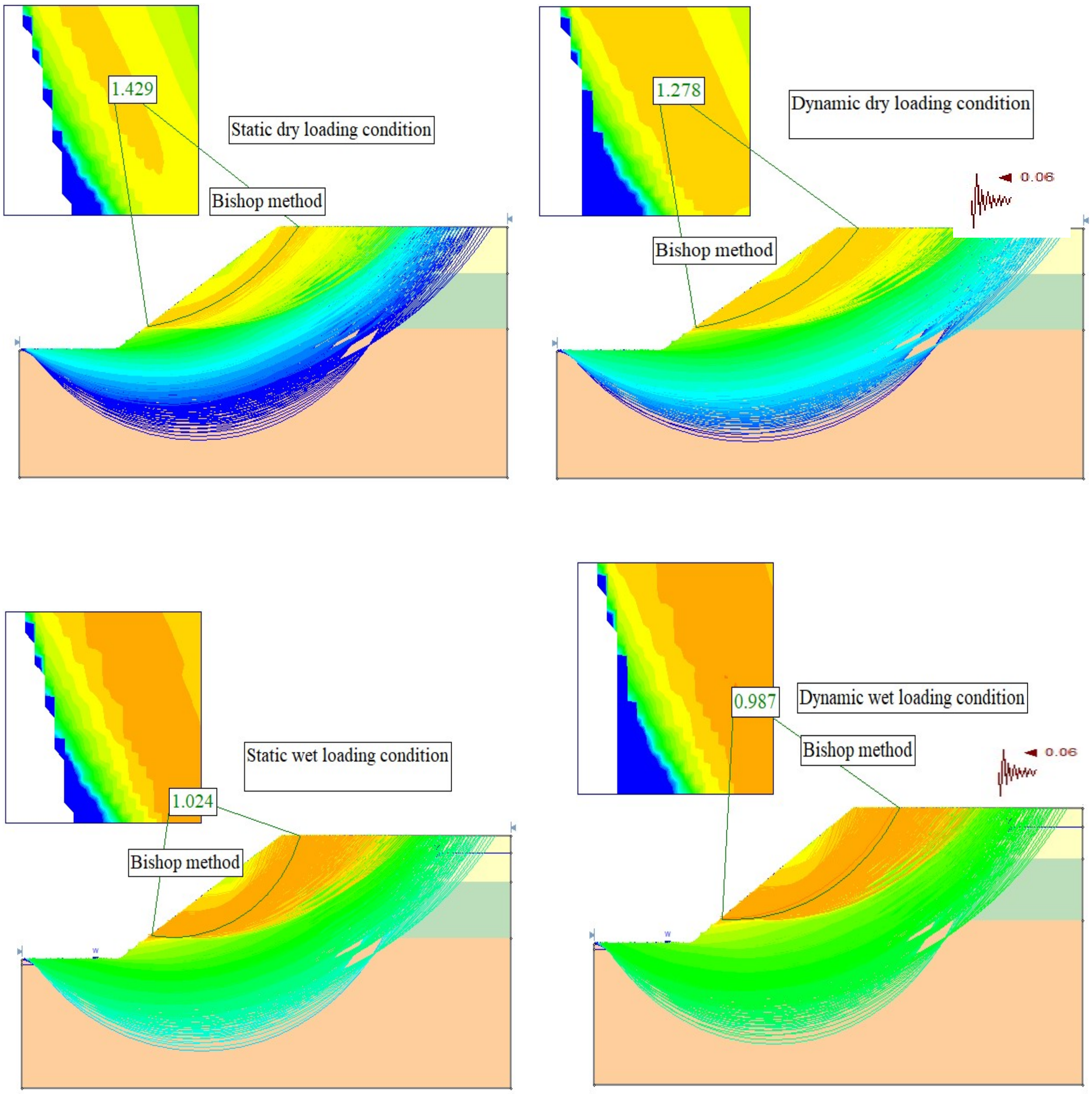


Figure A.9 Slope model and output from slide software (SCSS1).

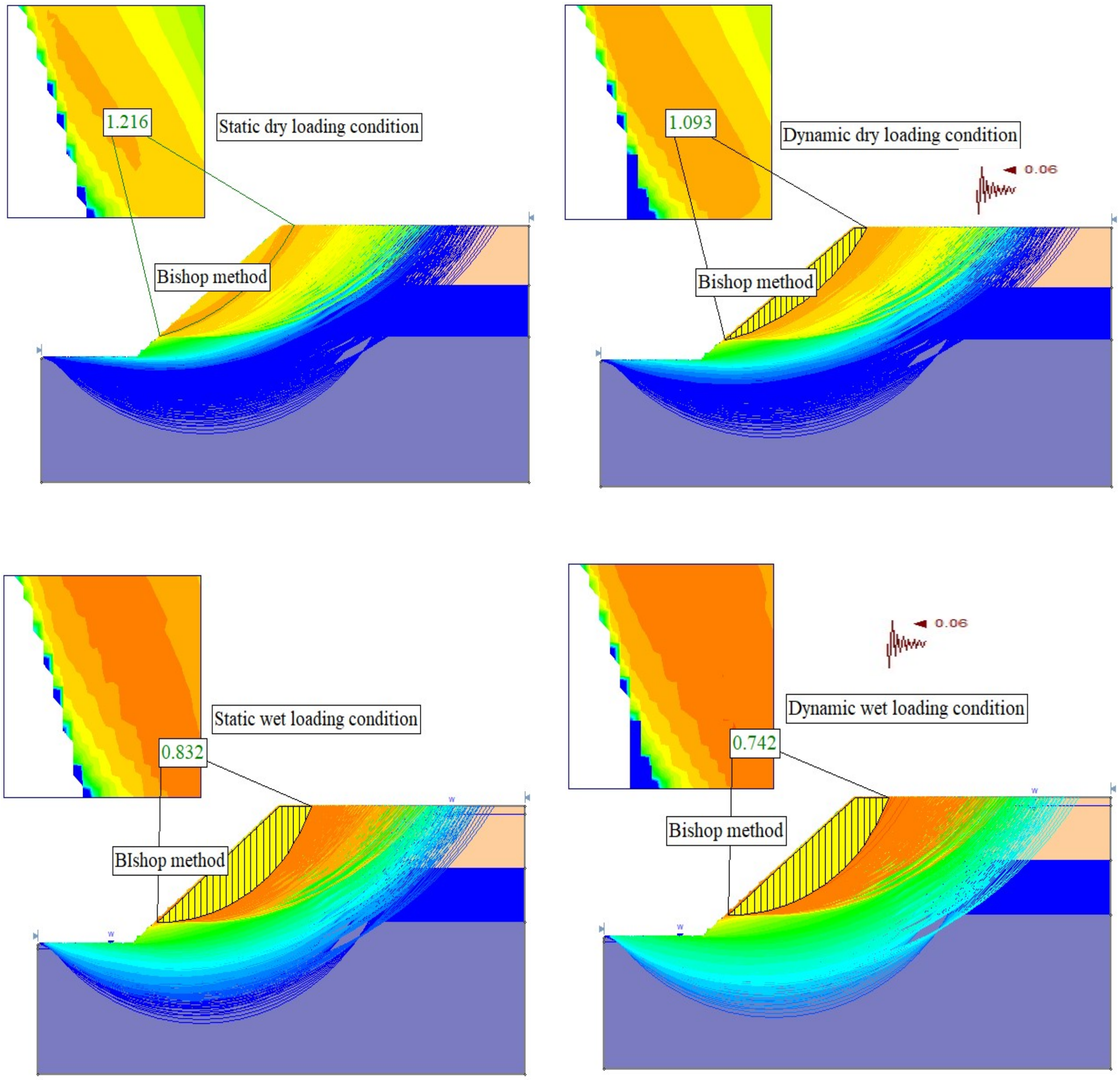


Figure A.10 Slope model and output from slide software (SCSS2).

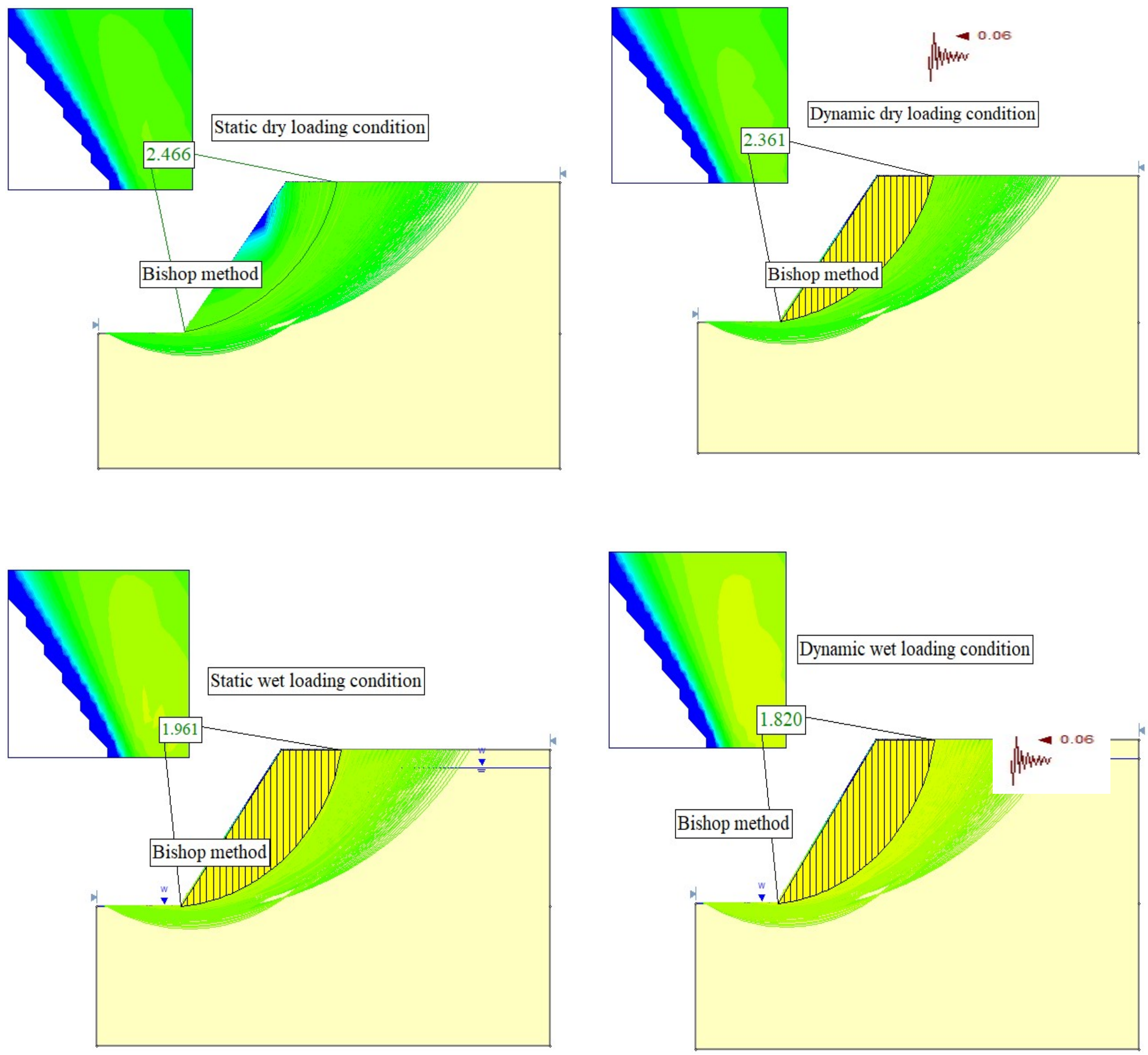


Figure A.11 Slope model and output from slide software (RCSS1).

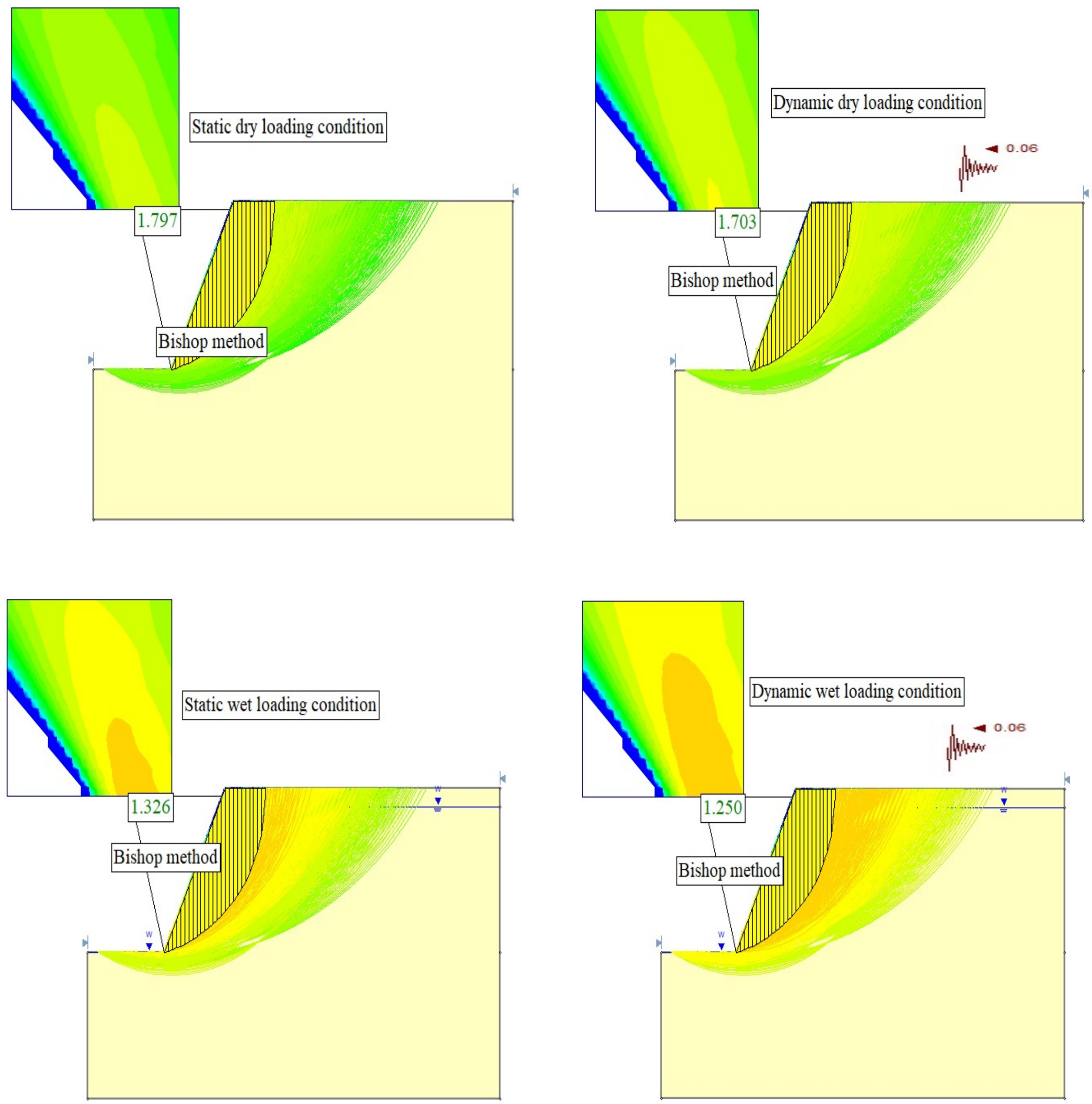


Figure A.12 Slope model and output from slide software (RCSS2).

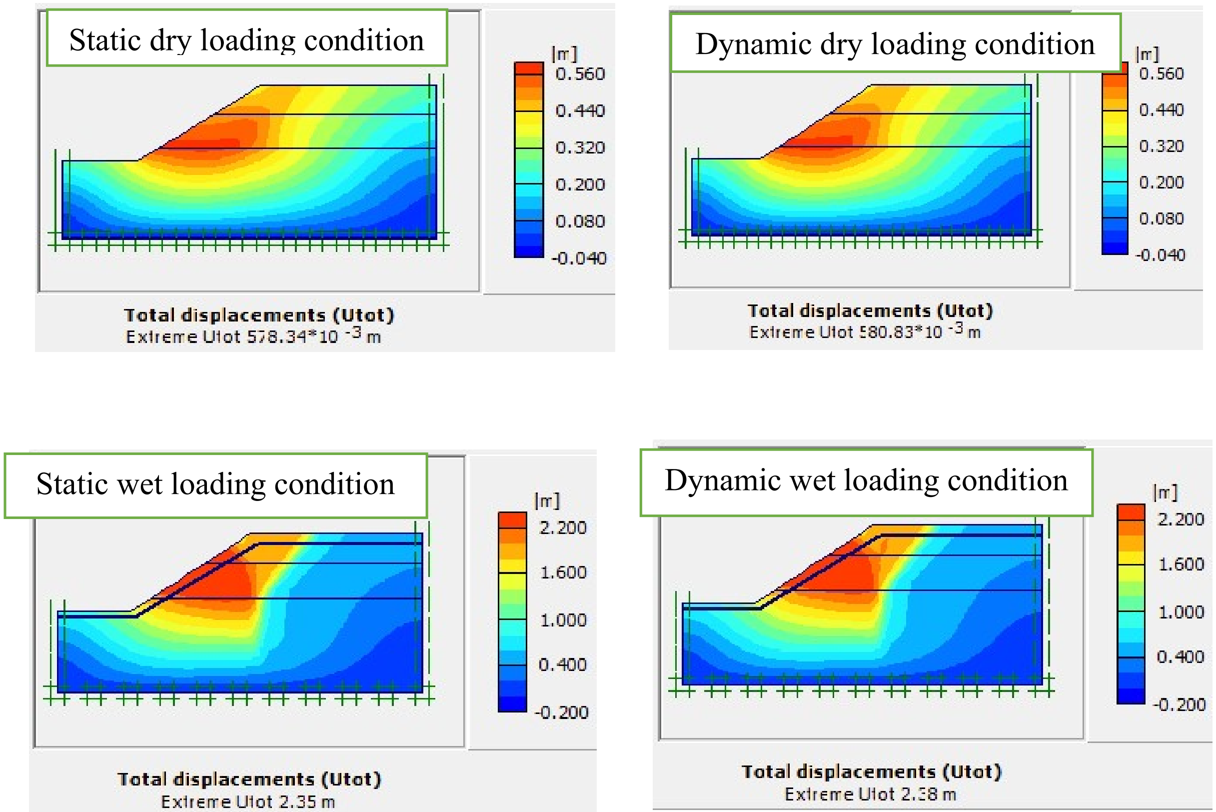
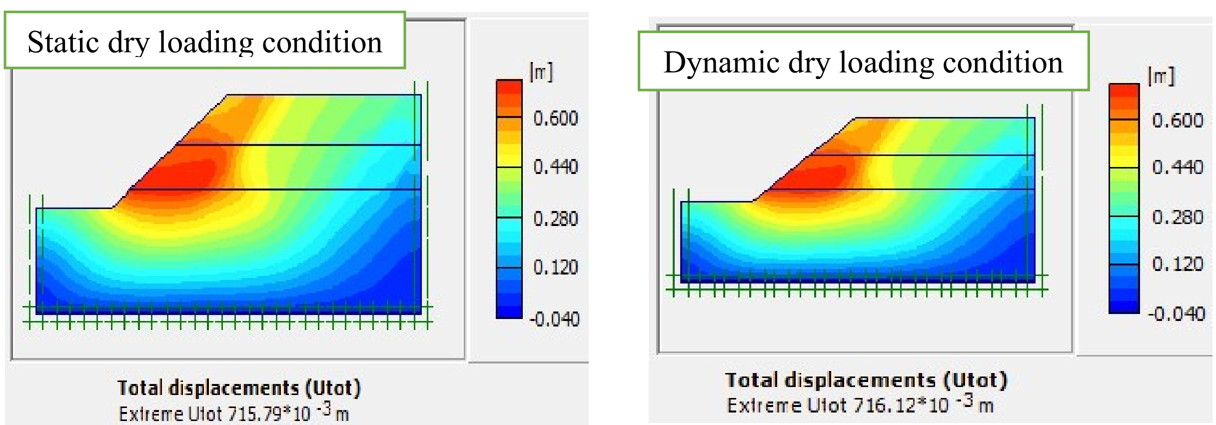


Figure A.13 Slope model and output from plaxis 2d (SCSS1).



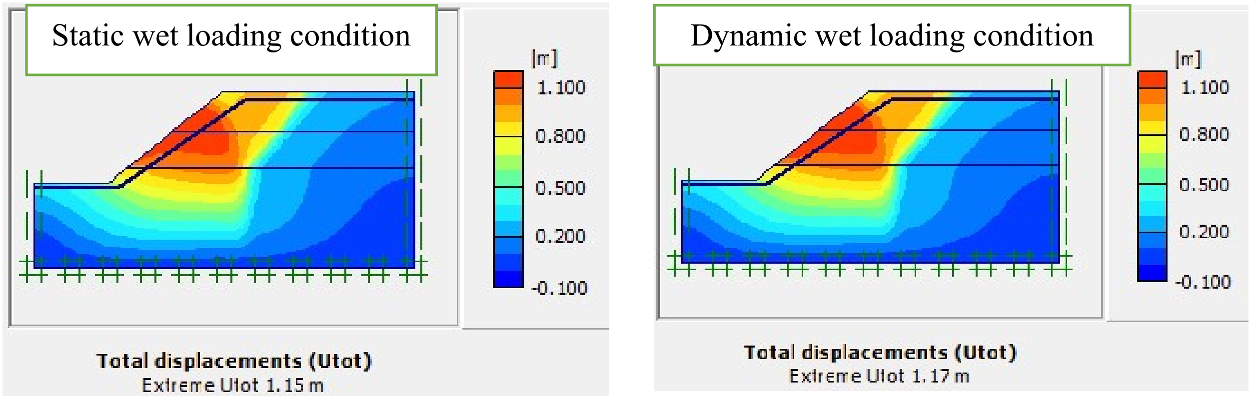


Figure A.14 Slope model and output from plaxis 2d (SCSS2).

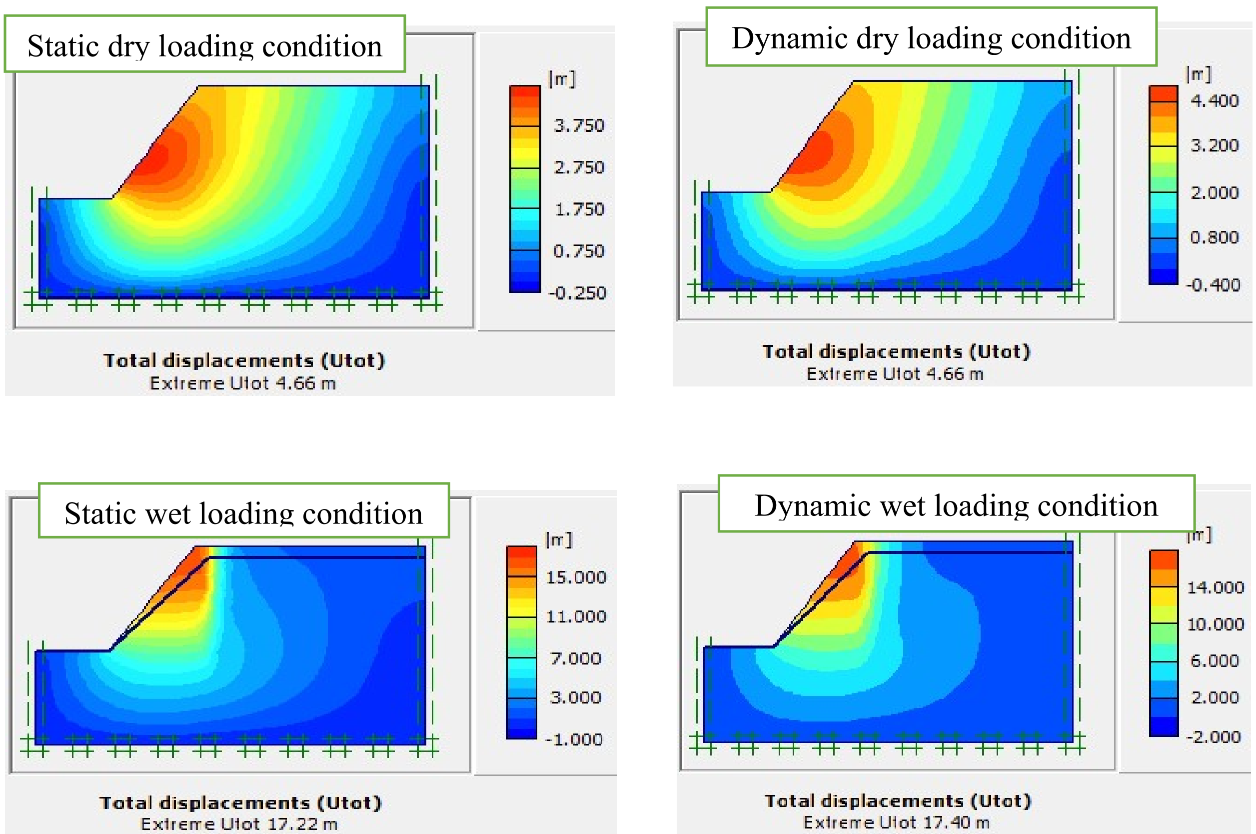


Figure A.15 Slope model and output from plaxis 2d (RCSS1).

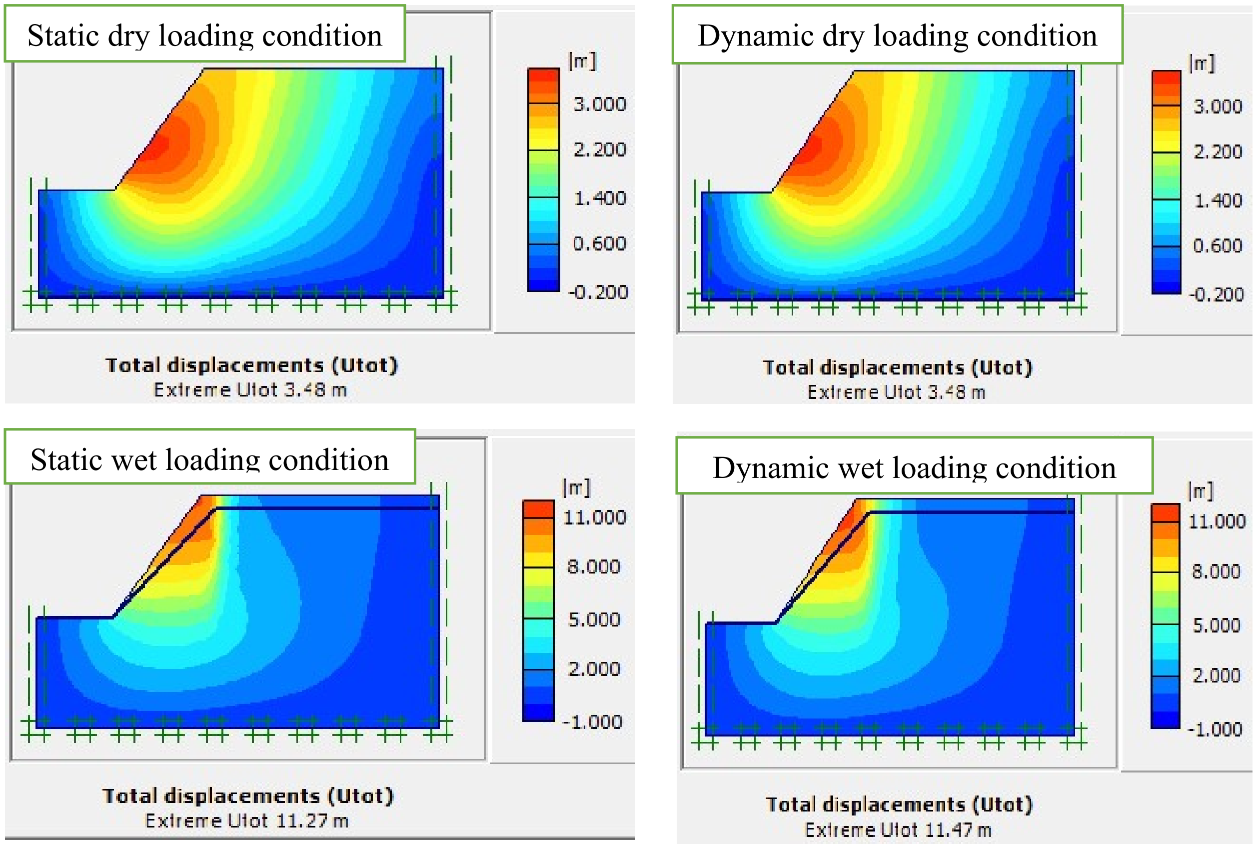


Figure A.16 Slope model and output from plaxis 2d (RCSS2).