

**SLOPE STABILITY ANALYSIS ALONG THE CRITICAL SECTIONS
OF WOLISO TO WONCHI LAKE ROAD, CENTRAL ETHIOPIA**



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Declaration

I hereby declare that this Master Thesis entitled “*Slope Stability Analysis along the Critical Sections of Woliso to Wonchi Lake Road, Central Ethiopia*” 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.

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LIST OF ACRONYMS

ABL	Ambo-Butajira Lineament
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing Materials Standard
DL	Didesa Lineament
FEM	Finite element method
FOS	Factor of safety
ISRM	International society of rock mechanics
JCS	Joint wall compressive strength
JRC	Joint roughness coefficient
Jv	Volumetric joint count
LL	Liquid Limit
PGA	peak ground acceleration
PI	Plasticity Index
PL	Plastic Limit
LEM	Limit equilibrium method
RMR	Rock mass rating
RQD	Rock quality designation
SMR	Slope mass rating
SRF	Stress reduction factor
YTVL	Yerer Tullu Welel volcanic lineament
USCS	Unified soil classification system
UCS	Uniaxial compressive strength

ABSTRACT

Slope instabilities are a major geological hazard that frequently leads to the failure of various civil engineering structures such as roads and open-pit mines. The road which connects Woliso town to one of the main tourist attraction in Ethiopia (Wonchi Lake) is also frequently affected by slope instabilities. Hence, this study is aimed at identifying and analyzing critical slope sections along this road by using Kinematic, Limit Equilibrium Method (LEM) and Finite Element Method (FEM). The input parameters for the analysis were determined through field investigation methods such discontinuity surveying, in-situ Schmidt hammer test, and sampling and via laboratory testing of samples. From field investigation, three critical structural controlled and six critical non-structural controlled slope sections were identified. Schmidt hammer rebound test conducted on the study area revealed that Uniaxial Compressive Strength (UCS) value ranges from 42.17MPa to 136.49MPa and 16.7 MPa to 28.9 MPa for rocks of structural and non-structural controlled slope sections, respectively. The stability analysis of structural controlled slope sections is first done via kinematic analysis and the results showed that RSS_1 is unstable for planar mode of failure whilst RSS_2 and RSS_3 became unstable for wedge mode of failure. Further stability analysis of planar mode of failure in terms of Factor of Safety (FOS) using LEM-based Rocplane 2.0 software revealed that RSS_1 is unstable under all anticipated conditions. Similar analysis for wedge mode of failure using Swedge 2.0 software also revealed that RSS_2 is unstable only under dynamic saturated condition while RSS_3 is unstable under all conditions due to intersection of JS1 and JS3, and stable under all conditions due intersection of JS2 and JS3. Similarly, stability analysis using both LEM-based Slide 6.0 and FEM-based Phase 2.0 software have also shown that all six non-structural controlled slope sections are unstable under saturated conditions. This is illustrated that rainfall/or saturation is the major factor contributing to slope failure in the study area. Moreover, the FOS obtained from LEM-based Slide 6.0 software and SRF obtained from FEM-based Phase 2.0 software are in a very close agreement which validated slope instability problems in the study area. Depending on the analysis of results, a total bolt capacity of 1960 and 4000 tons were designed for RSS_1 to stabilize planar failure due to JS1 and JS3 respectively. Similarly, rock bolts with a total capacity of 3750 ton and shotcrete with a shear strength of 1600 t/m² at RSS_2 , and rock bolt with a total capacity of 31200 ton and shotcrete with a shear strength of 2950 t/m² at RSS_3 were designed to stabilize unstable wedge. However, considering the budgets of bolts and the current stage of this road construction, removing unstable rock wedges is recommended. Moreover, for non-structural controlled sections, it is designed to reduce upper slope profile of SSS₂ to less than 23.19° and the middle slope profile to less than 33.69°, and top slope profiles of SSS₄ and SSS₅ to an angle less than 63.43° and 27.22°, respectively. Moreover, surface and subsurface drainage systems are also recommended to in these slope sections for better control of water effects.

Key words: Slope stability, Kinematic analysis, Lemit equilibrium method, Finite element method.

CHAPTER ONE

INTRODUCTION

1.1 Background

On the global scale, slope instabilities are a challenging geological hazard that frequently leads to the failure of various civil engineering structures such as dams, open-pit mines, and roads (Hoek and Bray, 1981; Pantelidis, 2009). Slope failure and landslides are globally widely distributed in areas that are characterized by rugged topography with high rainfall and/or seismicity (Pantelidis, 2009). In Ethiopia, in particular, slope failure along road sections passing through rugged topography of northern, southern, and western Ethiopian plateau and rift margins has been identified as one of the most frequent geological hazards (Ayalew, 1999). These instabilities have caused loss of lives, failures of engineering structures (mainly roads), damage to agricultural lands, and the natural environment in the abovementioned parts of the country (Temesgen et al., 2001; Abebe et al., 2010; Woldearegay, 2013). For example, according to Woldearegay (2013), over 300 human lives were claimed, above 200 dwelling houses, more than 200 km of asphalt road, and over 500 ha of land have been damaged in the past decade alone in the aforementioned parts of the country. Hence, the construction of transport pathways in these areas requires proper slope modification. It is, therefore, imperative to know the potential instability zones by carrying out a thorough slope stability analysis prior to any construction activity so that possible mitigation measures can be designed and implemented to avoid any slope aggravation.

Slope stability analysis is mainly related to the relationship between resisting and driving force of the slope which is dependent on slope governing factors: internal and external factors (Hoek & Bray, 1981; Bell, 2007; Raghuvanshi, 2017). The main internal governing factors are; the geometry of the slope, potential failure plane characteristics, and shear strength parameters of the slope (Wang and Niu, 2009; Ayalew et al., 2004; Turrini and Visintainer, 1998; Anbalagan, 1992), whereas the external factors are rainfall, seismicity, the influence of surface and subsurface water and manmade activities (Girma et al., 2015; Raghuvanshi et al., 2014; Wang and Niu, 2009; Dahal et al., 2006; Gorsevski et al., 2006; Malamud et al., 2004; Bommer and Rodri'guez, 2002; Keefer, 2000). These factors in combination are responsible for defining the stability condition of the slope.

Slope instability usually takes a different form/or mode depending on the geology. In rock slopes plane, wedge, circular, toppling and rock fall are common modes of failure whereas

failures solely follow a circular path in a soil slope. The plane failure in rock slope occurs when a structural discontinuity plane dips or daylight towards the valley at an angle smaller than the slope face angle and greater than the angle of friction of the discontinuity surface (Kovari and Fritz, 1984). Similarly, wedge failure occurs when the intersection of two discontinuities daylight on the slope face and it dips smaller than the slope face (Hocking, 1976; Hoek and Bray, 1981) while toppling and rock-fall mostly occur in steeply dipping slopes having vertical/or sub-vertical discontinuities (Hoek and Bray, 1981). A circular mode of failure in rocks occurs when rocks constituting the slope are highly weathered and intensively fractured that failure/s cannot be controlled discontinuities (Singh, 1992; Wyllie and Norrish, 1996).

Different methods were used by different researchers to assess and analyze the slope stability of the slopes. For example, Bieniawski's (1989) Rock Mass Rating (RMR) and Romana's (1985) Slope Mass Rating (SMR) have been used to analyze the stability of the slope. Moreover, in recent years, Kinematic, Limit Equilibrium Method (LEM), and Finite Element Method (FEM) are frequently being used for analyzing different modes of slope failures (Wyllie and Mah, 2004; Raghuvanshi, 2017). The kinematic method deals with the failure mode of the rock slope without reference to the forces that cause them to move (Goodman, 1989; Hudson & Harrison, 1997; Park and West, 2001). Limit Equilibrium Method (LEM) determines the stability of the slope by evaluating the Factor of Safety (FOS) which is the ratio of resisting force to driving force along the failure plane (Duncan, 2000; Chowdhury et al., 2009). On the other hand, the Finite Element Method (FEM) determines the stability of the slope by computing the Stress Reduction Factor (SRF) based on the relationship between stress and strain components of the slope (Eberhardt et al., 2004). Hence, this study utilized Kinematic, LEM, and FEM to analyze the stability of the slope and to forward appropriate remedial measures.

Generally, this research work is conducted to identify, locate and analyze slope instability problems along the road from Woliso to Wonchi Lake in the South West Showa Zone of Oromia Regional State. Considering the recent advancement in slope stability analysis techniques, Kinematic, Limit Equilibrium Method (LEM), and Finite Element Method (FEM) is used to achieve the mentioned objective.

1.2 Statement of the problem

This study intends to undertake slope stability analysis along Woliso to Wonchi Crater Lake. Woliso to Wonchi Town asphalt road is currently under-construction as part of the Ambo-Woliso Road Project while the construction of asphalt road connecting Wochi Town to Wonchi Crater Lake is recently started as part of (Dine for Ethiopia Project on Wonchi Lake) by the prime minister's office. The Wonchi Crater Lake is a good-looking heritage of a once-powerful volcano that erupted in the tertiary period of the Cenozoic era. The erupted volcano left a systematic caldera of rock which is now covered in lush green vegetation. The unique topography of the lake, waterfalls, beautiful valleys, farmlands, varieties of animal and plant species, aquatic birds, and indigenous trees like *Egynia Abyssinica* (Koso) are making the area more interesting for tourism. Hence, this lake is one of the major tourist destinations in Ethiopia.

Woliso to Wonchi Crater Lake road, however, passes through very rugged topography which is highly susceptible to slope instability problems (Figure 1.1 C). In addition, the study area is characterized by high rainfall (i.e. mean annual rainfall of around 1140 mm) which plays a great role in triggering slope instability problems in the study area. Moreover, the incorporation of the study area in the margin of the Main Ethiopian Rift (MER) and within Yerer Tulu Welel Volcanic Lineament (YTVL) also further aggravates this potential instability problem concerning seismicity in the study area. In addition, a field reconnaissance survey has shown that some slope cuts for road construction along Woliso to Wonchi Town are highly vulnerable to instability (Figure 1.2 A and B) while some of them have hindered traffic activities by damaging the road in the past. Similarly, the road from Wonchi Town to Wonchi Crater Lake (which was a gravel road) has been frequently affected by slope instability before its current construction began. However, no research has been conducted to identify, locate and analyze slope instability problems along Woliso to Wonchi Crater Lake road which plays a major role in increasing tourist attraction of this lake. Hence, in this research work, an attempt will be made to determine the geotechnical properties of rocks and soils along critical slope sections of Woliso to Wonchi Lake road and analyze their stability using LEM and FEMs.

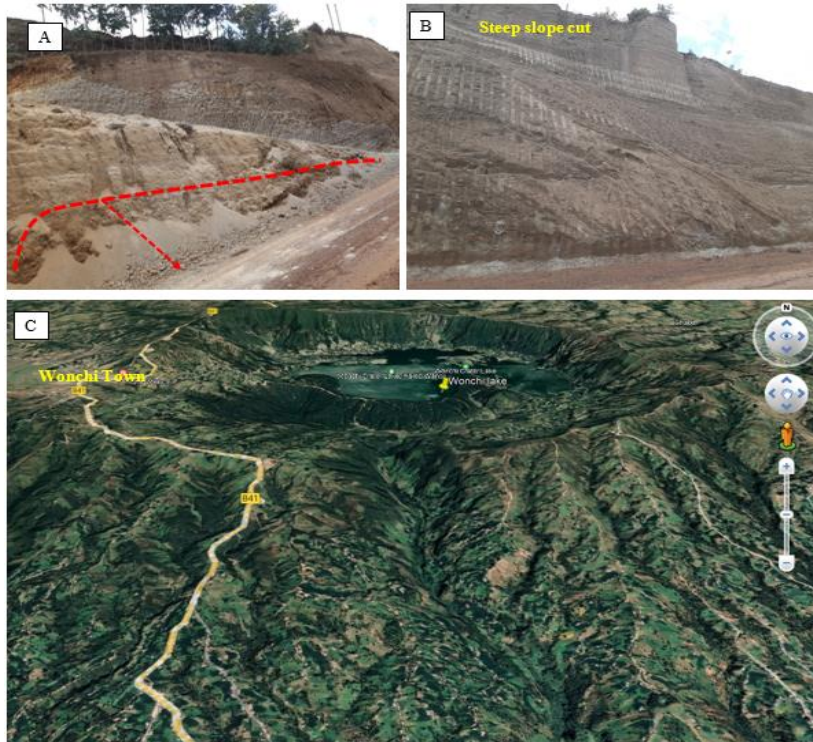


Figure 1. 1A) Falling slope mass and B) steep slope cut along Woliso to Wonchi Lake road, and C) google earth image of part of the study area showing rugged topography.

1.3 Significance of the study

The discoveries of this study can be considered as retribution for slope stability problems in road sections, it can be used as input data for the proposed construction road section. This will play a great role in assuring the stability of the road for a long period. This study also provides a suitable recommendation that will help to minimize slope stability problems that are repeatedly occurring on a road in Ethiopia.

1.4 Location and Accessibility

The study area, the road which connects Woliso to Wonchi Lake, is located in Southwest Shoa Zone, Oromia Regional State, and Central Ethiopia (Figure 1.2). It is incorporated in the North-Western part of the Ethiopian Plateau. This area is situated at around 154 km from the capital, Addis Ababa, in the western direction. It is bounded by geographic coordinates of 372753 to 390434 UTM E and 946115 to 973772 UTM N in the UTM Zone of 37. Topographically, it is characterized by flat to rugged topography with altitudes ranging from 2048 to 3450 m a.s.l. It is usually accessible via a 111 km Addis Ababa to Woliso asphalt road and then through a 43 km asphalt road that connects Woliso to the lake which is currently under construction.

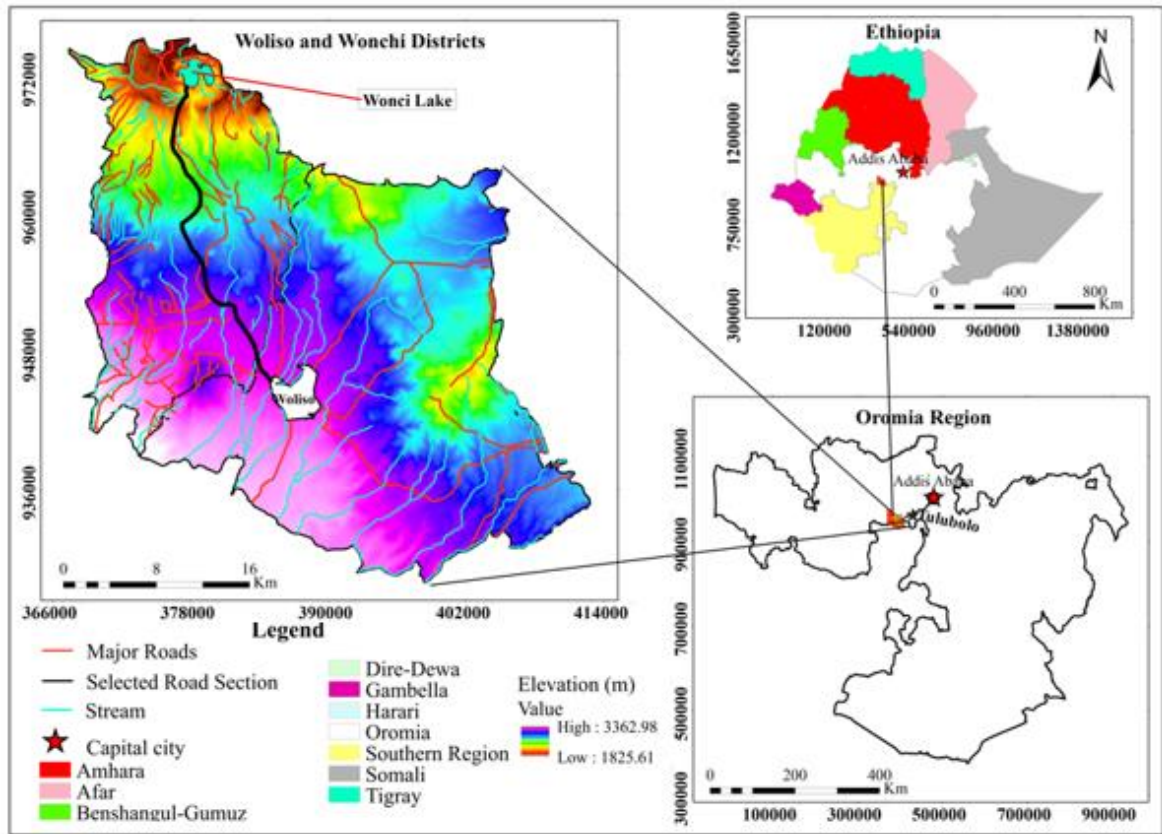


Figure 1. 2 Location map of the study area

1.5 Physiography and Topography

The physiography of the study area consists of numerous highly rugged topography such as Wonchi and Dandi Volcanic centers. The Woliso to Wonchi lake road in particular starts with relatively flat topography followed by an extremely rugged and steep escarpment near Wonchi Lake. Incised river valleys, deep gorges, and rolling plains are common characteristic features near Wonchi Lake. The altitude of the study area varies from 2300 m around Woliso town to 3300 m around Wonchi Lake where the failure is quite common (Figure 1.3).

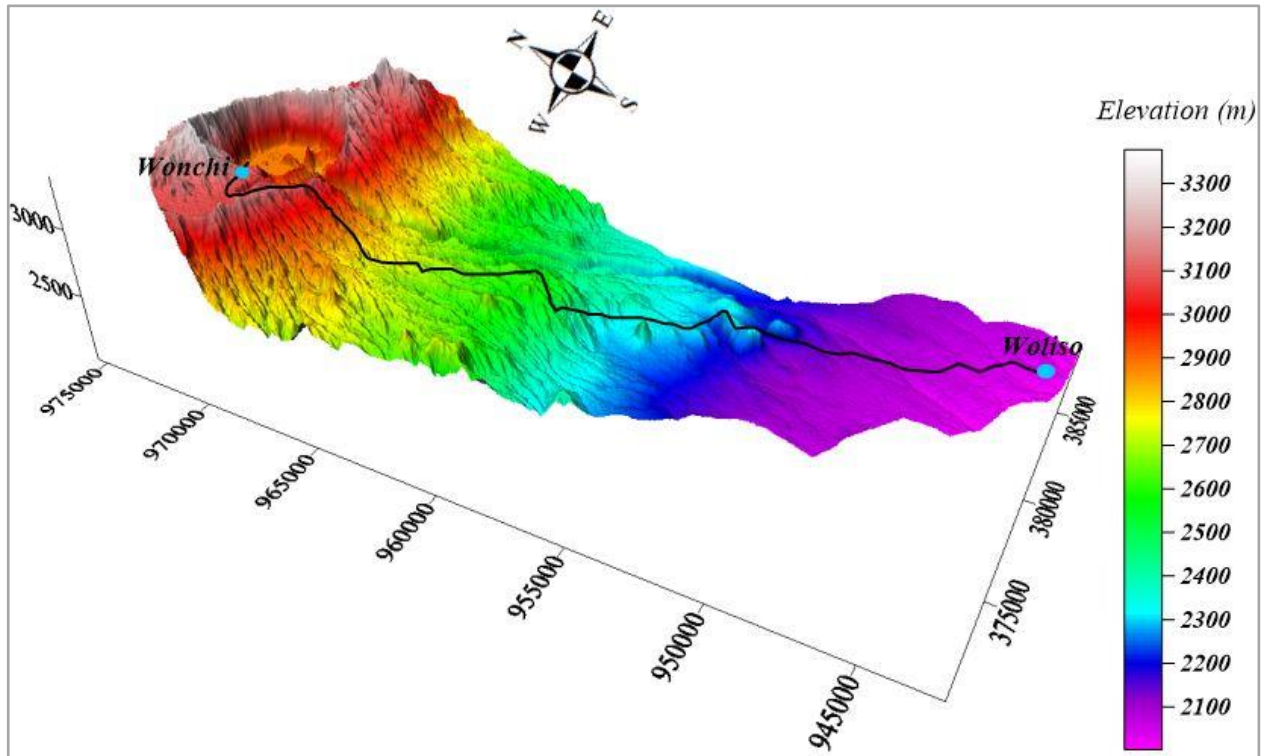


Figure 1. 3 Physiographic map of the study area.

1.6 Rainfall of the Study Area

Rainfall is one of the major triggering factor for both rock and soil slope failures (Raghuvanshi et al., 2014; Ermias et al., 2017) as it percolates into the upper slope through openings and apply pore pressure (Raghuvanshi et al., 2017). In the study area, this is also supported by the fact that most slope failures take place during the rainy season. The rainfall data of the study area collected from nearby stations such as Woliso, Busa, and Tulubolo stations showed that the rainy season of the study area starts in May and ends in September (Figure 1.4). The maximum mean monthly rainfall recorded at Woliso, Tulubolo, and Busa stations is 267.8 mm, 284.6 mm, and 378.4 mm, respectively. The mean annual of rainfall of the study area as determined from these three stations is also in the order of 1173.75 mm. In general, field observation and interviews of local people revealed that slope failure of the study mainly occurs during the months of July and August.

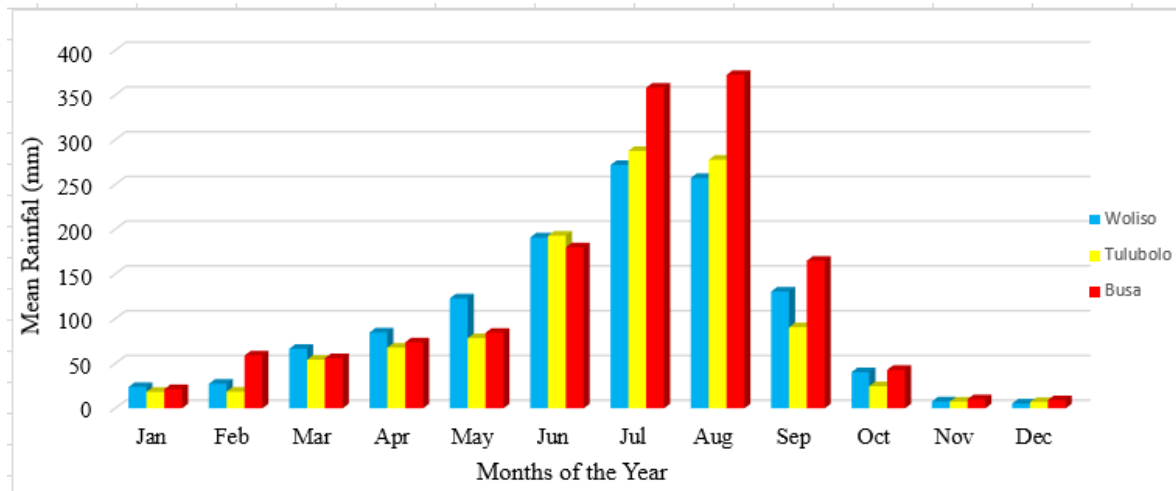


Figure 1. 4Monthly rainfall data of the study area from 2003-2017 (National Meteorological Service Agency, 2022).

1.7 The objective of the Research

1.7.1 General Objective

The main objective of this study is to conduct the slope stability analysis for the critical slope section along with Woliso town to Wonchi Lake road.

1.7.2 Specific objectives

- ✚ To identify the critical slope instability sections along selected road sections.
- ✚ To determine the geological and geotechnical properties of soils and rock of the critical slope sections.
- ✚ To carry out the kinematic analysis, a factor of safety and/or stress reduction factor for critical slope sections.
- ✚ To forward constructive remedial measures for the recognized problems.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

The consequence of slope failure leads to billions of dollars in infrastructure and property damages in the world each year (yang and Chen, 2010). The hillside and road cuttings of most parts of the north, south, and western regions of the Ethiopian Plateau and rift margins have a record of instability in both the superficial materials and the bedrock. Although landslides have been observed for at least the last three decades, in more recent times the sensitivity of both the public and administrative bodies to these events has increased as more and more people are now living in the areas affected (Ayalew,1999). Active slopes are found in almost all parts of the country and there is now a considerable concern for the safety of the population in these regions as well as the buildings and some infrastructures, such as the communication facilities. Almost 60% of the total population in Ethiopia lives in the highlands with an altitude of more than 1750 m (Woldearegay, 2014).

2.2 Controlling Factors of Slope Stability

There is always a factor that initiates and triggers the occurrence of a slope failure problem. The factor which decreases the shear strength of the slope material and increases the shear stress on the slope body can be nominated as the governing factor for the slope instability. The angle of the slope and the strength of the materials on it are the eventual determining factors for slope stability. Force of gravity pulls a block of rock situated on a rock slope toward the earth's center (vertically down). Relative to the slope, the vertical gravitational force can be split into two components: one which pushes the block downslope which is called shear force, and the other which pushes into the slope which is called the normal force.

Heavy precipitation, weathering conditions, high relative reliefs, seismic disturbance, and fragile geology with increased man-made activities such as roads on mountains have increased slope stability problems throughout the world. All these mentioned factors can be categorized into intrinsic and external triggering factors of slope instability. The intrinsic factors are predominantly; geological factors (lithology or soil type, structural discontinuity characteristics, shear strength of the slope material), the geometry of the slope (slope inclination, aspect, elevation, and curvature), and land use/land cover (Wang and Niu, 2009; Rahuvanshi et al., 2014; Hamza & Raghuvanshi, 2017; Lamessa and Meten, 2021). The

external factors which generally trigger slope failures are rainfall, seismicity, the influence of surface and subsurface water, and manmade activities such as; construction activities and cultivation practices in mountainous regions (Collison et al., 2000; Keefer, 2000; Bommer and Rodriguez, 2002; Dai, et al., 2002; Dahal et al., 2006; Wang and Niu, 2009; Raghuvanshi et al., 2014). Each type of governing factor is discussed as follows.

2.2.1 Intrinsic Factors

2.2.1.1 Geometry of Slope

Slope inclination, aspect, elevation, and curvature can be considered slope geometry parameters (Hamza & Raghuvanshi, 2017). Relative relief and slope morphometry of the slope are the two important parameters that slope geometry includes (Raghuvanshi et al., 2014). Relative relief is defined as the difference between the maximum and minimum elevation within feet. The steepness of the slope can be described by slope morphometry. According to Willy and Mah, (2004), slope instability problems continually occur along steep slope sections.

2.2.1.2 Material of Slope and Shear Strength Parameters of the Material

Soil can be the main constituent of a slope in one area and rock on the other or both of them can be the main constituents of a slope in one area. Stability analysis is mainly based on strength of these materials and their degree of weathering. The erosion of rocks mainly depends on strength of rocks. In the case of soil, the strength is influenced by genetic class and soil cover (Anbalagan, 1992; Raghuvanshi et al., 2014).

According to (Barton and Bandis, 1990; Raghuvanshi, 2017) shear strength parameters of materials that compose the slope are the principal factors which determine resisting force acting normal to the failure plane. The conditions of discontinuities such as roughness, continuity, aperture, and infilling materials are the criterion on which shear strength parameters depends on (Johnson and Dgraff, 1991). Barton and Bands, (1990) specifically determine shear strength along failure plane or natural joints using the empirical law of basic friction angle which is first proposed by (Barton, 1973) by using parameters like joint roughness coefficient (JRC), joint wall compressive strength (JCS), effective normal stress, peak shear strength, and basic friction angle. The nonlinear equation is written as follows.

$$\tau = \sigma n \tan [JRC \log_{10} (JCS / \sigma n) + \phi b] \quad 2.1$$

Where, τ = Peak shear strength (Kpa), JRC=joint roughness coefficient, σ_n =Effective normal stress (Kpa), JCS=joint wall compressive strength (Kpa), ϕ_b =basic friction angle.

Joint roughness coefficient (JRC)

JRC is a very important parameter to a discontinuity (Singhe and Goel, 1999) due to its ability to decline as aperture filling thickness and the degree of any permeable displacement increase. It is the estimated coefficient of joint surface roughness that ranges from smoothest (0) to roughest (20) (Barton & Choubey, 1977). According to Morelli (2014), JRC can be estimated by numerous methods, and the most commonly used rapid subjective one is by comparing discontinuity surface roughness profile visually which is developed by (Barton and Choubey, 1977). The appearance of the discontinuity surface is compared visually with the profiles shown and the JRC value corresponding to the profile which most closely matches that of the discontinuity surface is chosen. Additionally, it can be determined from the relationship between the length of the profile and maximum amplitude of the profile or through back analysis and tilt test of rock joints (Barton and choubey, 1977; Barton and Bands, 1982).

Joint wall compressive strength (JCS)

Barton and Choubey (1977) stated that JCS of discontinuity is a very essential component of shear strength. The wall strength JCS has an inverse relationship with aperture or filling thickness or the degree of the previous displacement.

In the field, the appropriate value of JCS can be measured by performing a Schmidt hammer test which is applied on a weathered rock surface with unit weight (γ). If the joint is un-weathered joint wall compressive strength (JCS) is equal to the unconfined compressive strength of un-weathered rock (Barton, 1976; Barton and Choubey, 1977, Barton and Bands, 1990). However, when there is no Schmidt hammer reading for weathered rock JCS will be considered as the friction of uniaxial compressive strength of the rock (Barton, 1976).

Basic friction angle (ϕ_b)

According to Goodman (1989), a basic friction angle is an angle with flat, planar, and un-weathered rock surfaces in which its value ranges from 21° to 40° for most of the rock. Barton (1973) also stated that, for most smooth, planar, and un-weathered rock surfaces, its value ranges from 25° to 35° . It can be determined from a tilt test on smooth, planar and un-weathered joints cut from rock surface by a saw or adopted from a basic friction angle determined by (Coulson, 1972; Barton and Choubey, 1977). For variably weathered joints,

basic friction angle can be correlated with residual friction angle as stated by Barton and Choubey (1977).

$$\phi_r = (\phi_b - 20) + 20 (r/R) \quad 2.2$$

Where r is the Schmidt rebound number of wet and weathered fracture surfaces and R is the Schmidt rebound number on dry un-weathered surfaces.

2.2.1.3 Structural Discontinuities

The rock mass can contain primary and secondary structures like bedding planes, faults, joints, foliations, folds, and other structural features which can have a significant role in controlling slope stability (Hoek and Bray, 1997; Raghuvanshi, 2014). Orientation, spacing, continuity, surface characteristics, separation of discontinuity surface and thickness, and nature of filling material within the discontinuity surfaces needs great attention in slope stability analysis of rock mass due to their influence on the stability (Johnson and DeGraff, 1991; Price, 2009). The stability condition of rock mass is highly affected by the orientation of the discontinuity plane. On account of the rock mass, may fail along one or more discontinuity planes, the discontinuity condition which makes the rock mass unstable require great attention and study to keep the rock mass safe. The following two conditions which make the slope kinematically unstable have to be taken into consideration for the stability of rock mass (Hoek and Bray, 1981; Johnson and DeGraff, 1991; Hoek and Bray, 1997; Yoon et al., 2002).

- a) The extent of parallelism between the directions of the discontinuity, or the line of intersection of two discontinuities and the slope face.
- b) Discontinuity plane or plunge of a line of intersection of two wedge forming planes daylight the slope at less than the slope angle.

The spacing of discontinuity is another factor that affects the stability of rock mass. The rock mass will be more susceptible to instability if the discontinuity spacing is less (Johnson and DeGraff, 1991). The discontinuity planes' continuity influences the stability of the rock mass provided the orientation of discontinuity is kinematically critical. The discontinuity plane with more continuity will show more unstable conditions than the discontinuity with less continuity. Johnson and Degraff (1991) state that surface characteristics of discontinuities are also vital in determining the stability condition of a rock mass. The waviness/ undulation of the surface, the smaller scale roughness of the surface, and the physical properties of the infilled material between the two surfaces of the discontinuity plane are the three factors

concluded in the consideration of the surface characteristics of discontinuities (Raghuvanshi et al., 2014).

2.2.1.4 Land Use and Land Cover

Land use and land cover have a great influence on the stability condition of hill slopes (Raghuvanshi et al., 2014). Thick vegetation cover contributes to slope stability because less runoff will be expected in such type of area (Arora, 2008). Besides, the root of plants can increase the shear strength of soil mass by holding the soil mass together (Turrini and Visintainer, 1998). Surface erosion in the form of gullies is also checked with thick vegetation cover. Barren areas and sparsely vegetated land are more prone to weathering and erosion and this will facilitate the washing of soil and rock material with water.

2.2.2 External Factors

2.2.2.1 Seismicity

The second external triggering factor for slope failure is seismic activity. Ground acceleration can be constructed from seismicity which can lead to slope failure (Keefer, 2000; Parise and Jibson, 2000; Bommer and Rodri'guez, 2002). Slope areas that might stay stable under the static load of overlying material can fail under dynamic loads due to seismic activity (Hoek and Bray, 1981). The minimum amount of earthquake about 4-5MM can cause slope failure (Pantelidis, 2009). When the slopes of discontinuous rock mass are faced to ground acceleration, it results in extending or opening of structural discontinuities. Consequently, it leads to a decrease in shear strength of structural discontinuities and facilitates instability of slopes (Raghuvanshi et al., 2014).

Slopes composed of unconsolidated soft sediments or surficial deposits having steep slopes, high seasonal groundwater levels, and shallow-rooted or sparse vegetation are also susceptible to failure under seismic loading (Raghuvanshi et al., 2014).

2.2.2.2The Influence of Surface and Subsurface Water

The surface flow of water on the upper slope can move through the discontinuities and other openings. The stability of the slope can be reduced by the pore water pressure developed by the water in the discontinuities (Hossain, 2011; Ahmadi and Eslami, 2011; Sharma et al., 1995; Hoek and Bray, 1981). According to different researchers, most slope instability problems aggravate by heavy rainfall (Ayalew et al., 2004; Collison et al., 2000; Dai and Lee, 2002; Dahal et al., 2006). It is noticeable that slope stability increases in the rainy season

(Ermias et al., 2017; Raghuvanshi et al., 2017, 2014). Rainfall can operate as a recharge mechanism for groundwater and in general, saturates the slope (Arora, 1997; Hoek and Bray, 1981; Wyllie and Mah, 2004). The water along the potential failure surface will reduce the shear strength and also lubricate the surface which may facilitate the sliding process (Raghuvanshi et al., 2017; Hack, 2002). Therefore, it is very important to consider surface drainage conditions while assessing the stability of the slope.

2.2.2.3 Human Activity

In addition to natural causative factors, manmade activities can also contribute to the possibility of slope instability (Wang and Niu, 2009). In building construction, cutting slopes steeply and removing the toe which is supported under will create an opportunity for the slope to fail easily. The excavated loose material dumped under the slope in an unorganized manner will destabilize the slope during the rainy season. Poor irrigation practice, making hill slopes flat, and cutting into terraced land for cultivation purposes will also provoke slope instability (Raghuvanshi et al., 2014).

2.3 Types of rock slope failures

According to Singh and Goel (1999), slope failure types can be classified based on the mode of failure. Types of slope failures are discussed as follows.

2.3.1 Planar (translational failure)

A failure to be planar (translational) failure has to take place through widespread and/or incessant joints dipping towards the slope with a strike nearly parallel ($\pm 20^\circ$) to the slope face and dips into the excavation at an angle greater than the angle of friction and less than the inclination of the slope (Hoek and Bray, 1981; Singh and Goel, 1999). Generally, the planar mode of failure depends on joint continuity.

2.3.2 Wedge failure

Wedge failure occurs when the discontinuities of the strike of two joints of different sets obliquely intersect across the slope face and their line of intersection day-lights in the slope face (Hoek and Bray, 1981; Singh and Goel, 1999).

2.3.3 Circular (rotational) failure

The circular or rotational mode of failure takes place onwards a surface that just moderately grows along joints but mainly crosses them. The failure can only happen in both very weak

and heavily weathered rock and heavily jointed rock masses with a very small block size (Singh and Goel, 1999).

2.3.4 Toppling failure

Toppling failure appears in slopes where frequently arranged joint sets or bedding planes strike parallel, or nearly parallel to the slope face and dip at a steep angle into the face (Goodman and Bray, 1976). It involves overturning of rock layers when the vector representing the weight of a block falls outside of the base, the block will topple; it will rotate about its lowest contact edge.

2.3.5 Ravelling Slopes (Falls)

When small parts of rock detached from the rock mass assembled at the foot of steep slopes and periodic expansion and contraction associated with freezing and thawing of water in cracks and fissures in the rock mass are the primary causes of falls (Singh, 1999).

2.4 Slope Stability Analysis Approaches

The main objective of slope stability analysis is to evaluate the safe design of man-made or natural slopes and to forward the effective remedial measure in order to control loss of life and property which is followed by slope failure (Salunkhe et al., 2017). Therefore, engineering geologists have to understand in detail all methods for slope instability analysis for examining the stability of slopes.

The empirical method, kinematic analysis, limit equilibrium methods (LEM), and numerical method of modeling are the common and mostly used slope stability analysis approaches (Abramson, et al., 2002; Raghuvanshi, 2017; Salunkhe et al., 2017).

2.4.1 Empirical Method

2.4.1.1 Rock Mass Rating (RMR)

Rock mass rating is also called geomechanics classification. It was firstly released by Bieniawski (1973) for the application of tunnels, dams, and excavation (Kaiser et al., 1986). Its application was developed for the slope stability study after years Bieniawski (1979).

Separation of the site according to a considerable geological structural unit is important for determining the RMR of each type of rock mass represented by a separate geological structural unit. Uniaxial compressive strength of rock material (UCS), rock quality designation (RQD), discontinuity spacing, joint condition, groundwater condition, and joint

orientation are the parameters that need to be determined for each structural unit in the task of RMR calculation (Singh & Goel, 1999).

Among these basic parameters, joint spacing can be measured by using a tape meter in the field, from the joint condition parameters aperture and persistence can also measure by using a tape meter, and infilling material can also be obtained by visual observation. The groundwater condition can be determined using visual observation.

Intact rock strength can be determined either from the uniaxial compressive strength of rock or the point load strength index. Uniaxial compressive strength of rocks (UCS) can be determined according to Baroch and Franklin (1972) empirical relation.

$$UCS = 24 \cdot I_{s(50)} \quad 2.3$$

Where $I_{s(50)}$ is corrected point load strength

Dincer et al, (2004) forwarded the empirical equation (eq. 2.3) for the calculation of UCS.

Correspondingly, rock quality designation (RQD) can be determined by using different equations according to different researchers. According to Deere et al. (1967), rock quality designation (RQD) is the percentage of intact rock pieces greater than 10cm in the total core length (eq. 2.4). However, this equation is only applicable if the core sample is available but not used for surface exposure of rock quality estimation. Palmstrom (1982) developed the volumetric joint method for the estimation of RQD of surface exposure (eq. 2.5). Palmstrom (2005) also modified the previous equation of RQD and J_v (eq. 2.6)

$$RQD = \frac{\sum \text{Length of Core Pieces} > 10\text{cm}}{\text{The total length of the Core}} \times 100\% \quad 2.4$$

$$RQD = 115 - 3.3 J_v \quad 2.5$$

Where J_v is Volumetric Joint count is given by the following equation.

$$J_v = \left(\frac{1}{S_1}\right) + \left(\frac{1}{S_2}\right) + \left(\frac{1}{S_3}\right) \dots \dots + \left(\frac{1}{S_n}\right) + \left(\frac{N_r}{5\sqrt{A}}\right) \quad 2.6$$

Where S_1 , S_2 , and S_3 are the average spacings for the joint sets, N_r is the number of random joints in the actual location and A is the area in m^2 .

$$RQD = 110 - 2.5 J_v \quad 2.7$$

2.4.1.2 Slope Mass Rating (SMR)

Slope mass rating (SMR) was proposed by (Romana, 1985) which was the extension of RMR after Beniaowski (1979), It was only used for a planar and toppling mode of failure before (Anbalagal et al., 1992) and modified to account for wedge mode of failure. It was calculated from basic RMR by adding adjustment factors derived from joint slope relationships and factors depending on the method of excavation. Finally, stability class and probability of failure of rock slope can be indicated by using the following equation.

$$SMR = RMR_{basic} + (F1 * F2 * F3) + F4 \quad 2.8$$

Where, RMR_{basic} is rock mass rating after (Beniawski, 1989), F1- depends upon parallelism between joints and slopes face strikes. $(\alpha_j - \alpha_s)$ where, α_j is a joint strike and α_s is a slope strike. F2- refers to joint dip angle (β_j) or plunge of a line of intersection of two wedge forming planes (β_j). F3- depends upon the relationship between joint dip or plunge of a line of intersection of two wedge forming planes and slope inclination. $(\beta_j - \beta_s)$ or $(\beta_i - \beta_s)$ where, β_s is the inclination of slope. F4- is considering about method of excavation. It includes the natural slope or the cut slope excavated by pre-splitting, smooth blasting, normal blasting, poor blasting, and mechanical excavation.

2.4.2 Kinematic Analysis

The kinematic approach is the technique that is widely utilized to analyze the different modes of failures of rock slopes that occur due to unfavorable orientation of discontinuities such as joints, faults, bedding planes, foliation, and shear zones that can potentially serve as failure planes or plane of weaknesses (Hoek and Bray, 1981; Wyllie and Norrish, 1996; Wyllie and Mah, 2004).

The technique utilizes the orientation of discontinuity which is best defined by two parameters (dip and dip direction of discontinuity and slope face) for analysis and interpretation through stereographic projections that allow the three-dimensional orientation data to be represented and analyzed in two dimensions (Wyllie and Norrish, 1996). In-depth treatments of the principles of this technique for different modes of failure for plotting, analyzing, and interpreting data were given by (Markland, 1972; Goodman, 1989; Hock and Bray, 1981; Rockscience, 2004). Accordingly, the analysis can be done either manually using streonet or with dips software using friction angle, dip and dip directions of a slope, and discontinuities. The former involves the identification of failure mechanisms through the great circle on streonet and is very tedious when large orientation data are involved

(Richards and Atherton, 1987). The latter involves the utilization of dips software with friction angle, dip and dip directions of a slope, and discontinuities as input parameters. In such a way, the kinematic admissibility for each type of mode of failure will be carried out.

2.4.3 Limit Equilibrium method (LEM)

This type of approach is one of the most frequently used methods for the slope stability analysis of both rock and soil (Aprilia and Indrawan, 2014). This technique determines stability analysis in terms of the factor of safety by comparing the available shear strength along the sliding surface with the force required to maintain the slope in equilibrium (Wyllie and Norrish, 1996; Paradeshi et al., 2013). For rock slope stability analysis, once the mode of rock slope failure is identified, this technique determines the safety factor by correlating the force leading the rock mass to fail along the failure surface and the force resisting the rock mass to fail (Wyllie and Norrish, 1996; Aprilia and Indrawan, 2014). The modes of rock slope failures are usually identified through the aforementioned kinematic analysis (Wyllie and Norrish, 1996).

In a soil slope stability analysis, unlike rock slope failures there is no predefined failure surface and the failure tends to have a circular shape (Wyllie and Norrish, 1996; Wyllie and Mah, 2004). Accordingly, in this scenario safety factor corresponding to the circular mode of failure of soil slope can be determined using a ratio of shear strength available to resist sliding to shear stress driving the sliding. For stability analysis of slope, Abramson et al. (2002) and Krahn (2004) stated limit equilibrium techniques analyze stability through safety factors by dividing soil mass into different slices after which normal and shear inter-slice force and equilibrium can be determined from it.

Since the 1900s various limit equilibrium methods were developed for slope stability analysis (Aryal 2008; Wubalem, 2020). The stability of weak rock formation and soil which can be determined as having a rotational mode of failure can be analyzed by traditional limit equilibrium methods like Bishop's Method (BM), Janbu Generalized Method (JGM), Spencer's Method, and Morgenstern-Price method. Complex slopes also can be modeled by using modern limit equilibrium software like Geo slope (Slope \W) software.

2.4.3.1 Limit Equilibrium Method for Weak Rock and Soil Formation

2.4.3.1.1 Bishop's Method (BM)

Before the development of the general limit equilibrium method, the bishop method was mostly used in circular shear slip surfaces, but now it can be used for all types of slip surfaces

(Fredlund et al., 1981; Aryal, 2008; Nash, 1987; Wubalem, 2020). BM satisfied moment equilibrium, and vertical force and considers inter-slice normal force but it ignores the inter-slice shear force and horizontal force equilibrium (Fredlund et al., 1981; Aryal, 2008; Nash, 1987; Wubalem, 2020).

Generally, a satisfaction of moment of equilibrium for FOS, but force equilibrium satisfies only in the x-direction, and its performance in practical application makes it preferable even if it is mostly applicable for circular shear surfaces.

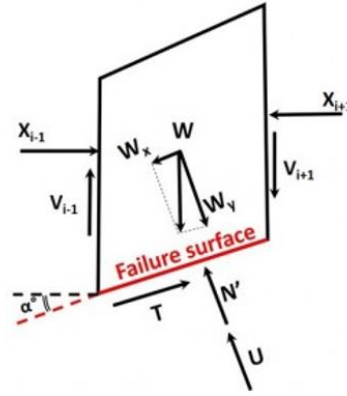


Figure2. 1 Forces acting on a single slice.

Where, W =weight of the soil above the failure surface X_{i-1} , X_{i+1} , V_{i-1} , V_{i+1} =interslice reactions from the adjacent slices, N' = normal effective, T = shear component, U = the boundary water force.

The shear force T acting at the base of each slice (Figure 2.4) can be calculated by using equation 2.9.

$$T = \frac{c * L + N' \tan(\phi)}{FOS} \quad 2.9$$

Where, L = the arc length of the slice's base which is approximated to a line segment, c = the cohesion and ϕ = the friction angle of the soil.

The equilibrium force in the vertical direction is also considered.

$$\Sigma F_{\text{vertical}} = 0 \longleftrightarrow (N' + U) * \cos(\alpha) + \frac{1}{FOS} * [(cL + N' \tan(\phi)) * \sin(\alpha)] - W = 0 \quad 2.10$$

Where, α is the angle of inclination.

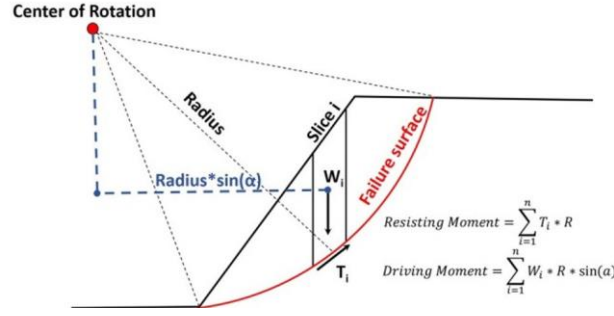


Figure2. 2Moments about the center of rotation generated by each slice.

The moment equilibrium around the center of rotation yields the following equation:

$$\sum T * R = \sum W * R \sin(\alpha) \quad 2.11$$

FOS can be easily calculated by dividing the resisting moment to the driving moment. The FOS for the entire failure surface can be computed by using equation 2.11.

$$FOS = \frac{\sum T_{max} * R}{\sum W * R * \sin(\alpha)} = \frac{\sum (c * L_i + N_i \tan(\phi))}{\sum W * \sin(\alpha)} \quad 2.12$$

Where R is the radius of the rotational surface.

Utilizing equation 2.10 and substituting the length of the slice's base as a function of its width and inclination ($L = B * \sin(\alpha)$), FOS can also be calculated by the following equation.

$$FOS = \sum \left\{ \frac{[(c * b + (W - \frac{U}{\sec(\alpha)}) * \tan(\phi)) * \sec(\alpha)]}{1 + [\frac{\tan(\alpha) \tan(\phi)}{FOS}]} \right\} \frac{1}{\sum W * \sin(\alpha)} \quad 2.13$$

It will become more effective utilizing the r_u coefficient to take into consideration of the water force in the calculation of the factor of safety.

$$FOS = \sum \left\{ \frac{[(c * B + W (1 - r_u) * \tan(\phi))] \sec(\alpha)}{1 + [\frac{\tan(\alpha) \tan(\phi)}{FOS}]} \right\} \frac{1}{\sum W * \sin(\alpha)} \quad 2.14$$

Where $r_u = U/\sigma$, U = pore water pressure, and σ = normal stress at the point of interest.

2.4.3.1.2 Janbu Generalized Method (JGM)

JGM imposes a stress distribution on the potential sliding mass by defining a line of thrust (Janbu, 1996). The inter-slice shear normal resultant is assumed to act where the line of thrust intersects the slices, which typically is lower than one-third of a point along the sides of the slices to represent a hydrostatic stress distribution (Abramson et al., 20002; Fredlund et al.,

1981; Aryal, 20008;). The assuming position of the resultant helps to compute the interslice shear forces by taking moments as the slice base center, the inter-slice base normal is first computed using Janbu's simplified method. The interslice shear forces are then determined by taking moments as the base of the slice [(Janbu, 1954; Abramson et al., 20002; Fredlund et al., 1981; Aryal, 20008; Wubalem, 2020). This method satisfies only the overall force equilibrium at the slice level. As stated in GeoStudio international Ltd (2004-2018), in slope/w, the JGM only can compute the force equilibrium factor of safety, but cannot compute the moment equilibrium factor of safety (Fm).

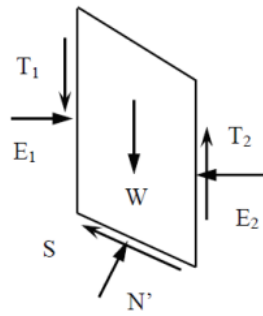


Figure2. 3Forces considered in JGM (Wubalem, 2020).

$$FOS = \frac{\sum(c' + (N - ul)\tan\phi')\sec\alpha}{\sum(W - (T_2 - T_1)\tan\alpha)} \quad 2.15$$

Where, $\Delta E = E_2 - E_1$ = net interslice normal forces (zero if there is no horizontal force), N = normal force, T = shear force, α = inclination of slip surface at the middle of slice c' and ϕ' = cohesion and friction angle respectively, W = weight of sliding mass, u = pore pressure, l = slice base length.

2.4.3.1.3 Spencer's Method

This method was proposed by Spencer (1968) depending on the methods of Fellenius (1936) and Bishop (1956). Spencer's method has been well known for coming up with rational and precise slope stability analysis of general arbitrarily shaped failure surfaces. Even if, different iterations are needed for gaining the exact value of the factor of safety in the assuring the complete equilibrium, the Spencer method satisfies both force equilibrium and moment of the sliding mass. PCSTABL5M computer program empowers to dig the most clarified Janbu method and, subsequently, analyze it with Spencer's method. The analysis by using the Spencer method forwards the following equation (eq. 2.16) to

calculate the factor of safety and the interslice force is assumed as parallel (Digvijay P. et al., 2017). This method requires a satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure.

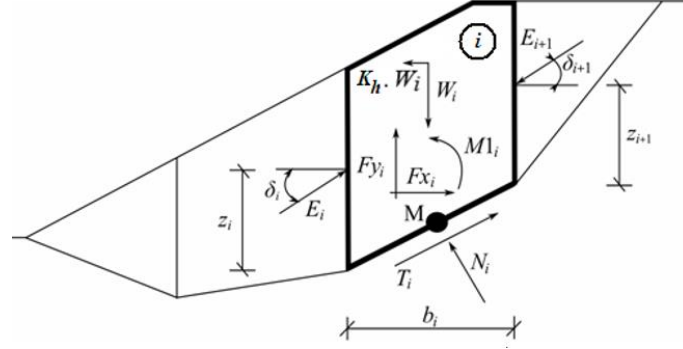


Figure 2.4 Static scheme- spencer method (Spencer, 1968).

Where, W_i = block weight, including material surcharge having the character of weight including the influence of the coefficient of vertical earthquake K_v , $K_h W_i$ = horizontal inertia force representing the effect of an earthquake, K_h = the factor of horizontal relation during the earthquake, N_i = normal force on the slip surface, T_i = shear force on the slip surface, E_i and E_{i+1} = forces extend by neighboring blocks (they are inclined from the horizontal plane by angle σ), Fx_i and Fy_i = other horizontal and vertical force acting on the block, $M1_i$ = moment of forces Fx_i and Fy_i rotating about point M (which is the center of the i^{th} segment of the slip surface), U_i = pore pressure resultant on the i^{th} segment of the slip surface.

The factor of safety can be calculated by using the following equations.

$$FOS = \frac{\text{Shear strength available}}{\text{Shear strength mobilized}} \quad 2.16$$

$$FOS = \tan \phi_i \tan (\sigma_{i+1} - \alpha_i) \quad 2.17$$

Where, ϕ_i = angle of internal friction of soil on the slip surface segment, α_i = inclination of the slip surface segment.

2.4.3.1.4 Morgenstern-Price Method

This method is a general method of slice developed based on limit equilibrium. Satisfaction of equilibrium of forces and moments acting on individual blocks is needed. Separation of the soil which overlays the slip surface by dividing planes creates the blocks. The figure below shows the forces which act on individual blocks.

According to Morgenstern and Price (1965) the interslice force inclination can vary with an arbitrary function ($f(x)$) as:

$$T = f(x)\lambda E \quad 2.18$$

Where, $f(x)$ = interslice force function that varies continuously along the slip surface, λ = scale factor of the assumed function.

For a given force function, the interslice forces are computed by iteration procedure until F_f is equal to F_m .

$$F_f = \frac{\sum (c'l + (N + ul) \tan \phi') \sec \alpha}{\sum (W - (T_2 - T_1)) \tan \alpha + \sum \Delta E} \quad 2.19$$

$$F_m = \frac{\sum (c'l + (N + ul) \tan \phi')}{\sum W \sin \alpha} \quad 2.20$$

2.4.3.2 Limit Equilibrium Method for Rock Slope

2.4.3.2.1 Stability Analyses of Plane Failure

Plane failure can occur along discontinuity when the failure of the slope is initiated by the presence of sole discontinuity or by the development of a single plan by serious discontinuities (figure 2.9).

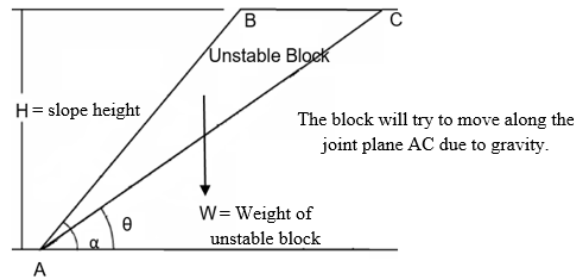


Figure2. 5 Geometry of a slope for plane failure (Wang & Ji, 2013).

The factor of safety can be calculated by using the definition of limit equilibrium as the ratio of shear strength to shear stress.

$$FOS = \frac{c + \sigma \tan \phi}{\tau} \quad 2.21$$

$$\text{Length of the discontinuity (AC)} = \frac{H}{\sin \theta} \quad 2.22$$

$$W = \frac{\gamma H^2}{2} (\cos \theta - \cot \alpha) \quad 2.23$$

$$\sigma = \frac{W \cos \theta}{A} \quad 2.24$$

Where, τ = Shear Stress (MPa), σ = Normal Stress (MPa), c = Cohesive strength of failure surface (MPa), ϕ = Friction angle of failure surface (degree) γ = Density of rock (KN/m³), γ_w = Density of water (KN/m³), θ = angle of inclination, A = Area of failure surface; $AC \times 1$ m² (unit thickness of slope is assumed)

Therefore factor of safety can be calculated by using the following equation.

$$FOS = \frac{c + \frac{W \cos \theta}{A} \tan \phi}{\frac{W \sin \theta}{A}} = \frac{CAW \cos \theta \tan \phi}{W \sin \theta} \quad 2.25$$

On the other case, the water can fill the discontinuity and contributes to decreasing the shear strength by exerting water pressure. The effect of water pressure can be estimated by determining the effective normal stress.

$$\sigma' = \sigma - \mu \quad 2.26$$

$$\mu = \frac{1}{4} \rho g h^2 \quad 2.27$$

Where, μ = water pressure (MPa), σ' = effective normal stress (MPa), σ = normal stress (MPa), ρ = density of water (kg/m³), h = height of water column (m)

If tension crack is present in the upper slope surface, the factor of safety can be calculated by using equation 2.26.

$$FOS = \frac{cA + w \cos \theta \tan \phi}{w \sin \theta} \quad 2.28$$

The installation of rock bolts has also an effect on the stability of the rock. A rock bolt installed at an angle β from the normal to the failure plane increases the normal stress by $T \cos \beta$ and reduces the driving force by $T \sin \beta$ where T is tension in the rock bolt. The factor of safety can be expressed as follows.

$$FOS = \frac{cA + (w \cos \theta - U - V \sin \theta + T \cos \beta) \tan \phi}{w \sin \theta + V \cos \theta - T \sin \beta} \quad 2.29$$

2.4.3.2.2 Stability Analyses of Wedge Failure

Wedge mode of failure can be analyzed either by considering only frictional resistance and neglecting the cohesion or by considering both cohesion and friction angle.

The assumption in the first case states that the sliding is resisted only by friction angle and the factor of safety is given by equation 2.28.

$$FOS = \frac{(R_A + R_B) \tan \phi}{W \sin \theta} \quad 2.30$$

Where, R_A and R_B are the normal reactions provided by planes A and B (figure 2.8), and the component of the weight acting down the line of intersection is $W \sin \theta$.

$$FOS_w = K FOS_p \quad 2.31$$

Where, FOS_w is the factor of safety of the wedge supported by friction only, and FOS_p is the factor of safety of a plane failure in which the sliding plane with friction angle ϕ , dips at the same angle as the line of intersection angle θ . The wedge factor (K) depends upon the included angle of the wedge and the angle of tilt of the wedge.

The analysis of wedge failure with cohesion and friction angle takes into account the dimensions and shape of the wedge, different cohesion and friction angles on each slide plane, water pressure, and several external forces. The stability of the wedge can be evaluated by resolving the forces acting along normal to the discontinuities and in the direction parallel to the line of intersection (Hoek and Bray, 1981).

$$FOS = \frac{3}{\gamma_r H} (C_a X + C_b Y) + (A - \frac{\gamma_w}{2\gamma_r} X) \tan \phi_a + (B - \frac{\gamma_w Y}{2\gamma_r}) \tan \phi_b \quad 2.32$$

Where, C_a and C_b are the cohesive strength of plane “a” and “b”, ϕ_a and ϕ_b are the angles of friction along with plane “a” and “b”, γ_r is the unit weight of the rock, and H is the total height of the wedge. X , Y , “A” and “B” are dimensionless factors, which depend upon the geometry of the wedge, Ψ_a and Ψ_b are the dips of planes “a” and “b”, whereas, Ψ_i is the plunge of the line of their intersection.

The water pressure is zero in fully drained condition and cohesion is zero the consideration of only frictional resistance. The factor safety in both conditions is stated in Equations 2.31 and 2.32 respectively.

$$FOS = \frac{3}{\gamma_r H} (C_a X + C_b Y) + A \tan \phi_a + B \tan \phi_b \quad 2.33$$

$$FOS = A \tan \phi_a + B \tan \phi_b \quad 2.34$$

2.4.3.2.3 Stability Analyses of Toppling Failure

In the LEM analysis, the following conditions have to be fulfilled for the existence of toppling failure.

- The h block must rest on an inclined surface.
- The joint sets need to be about 90° to the inclined surfaces.
- $\Theta < \phi$ and $\frac{t}{h} < \tan\beta$

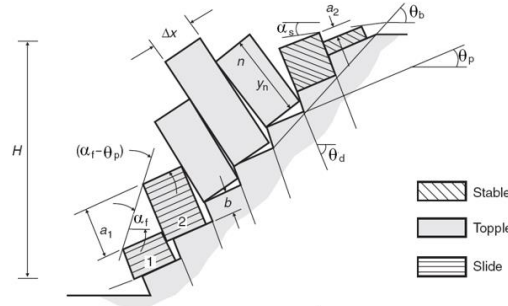


Figure 2.6 Model for limiting equilibrium analysis of toppling on a stepped base (Goodman and Bray, 1976).

$$FOS = \frac{\Sigma(\text{resisting moment})}{\Sigma(\text{driving moment})} = \frac{W \cos \Theta * (\frac{t}{2})}{W \sin \Theta * (\frac{h}{2})} = \frac{(\frac{t}{h})}{\tan \Theta} \quad 2.35$$

Where, h= height, t= width, Θ = dip angle and W= weight of the block.

2.4.4 Numerical Method

Analyzing slope stability by using conventional methods like the limit equilibrium method (LEM) was frequent practice for several years. But it is difficult for analyzing complex slopes with variable geometry, heterogeneous geological formation, nonlinear potential plane, uneven placement of water force, and initiation seismic loading conditions (Alzo'ubi, 2016). Because of the limitation and neglecting of some principal aspects of LEM, the majority of the researchers prefer the improved numerical method as a consequence of its rapid and effective development of computing capacity (Eberhardt, 2004). Numerical modeling is concerned with separating slopes in the process of examination into several finite elements or zones. Subsequently, the forces and strains in each zones will be analyzed using suitable constitutive laws. As a whole, numerical methods are divided into three approaches: continuum, discontinuum, and hybrid methods (Tang et al., 2016; Eberhardt, 2004).

2.4.4.1 Continuum Modelling

For slopes which considered as continuous like massive rock masses, weak rocks, and soil-like or serious fractured rock masses, the continuum modeling approach can be applied (Abd El-Latif, 2012; Raghuvanshi, 2017). The finite Difference Method (FDM), Finite Element Method (FEM), and Boundary Element Method (BEM) are categorized under continuum methods (Jing, 2003). The continuum codes perform simulation by assuming the material as it is continuous throughout the body. However, some of these continuum codes include discrete fractures such as faults and bedding planes but are inappropriate for the analysis of blocky mediums (Abd El-Latif, 2012). According to Raghuvanshi, (2017) constitutive model, in-situ stress, shear strength parameters for surfaces, groundwater, etc. are the basic input parameters that are needed for the continuum modeling approach. The final results are highly influenced by the assumption of continuum techniques defining rock mass as a continuum, for which simplifications are needed (Hack, 2002). FEM and FDM the two most widely utilized continuum methods are discussed below.

2.4.4.1.1 Finite Element Method (FEM)

FEM is the most applicable numerical method for rock mechanics and rock engineering since the early 1960s (Abd El-Latif, 2012). This method is preferable due to its flexibility in the analysis of heterogeneous slope materials, non-linear deformability, in-situ stress, and complex boundary condition (Coggan, et. al., 1998). Although the FEM is a continuum method, special elements such as joints or faults have been developed to directly represent the discontinuous behavior characteristic of joints and interfaces between adjacent blocks of material. These elements can have either zero thickness or thin, finite thickness. They can assume linear elastic behavior or plastic response when stresses exceed the strengths of discontinuities. Plaxis software is an example of finite element modeling in rock slopes. The disadvantage of this method is that considerable time is required in preparing input data for a typical problem. Moreover, unlike the discontinuum models such as DEM, which can readily model large displacements of blocks, most FEM codes only model small strains.

2.4.4.1.2 Finite Difference Method (FDM)

FDM, on the other hand, is one of the oldest numerical techniques used for the solution of sets of differential equations, given initial values and/or boundary values (e.g. Desai and Christian, 1977). In this technique, the slope geometry will be represented by a triangle, and differential equations for determining forces acting on the slope will be derived from this triangle by dividing it into connected series of discrete points called nodes from the

generalized form of Gauss divergence theorem (e.g. Malvern, 1969). This method does suffer from shortcomings, most of all in its inflexibility in dealing with fractures, complex boundary conditions, and material heterogeneity. This makes the standard FDM generally unsuitable for modeling practical rock mechanics problems.

2.4.4.2 Discontinuum Modelling

Discontinuum methods treat the rock slope as a discontinuous rock mass by considering it as an assemblage of rigid or deformable blocks and is suitable where the failure mechanism is controlled by pre-existing discontinuities (Abd El-Latif, 2012; Raghuvanshi, 2017). The analysis includes sliding along and opening/closure of rock discontinuities controlled principally by the joint normal and joint shear stiffness. Distinct Element Method (DEM), Discrete Element methods (DEM'), and Discontinuous Deformation Analysis (DDA) are discontinuum methods (Abd El-Latif, 2012; Stead et al., 2001). Slope geometry, discontinuity characteristic, discontinuity shear strength and stiffness, in-situ stress, and groundwater data are the information that is needed as input parameters (Raghuvanshi, 2017). The discontinuum method is quite effective to simulate the discontinuity behavior and assessing the rock slope stability for a given condition.

2.4.4.2.1 Distinct Element Method (DEM)

DEM is developed by Cundall, (1979) and it is a numerical model which describes the mechanical behavior of discontinuous bodies. This method monitors the particle interaction through contacts and calculates the motion of the particle by using Newton's second law based on the use of an explicit scheme (Abd El-Latif, 2012). Among the available distinct element programs, UDEC and 3DEC are the most known (Cundall, 1971; Itasca, 2004). Both types of programs require that the locations of preexisting discontinuities are known before analysis is begun. By including a large number of discontinuities, it is also possible, to some extent, to simulate the path that failure will take, under the assumption that failure only occurs along discontinuities. The disadvantage of this method is that an experienced user is needed for modeling.

2.4.4.2.1 Discrete Element methods (DEM')

The Discrete Element Method (DEM') is a numerical method similar in principle to the Distinct Element Method (DEM). It is used to compute the stresses and displacements in a volume containing a large number of particles of granular material such as grains of sand (Abd El-Latif, 2012). Today, DEM' is becoming widely accepted as an effective method of

addressing engineering problems in granular and discontinuous materials, especially in granular flows, powder mechanics, and rock mechanics. The particle shapes and geometries are specified by the user. Spheres or ellipsoids are commonly used.

2.4.4.3 Hybrid modeling

Hybrid modeling approaches utilize the combined capabilities of both continuum and discontinuum methods. The strength of the rock mass is a combined effect of strength along the rock joints and the strength of the intact rock between the joints (Alzo'ubi, 2016). Hybrid methods combine FEM/BEM, DEM/FEM, and DEM/BEM to take advantage of the strength of each method while avoiding many of its disadvantages (Abd El-Latif, 2012). FEM/DEM is used for non-linear or fractured near fields where an explicit representation of the fracture is needed. BEM is used for simulating far-field rocks as an equivalent elastic continuum. Hence, the hybrid of these methods provides a numerical technique for the effective representation of the effects of the far-field to the near field rocks.

2.5 Remedial measures (Stabilization methods) for slope instability

Slope instability can have significant socio-economic impacts even in the absence of serious injuries and facilities. In Ethiopia, slope failures from 1985 to 2012 (Woldearegay et al, 2011) have damaged dry weather roads, asphalt roads, and other facilities (like mini-hydropower, mills, and schools). Accordingly, considering the mitigation methods for the unstable slopes of rocks and soils is an essential task in engineering geology. From the various mitigation methods of slope instability, a few of them are listed below. The slope stabilization method principally increases the factor of safety through diminishing driving force by the way of which is listed in the following flow chart (Abramson et al., 2002).

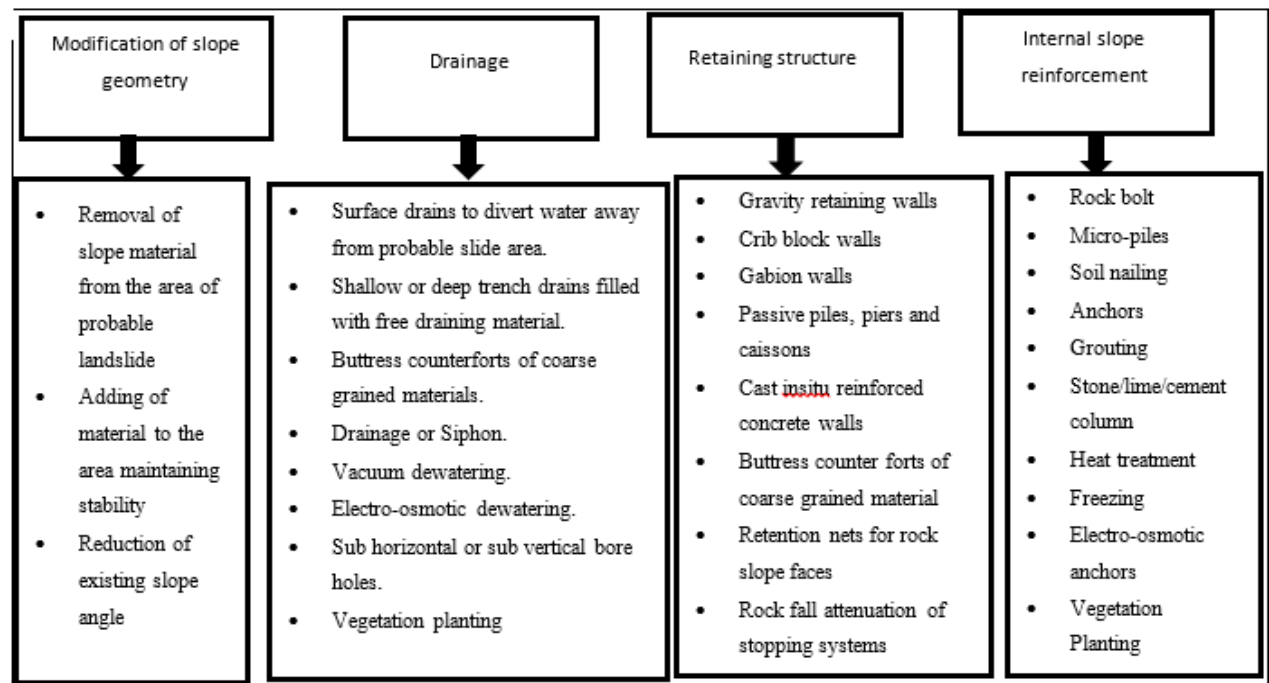


Figure2. 7 Flow chart showing different remedial measures for the slope stability problem.

2.6 Slope Stability Studies in Ethiopia

The slope instability problem in Ethiopia is extensively laid out in most of the roads constructed by crossing through mountainous regions in the country. Different investigators were carried out to look over the causative factors and to forward accurate remedial measures.

Lamessa and Meten, (2021) conducted a Stability analysis of rock slope along selected road sections from Gutane Migiru town to Fincha sugar factory, Oromiya, Ethiopia by using a detailed field survey, the basic rock mass rating, kinematic analysis and limit equilibrium methods (LEM) for the analyses. Planar and wedge modes of failure were identified. They concluded that potentially unstable rock slopes that caused frequent and significant damage to the road and natural environments were identified based on the field manifestations.

Cut soil slope stability analysis along National Highway at Wozeka–Gidole Road, Ethiopia was carried out by Bushira et al., (2018). Landslides were triggered by the construction of the section of the road between Wozeka and Gedole. The finite element method (FEM) and limit equilibrium method (LEM) were used for the identification of soil slope failure. Numerical results of FEM and LEM were compared using the calculated factor of safety (FOS). The comparison was made by analyzing modeling results from GEO-SLOPE 2007 (LEM based) and PLAXIS (FEM based) software. Shear strength parameters obtained after

saturation which was the worst-case scenario and shear strength parameters obtained by using field moisture content during sampling were used for comparisons.

Slope failures are the most common and widespread types of geo-hazard in Ethiopia where over 700 slope failure sites were recorded which mostly affect rural communities, infrastructures, farmlands, dwelling houses, and around 54 sites along road sections (Woldearegay, 2013). Slope instability problems in the form of rockfall, rotational failure of colluvial material, and debris slides are common events that can happen along the road section. Most of the time presence of loose unconsolidated materials (colluvial materials), highly weathered rock, steep natural slopes, poor drainage conditions, and the occurrence of high seasonal rains created favorable conditions for the occurrence of slope instability (Woldearegay, 2013).

According to Abebe et al (2010) slope failures in the Ethiopian highlands and the rift, margins were conducted for identifying slope failure types and differentiating the main triggering factors. The main predisposing factors of slope failure in the Ethiopian highlands pointed out as the high relief (on average between 2000 and 3000 m) induced by the large-scale Pliocene-Quaternary uplift (Almond, 1986; Mohr, 1986), and the resulting rugged morphology characterized by very deep valleys and gorges with high steep slopes. Active river incision is widespread and causes further slope steepening and instability. Steep escarpments, up to several hundred meters high, are also produced by faulting which is still active on the Rift margins which is the main triggering factor for the slope instability (Gouin, 1979).

Generally, this study conducted LEM and FEM-based slope stability analysis. LEM stability analysis was used for the structural controlled slope sections and both LEM and FEM were used for the non-structural controlled slope section, the addition of FEM is needed because it considers stress and strain conditions. In the analysis by LEM for the non-structural controlled slope section, the spencer method was used because it considered force equilibrium and moment equilibrium.

CHAPTER THREE

METHODS AND MATERIALS

3.1 Introduction

A coherent and, appropriate methodology is a principal task on the subject of achieving the objectives, answering research questions, and, resolving research problems identified earlier in the introduction. Consequently, different information about the selected road section for identifying the critical slope section was collected in this study. The research work was conducted by detailed field survey and sampling, laboratory testing of soil and rock samples, and, slope stability analysis. A literature review or initial desk study, field surveying and testing, laboratory testing of soil and rock, data processing, analysis, and interpretation were the general steps followed in this study. The general methodology and approaches used in this research work will be discussed as follows.

3.2 Desk Study, Reconnaissance Survey, and Secondary Data Collection

In this stage, a detailed review of different published and unpublished journals, reports, scientific conferences, standards, and guidelines was carried out on the subject matter related to the research topic. For example, a review on causative factors of slope instability, and theories and methodologies that are widely adopted for slope stability analysis was carried out. Moreover, the study area was also studied based on secondary data obtained from satellite imagery (i.e. Google Earth) and DEM obtained from USGS Earth Explorer to infer critical slope sections along the study area. This is then followed by a brief site reconnaissance survey to acquire information on the general condition of slope stability of the study area. Afterward, a conceptual framework and general methodology (Figure 3.6) were developed from the knowledge acquired through literature review and reconnaissance survey.

After the conceptual framework was developed, secondary data that are crucial for slope stability analysis such as geological maps, meteorology data, and topographic maps were collected from the Ethiopian Geological Institute, National Meteorological Service Agency, and Geospatial Agency of Ethiopia, respectively. Rainfall from meteorological data, in particular, is one of the major triggering factors for slope instability (Raghuvanshi, 2017) and hence, the graph of mean monthly rainfall distribution (Figure 1.4) was plotted to understand the influence of precipitation on the general stability of condition of the study

area. Seismicity, the another main triggering factor for slope instability, was adopted from the seismic hazard map of Ethiopia (Rawat & P, 2019) which provides amplitude of ground acceleration to be expected during 100 years return period.

3.3 Detailed Investigation

To generate primary data that were used for the stability analysis, a detailed field survey was conducted in the study area. The field survey included identification of critical slope sections, geological mapping, discontinuity surveying, in-situ strength test, and sampling. Identification of the critical slope section was done by using different indicators such as removal of slope toes, steep slope cuts, failure of geologic materials at the toe of the slope, presence of springs which indicates shallow groundwater, and orientation of discontinuities which favors slope instability (Figure 3.1). Once critical slope sections were known, geological units that constitute the corresponding slopes were determined, and slope sections were classified into structural controlled and/or non-structural controlled sections so that the data will be generated accordingly.



Figure 3. 1 A) Figure showing loose rock falling along the road section, B) evidence of spring on the slope surface, C) Spring called Chefe Hada Ragatu along the road section around Lake Wonchi and D) spring evidence along the road section to Woliso town.

3.3.1 Geological Mapping

Geological mapping was conducted along systematically selected traverse lines that cross the strike directions of major lithological units and geological structures of the study area. The mapping was carried out on the base map prepared during desk study with aid of a Global Position System (GPS), notebook, color pencil, and others. The result obtained from the fieldwork was converted into a geological map (1:50,000) of the study area using ArcGIS software (Figure 4.10).

3.3.2 Discontinuity Survey

A discontinuity survey has been carried out to provide parameters for slope stability analysis and rock mass classification. Accordingly, data on the parameters of the discontinuity was acquired following ISRM (1978) guideline. The orientations of discontinuities and slope were measured with the aid of the Brunton compass. Aperture, spacing, and persistence of discontinuities were measured using a tape meter. The degree of weathering of rocks was also determined based on the response of blows by the geological hammer. Similarly, other conditions of discontinuities such as surface roughness, infill material, and associated groundwater condition were visually estimated in the field.

3.3.3 In-situ Strength Test

The intact rock strength was determined in the field by using a simple and portable instrument called the Schmidt hammer. Aydin (2009) suggested that N-type hammers are less sensitive to surface irregularities and are preferred for field surveys hence, this hammer type was utilized in this study. The test was conducted following ASTM C805 (2018) guidelines as follows. The instrument was pressed against a random surface so that the spring will be released and the instrument is ready for the test.

- The instrument was then perpendicularly held against the test surface (Figure 3.2) and gradually hard-pressed until the hammer impacts the test surface with full spring.
- The hammer was then locked and the corresponding rebound value was recorded.
- Ten (10) readings were taken at each test location with each test point being far away by 25 mm.
- A representative rebound value was obtained by discarding the rebound values that differ from the average of 10 readings by more than 7 units and then by determining the average values of the remaining readings. Besides, the entire readings were

discarded and the rebound values were determined at other nearby locations in case more than two readings differ from the average of 10 readings by more than 7 units.

- Finally, the UCS was determined by using the following equation (eq3.1) according to Barton and Choubey (1977).

$$\text{Log (10) UCS} = 0.00088\gamma R + 1.01 \quad 3.1$$

Where UCS is uniaxial compressive strength in Mpa, γ is the dry unit weight of the rock in KN/m^3 , and R is the representative rebound value.

Joint wall Compressive Strength (JCS) was also determined from Schmidt hammer derived UCS as it is a crucial parameter for estimating shear strength parameters of failure plane. According to Barton (1976), and Bell (2007), JCS is equal to UCS if the rock is un-weathered/nearly un-weathered. However, JCS can be as low as 25% of UCS if the wall of the rocks is highly weathered (Barton, 1976; ISRM, 1978; Sharma et al., 1999). Hence, in this study concept was used to determine JCS from Schmidt hammer derived UCS and the degree of weathering of rocks.



Figure 3. 2 Insitu strength test.

3.3.4 Sampling

A representative rock and soil samples were taken from critical slope sections to generate input data that can be used for laboratory analysis via laboratory testing. Rock samples were acquired with the help of a geological hammer whilst soil samples were obtained from six test pits which were manually dug to the depth of 1.6 – 2.5 m. The samples were kept in a plastic bag to keep their natural moisture content.

3.4 Laboratory Test

Laboratory tests were conducted on the representative rock and soil samples to determine different parameters which are needed for the slope stability analysis. The laboratory tests that were conducted in this study will be discussed as follows.

3.4.1 Laboratory Test on Rock Sample

3.4.1.1 Unit Weight Test

Unit weight of the rock is a crucial parameter in rock slope stability analysis. It can be determined in the laboratory either through buoyancy or saturation technique (Abdulazeez, 2014). In this study, a buoyancy technique was adopted to determine the unit weight of irregular rock samples following the guideline suggested by ISRM (1978) at the laboratory of Oromia Engineering Corporation (ECO). Accordingly, the following steps were followed during this test.

- ✚ The rock specimens were weighed at their natural moisture (m_1) content using a digital balance.
- ✚ The rock specimens were then oven-dried at 105°C for (24 hrs.) and their corresponding dry masses (m_2) were recorded.
- ✚ The oven-dried rock samples were then immersed into a cylinder with water of known volume (V_1) and the corresponding change in volume due to this immersion of rock was recorded as V_2 . The volume due to the rock specimens was then calculated by subtracting V_2 from V_1 .
- ✚ Finally, the dry density and dry unit weight of rocks were determined using Eq. 3.2 and Eq. 3.3, respectively.

$$P_d = \frac{m_2 - m_1}{V_2 - V_1} \quad 3.2$$

$$\gamma_d = P_d \times g \quad 3.3$$

Where P_d is dry density, γ_d is dry unit weight and g is gravity.

The saturated unit weight was also determined using the same procedure. However, during this test saturated weight was first determined by keeping the rock specimen in the water for (24 hrs.) instead of determining dry weight.



Figure 3. 3 Unit weight test on an irregular rock sample.

3.4.2 Laboratory test on Soil Sample

The shear strength parameters and unit weight of soil are the predominant input parameters for the slope. Hence, direct shear tests and unit weight tests were conducted based on ASTM standards (Table 3.1) on representative soil samples obtained from critical slope sections. Moreover, tests such as grain size analysis and Atterberg limit were also conducted to obtain general information on types of soils.



Figure 3. 4 Sample preparation for the laboratory tests.

Table 3. 1 Type of test and standard employed for each type of soil laboratory tests

No	Type of test	Standard employed
3	Grain-Size Analysis	ASTM D 422 (1998)
4	Atterberg Limit	ASTM D 4318 (1998)
5	Unit Weight	ASTM D 7263 (2009)
6	Direct Shear	ASTM D 3080 (1998)

3.4.2.1 Particle size analysis

The particle (grain) size analysis test was conducted according to ASTM D422 (1998) standard for each soil sample which was taken from the slope on the verge of failure. The particle size analysis consists of two procedures (Arora, 1997) mechanical sieve analysis and hydrometer analysis. The mechanical sieve analysis involved the use of a series of mechanical sieves and was helped for the determination of the grain size distribution within the coarse-grained fraction of the soil. The hydrometer analysis was conducted on the fine-grain portion of the soil material. The soil that was passed in sieve number 200 was considered fine-grained. The result which was acquired from these tests were used to characterize the soil.

First 500g of oven-dried soil was prepared and the sieves were cleaned by using a brush without sieve number 200 in order to save the screen from damage then the soil was put in the sieve which was placed in decreasing order of sieve size. After shaking the sieve the material that was retained on each sieve was determined and from this percentage retained was determined. The material that was retained on the pan which was passed the sieve number 200 was considered as a fine-grained portion of the material and was used for hydrometer analysis.

For the hydrometer analysis, the first 60g of soil was measured from the soil which was left on the pan. Then the soil was mixed with sodium hexamethyl phosphate in a small evaporation dish. Next, the sodium hexamethyl phosphate soil solution was transferred into a dispersion cup and was mixed with the shake mixer for two minutes and then all the sodium hexamethyl soil solution was put into a clean sedimentation jar and was left for 24hrs. After the two minutes, the hydrometer was inserted and the first reading was taken at two minutes then the thermometer was inserted to take a temperature reading. The hydrometer and temperature readings were taken at 4, 6, 16, 30 minutes, and 1,2,4,8 and 24 hrs.

3.4.2.2 Atterberg limit Test

The test includes the liquid limit and plastic limit tests and they are used to classify the fine-grain portion of the soil. The practical application of atterberg limit in engineering geology includes soil description, quantitative soil classification, and correlation to engineering properties like shear strength which has a great contribution to slope stability analysis. The water content at which the soil changes from one state to another can be defined by atterberg limit. A soil with high water content has less resistant to deformation differently the soil with low water content has more shear strength (Arora, 1997).

Casagrande method was used in this study in order to determine atterberg limit values and the procedures forwarded by ASTM D4318 were followed. The air-dried soil sample which passes sieve No.40 (0.425mm) was prepared and mixed with distilled water with the help of a spatula up to it become a thick homogeneous paste. The water content required to close grooved soil samples by a distance of 13mm at 25 number of blows applied at a rate of 2 revolutions per second was taken as the liquid limit (LL) of soil. Furthermore, parts of soil specimens, that were used for liquid limit determination were rolled into a thread of 3mm diameter at different water content on a ceramic plate. The part of soil specimens that were crumbled when rolled into a thread of 3mm diameter was then taken for water content determination which corresponds to the plastic limit (PL). The plasticity index (PI) of the corresponding soil specimens was then determined using the following equation:

$$PI = \text{Liquid Limit (LL)} - \text{Plastic Limit (PL)} \quad 3.4$$

Finally, the soil was classified according to USCS depending on the coefficient of curvature and uniformity, particle size analysis, and atterberg limit test. The soil was also additionally classified according to AASHTO because it was taken from the critical sections of the road and also classified according to USCS. The group index was determined by using equation (3.5).

$$GI = (F_{200}-35) [0.2+0.005(LL-40)] + 0.01(F_{200}-15) (PI-10) \quad 3.5$$

Where, GI=group index, F_{200} =percentage passing sieve No.200, LL=liquid limit, PI=plasticity index.

3.4.2.3 Direct Shear Test

The shear strength of the soil is the resistance to deformation of soil particles upon the action of the shear stress. The shear strength of the soil which constituted the critical slope section

was determined by using direct shear apparatus under the consolidated drained condition according to ASTM D 3080 (2002) guideline at the Engineering Corporation of Oromia (ECO) laboratory. The test was conducted on soil specimen's dimension of 60*60*20mm which was compacted and molded in their natural moisture content. 100Kpa, 200Kpa, and 300Kpa consolidated pressure were applied respectively for shearing of the specimen until it fails. The horizontal and vertical shear displacements were then recorded at regular intervals. By using the test result (the shear strength parameters) then the shear strength was calculated by using equation (eq3.6) based on Mohr-Coulomb failure criteria.

$$S = c + \sigma \tan \phi \quad 3.6$$

Where, S=shear strength, c=cohesion, ϕ =friction angle, and σ =normal stress.



Figure 3. 5 Figure showing a direct shear test of soil in the laboratory.

3.5 Data Processing, Analyzation, and Interpretation

3.5.1 Rock Mass Quality

Rock Mass Quality is one of the empirical methods used to determine the quality of rock mass and analyze the stability of the slope section. In this study, Bieniawski's (1989) RMR system was adopted to determine the quality of rock mass at structural controlled critical slope sections. Of the five parameters used in this classification system, UCS of intact rocks was determined from the rebound value obtained from the in-situ Schmidt hammer test using

Eq. 3.1. The RQD was also determined using Palmström's (2005) Volumetric Joint Count Method as stated in Eq. 2.7. Similarly, remaining parameters such as Spacing and Condition of discontinuities, and Groundwater Condition were also directly estimated in the field. Accordingly, spacing of discontinuities and discontinuity conditions such as aperture and persistence was measured using a tape meter. The groundwater condition and the remaining discontinuity conditions such as roughness, wall weathering, and infill material were visually estimated. Upon determination of these five parameters, the rock mass quality at the selected slope sections was determined by adding a rating of five basic parameters as per Bieniawski's (1989) rating system.

3.5.2 Estimation of Shear Strength Parameters and Elastic Modulus of Rocks

According to Wyllie and Mah (2004), shear strength parameters (i.e. C and ϕ) of discontinuities for structural controlled slope sections, and of rock the mass for non-structural slope sections are very crucial for slope stability analysis using LEM and FEM. Similarly, the elastic modulus is also an important parameter during FEM analysis for non-structural controlled slope sections (Brinkgreve et al., 2013). In the light of these concepts, the aforementioned parameters were separately determined in this study using Rockdata software for structural controlled (i.e. failure along discontinuities) and non-structural controlled (i.e. failure with the rock mass).

In this study, an attempt was made to determine the shear strength of discontinuities (i.e. failure planes) using Rockdata software (Rocdata 3.0 Rocscience, 2004) following Barton and Bandi's (1990) non-linear failure criteria. This failure criterion uses parameters such as Residual Friction angles, Joint Roughness, Joint Wall Compressive strength, slope height, and unit weight of the rock to determine Mohr-Coulomb equivalent shear strength parameters. Accordingly, the following procedures were followed in this software:

- ✚ The residual friction angle used for the analysis was determined using Eq. 2.2 built-in Rockdata software. The input parameter for this equation such as dry and wet rebound values was determined using the in-situ Schmidt hammer test. Similarly, a basic friction angle of 32.5° for ignimbrite rock (the only exposed rock at all structural controlled slope sections) was adopted from the work by Ulusay & Karakul (2015).
- ✚ Joint Roughness was estimated by comparing the standard roughness profile suggested by Barton and Choubey (1977) which is built-in Rockdata software to those determined in the field.

- ✚ Joint wall Compressive Strength (JCS) was also determined through a comparison between Schmidt hammer-derived UCS and degree of weathering.
- ✚ The unit weight of rock (Table 7) was determined in the laboratory whilst slope height was determined in the field using a tape meter.
- ✚ Finally, the shear strength parameter (i.e. C and ϕ) of the discontinuities were determined using Barton and Bandi's (1990) non-linear failure criteria (Appendix 4).

Similarly, the shear strength parameters and elastic modulus of rocks at non-structural controlled slope sections (i.e. failure within the rock mass) using Rockdata software (Rockdata 3.0 Rocscience, 2004) following generalized Hoek-Brown Failure criteria (Hoek et al., 2002). This failure criterion uses input parameters such as UCS of intact rocks, GSI, intact rock material constant (m_i), disturbance factor (D), slope height, and unit weight of rocks. Afterward, the following procedures were followed in this software.

- ✚ UCS of intact rocks which is derived from Schmidt hammer rebound value using Eq. 3.1 was used in the analysis.
- ✚ GSI was directly determined in the field through comparison with a standard graph (Appendix 7) built in the Rockdata software.
- ✚ The material constant of intact rock (m_i) was adopted from the standard table of Rockdata software (Hoek et al., 2002) (Appendix 8).
- ✚ The disturbance factor was obtained from Wyllie and Mah, (2004) guideline as stated in this software (Appendix 9) based on the degree of disturbance of rock mass subjected to mechanical excavation.
- ✚ The unit weight of rock was determined in the laboratory whilst slope height was determined in the field using a tape meter.
- ✚ Finally, the Mohr-Coulomb equivalent and modulus of deformation were determined using generalized Hoek-Brown Failure criteria.

3.5.3 Soil Stiffness and Poisson Ratio of Rocks

Young's modulus (E) and Poisson's ratio (ν) are one of the crucial parameters in slope stability analysis using FEM. In this study, an attempt was made to determine the Poisson ratio and Young's modulus of soil empirical equations. The drained Poisson ratio of soils was estimated using Eq. 3.7 as forwarded by Kulhawy & Mayne (1990).

$$\nu_d = 0.1 + 0.3\left(\frac{\phi - 25}{20}\right) \quad \text{For } \phi < 45^\circ \quad \text{Eq. 3.7}$$

Similarly, Young's modulus of soils was estimated from the corrected standard penetration (SPT) N value using Eq. 3.8 as suggested by Begemann (1974).

$$E = 1200(N + 6) \frac{KN}{m^2} \text{ for } N < 15 \quad (\text{Eq.3.8})$$

$$E = 4000 + 1200(N - 6) \frac{KN}{m^2} \text{ for } N > 15 \quad (\text{Eq.3.9})$$

In this study, the SPT N value used in Eq. 3.8 and 3.9 above were determined using Eq. 3.10 which was forwarded Japan Road Association (1990) for soils dominated by sand as in the case of the study area.

$$\phi = (15N)^{0.5} + 15 \text{ for } \phi < 45^\circ \quad 3.10$$

As for rock slope sections, Young's modulus was determined using Rockdata software as discussed in section 3.4.2. The Poisson ratio (ν) was, however, determined using Eq. 3.11 forwarded by Vásárhelyi (2009) from input parameters such as GSI and intact rock material constant.

$$\nu = -0.002GSI - 0.003mi + 0.457 \quad \text{Eq. 3.11}$$

3.5.4 Slope Stability Analysis

Upon collection and extraction of all necessary data, slope stability analysis of critical slope sections was carried out via the application of LEM and FEMs. The analysis was first initiated by categorizing slope sections into either structural controlled or non-structural controlled based on their field manifestation. For structural controlled slope sections, the analysis was first conducted using the kinematic method to identify the mode of failure followed by the application of LEM for the determination of FOS. For non-structural controlled slope sections, on the other hand, FOS and SRF were directly determined by using LEM and FEM, respectively.

3.5.4.1 Kinematic Analysis

Rock slope stability analysis mostly starts with a kinematic analysis to check whether or not the orientation of discontinuities leads to slope failure (Wyllie and Norrish, 1996). In this study, kinematic analysis was executed using Dips6.00 software (Dip6.0 rocscience, 2004) for structural controlled slope sections to check their kinematic admissibility. This software utilizes friction of angle of discontinuities (predefined failure plane), dip amount and dip direction of discontinuities, and slope as an input parameter. The friction angle of discontinuities was determined using Rockdata software (Rocdata 3.0 Rocscience, 2004)

following Barton and Bandi's (1990) non-linear failure criteria as stated in section 3.4.2. The remaining parameters such as dip amount and dip direction of discontinuities and the slope were directly measured in the field using Brunton Compass. Afterward, the kinematic admissibility of each discontinuity for planar, wedge, and toppling mode of failures was done using Dips6.00 software (Dip6.0 rocscience, 2004) (Figure 5.1).

3.5.4.2 Slope Stability Analysis Using LEM

The kinematic analysis provides the likely mode of rock slope failure without information on the stability of the slope in terms of FOS or probability of failure (Park and West, 2001). Hence, in this research work, further stability analysis was performed in terms of FOS using LEM based on the mode of failure identified by kinematic analysis for structured controlled slope sections. In this regard, only two modes of failure namely planar and wedge modes of failures were identified for structural controlled rock slope sections of the study area. Accordingly, the FOS for the planar mode of failure was then determined using LEM-based Rocplane 2.0 software (Rocplane 2.0 Rocscience, 2004) based on the study by Hoek & Bray (1981) and Sharma et al. (1995). Similarly, the FOS for wedge mode of failure was also determined using LEM based approach developed by Hoek & Bray (1981) using Swedge 4.0 software (Swedge 4.0 Rocscience, 2004). Both software utilizes shear strength parameters, the orientation of discontinuities, slope geometry, and unit weight of rocks as input parameters (Tables 11 and 13) to determine the FOS of the slope under consideration. The shear strength parameters were determined using RocData software (Rocdata 3.0 Rocscience, 2004) as per Barton and Bandis's (1990) non-linear failure criteria as discussed in section 3.4.2. The orientation of discontinuities and slope geometry were determined during the discontinuity survey whilst the unit weight of the rocks was determined in the laboratory using buoyancy techniques as per ISRM (1978) guidelines. Afterward, the analysis was executed to determine FOS under found conditions namely static dry, static saturated, dynamic dry, and dynamic saturated conditions (Table 12 and Table 14). For the dynamic condition, horizontal seismic acceleration of 0.07g was adopted for the study area from (Rawat & P, 2019).

The stability of non-structural controlled slope sections (i.e. slope sections constituted by soils and/or highly weathered rocks) was also determined using LEM with the aid of Slide 6.0 (Slide 6.0 Rocscience, 2004) following the Mohr-Coulomb strength criterion to define the material properties of the slope. This software uses shear strength parameters, slope

geometry, and unit weight of material/s constituting the slope under consideration to determine stability in terms of Factor of Safety (FOS). The soil shear strength parameters were determined using a direct shear test whilst the shear strength parameters of the rock mass were determined from Rockdata software (Rockdata 3.0 Rocscience, 2004) following Hoek-Brown Failure Strength Criteria as stated in section 3.4.2. The remaining parameters such as unit weight and slope geometry were determined via laboratory testing and discontinuity survey, respectively. Upon completion of input parameters, the analysis was done using the Auto Grid Search Method for determining the minimum slip surface corresponding to FOS. Moreover, the Spencer and GLE/Morgenstern-Price methods were used to determine FOS as both methods utilize both force and moment equilibrium to determine the FOS.

3.5.4.3 Slope Stability Analysis Using FEM

One of the drawbacks of LEM is its inability to consider in-situ stress and non-linear deformability (Coggan, et. al., 1998, Abd El-Latif, 2012). The FEM, however, considers stress conditions, and the shape of the failure surface is not predetermined as opposed to LEM. Having this notion, the stability of non-structural controlled slope sections was also determined using FEM with the aid of Phase 2.0 (phase 2.0 Rockscience, 2004) and Plaxis (Brinkgreve et al., 2013). The input parameters for both software's are similar to that of Slide6.00 with exception of the Poisson ratio and Young's modulus. For the analysis, these two parameters were determined using different empirical equations as discussed in sections 3.4.2 and 3.4.3. Finally, the Stress Reduction Factor (SRF) was computed under static dry, dynamic dry, static saturated, and dynamic saturated via Phase 2.0 software, and under static dry and static saturated conditions via Plaxis software. Likewise in the LEM, during the dynamic condition horizontal seismic acceleration of 0.07g was adopted for the study area from EBCS (1995).

Finally, FOS obtained from LEM-based Slide 6.0 software and FEM-based Phase 2.0 and Plaxis software were compared to each other for the validation of the results.

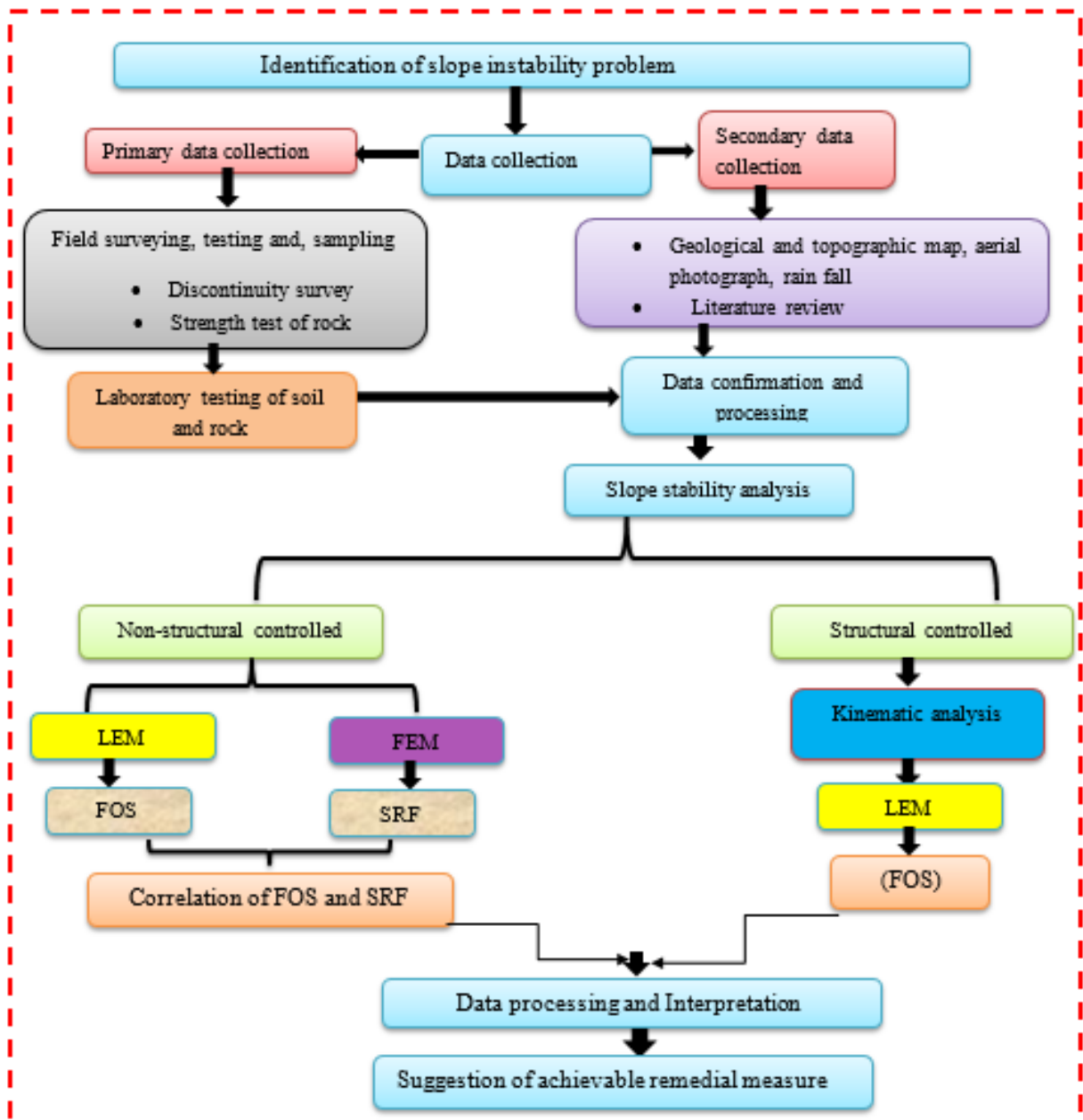


Figure 3. 6 Methodology flow chart.

CHAPTER FOUR

GEOLOGY OF THE STUDY AREA

4.1 Regional Geology

The geology of Ethiopia consists of Precambrian basement, Paleozoic to Mesozoic sediments, and Cenozoic rocks which include tertiary to Quaternary volcanic and quaternary sedimentary deposits (Mengesha et al., 1996; Tadesse et al., 2003; Abbate et al., 2014). The Precambrian basement rocks of Ethiopia are associated with Pan-African Orogeny. This Orogeny is divided into the Mozambique Belt and Arabian Nubian Shield and is formed by the collision between east and west Gondwana at the end of a Wilson Cycle that encompassed most of the Neoproterozoic history of Ethiopia (Stern, 1994). These Neoproterozoic basement rocks of Ethiopia are widely exposed in the southern, northern, and western parts of the country. The Neoproterozoic rocks were followed by a long period of peneplanation in the Palaeozoic, with the pre-Cambrian orogenic mountain ranges almost completely worn down by denudation during this period (Mohr, 1962). The Palaeozoic peneplanation was followed by extensional deformation associated with Karroo type rifting which lead to the transgression of the Indian Ocean along the Horn of Africa (Guiraud et al., 2005). This transgression has led to the formation of sedimentary rocks in Ethiopia which roughly accounts for 33 % of the total surface of the country in five major basins (the Ogaden Basin, the Blue Nile Basin, the Gambela Basin, the Southern Rift Basin, and the Mekelle Outlier) (Wolela, 2008). The Mesozoic sediments of Ethiopia are overlain by Cenozoic volcanism which started during the Eocene-Late Oligocene with the eruption of the Ethiopia–Yemen flood-basalt province (Trap series) (Tefera et al 1996; Corti, 2009). These Cenozoic volcanic cover about 52 % of the surface area of the country and are widely distributed in the eastern and western Ethiopian highlands and within the Main Ethiopian Rift (MER) and the Afar depression (Mohr & Zanettin, 1988). Quaternary sedimentary rocks are also reported in eastern Ogaden, the Danakil depression, and the Lower Omo Valley. (Kazmin 1972; Mohr and Zanettin 1988; Tefera et al 1996).

The Cenozoic volcanics in particular mostly erupted 30 Ma ago as flood basalt during a short 1M year period to form a vast volcanic plateau: the western and eastern Ethiopian plateau (Hofmann et al., 1997). This volcanic episode was characterized by the extrusion of flood tholeiitic to alkaline lava flows, with magmatic characteristics varying in different regions of the plateau (Kieffer et al., 2004); interbedded with the flood basalts, particularly at upper

stratigraphic levels, are felsic lavas and pyroclastic rocks of rhyolitic, or less commonly, trachytic compositions (Mohr and Zanettin, 1988). The flood basalt volcanism (trap series) was followed by rifting (Mohr, 1983; Mohr & Zanettin, 1988) that opened along the Red Sea and the Gulf of Aden eventually leading to the formation of the Main Ethiopian Rift (MER) (Baker et al., 1996).

A regional-wise, the study area is located between the western Ethiopian plateau and the margin of the central Main Ethiopian Rift (MER) specifically within the central part of the Ethiopian Volcanic Province. It is bounded to the north by E-W-trending Yerer-Tulu-Welel Volcano-tectonic Lineament and holds a transition zone between the Ethiopian plateau and MER forming a remarkable embayment determined by ESE-WNW cross rift structures.

Regionally, about eight geological units were identified by Alemu et al. (2010). These include the Alluvium deposit, Babich-Guder Basalt, Dendi-Wonchi Pyroclastic deposit, Gesh Megal Rhyolite, Tarmaber Megezez Formation, Wechecha Trachyte and welded to partially welded pyroclastic deposit (Figure 4.1).

4.2 Regional Structures

Lineaments are the main regional structures of the study area. They are dominantly characterized by four distinct sets of linear features trending in NE-SW, NW-SE, N-S, and rarely E-W directions (Figure 4.1). In addition to lineaments, several subordinate jointing and fracturing are also common structural features of the study area with a nearly regular cutting in the different rock types.

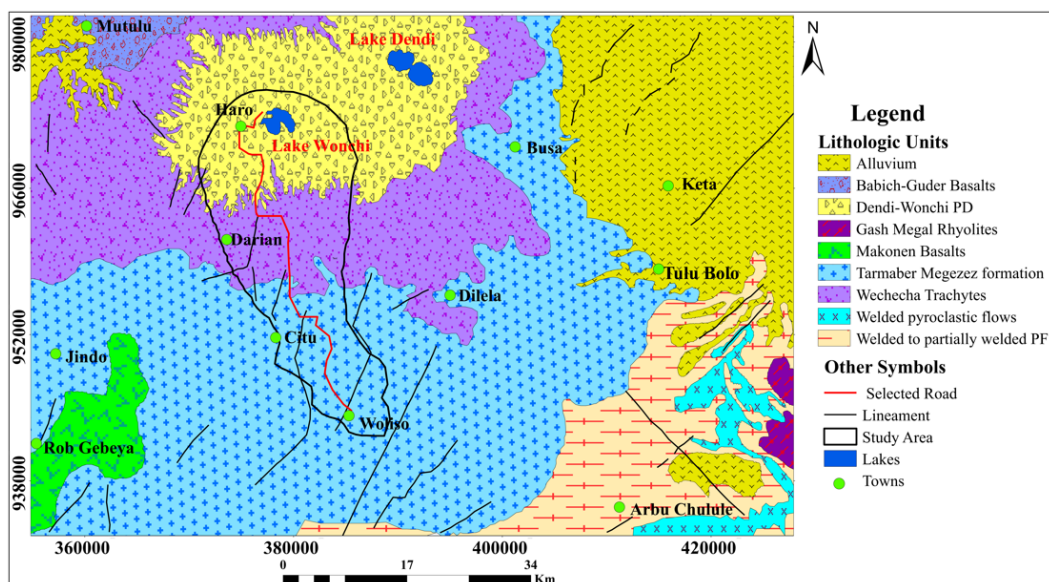


Figure 4. 1 Regional geological map of the study area (After Alemu et al., 2010).

4.3 Local Geology and Structures

Based on the regional geological map of the study area produced by Alemu et al. (2010), the entire study area selected for local geology was mapped as trachyte, pyroclastic deposit, and basaltic rocks in the Akaki Beseka geological map sheet. In this study, however, an attempt was made to produce a geological map at a scale of 1:50,000. Accordingly, Quaternary soil deposits and volcanic rocks such as pumice, scoria, aphanitic and vesicular basalt, ignimbrite, and tuff rock units were identified in the study area (Figure 4.10).

4.3.1 Pumice

This rock unit is exposed in large amounts, especially on the road from Haro town to Lake Wonchi. It is coarse-grained and light to dark gray colored (Figure 4.2). As per Bonewitz (2012) pumice is known to show glassy nature whilst the pumice of the study area has lost its glassy appearance due to its weathering and exposure to sufficient water. The pumice found in some parts of the study area is converted to grey color soil. Field investigation revealed that some critical slope sections are located within this rock where it dips toward the slope face/or road.

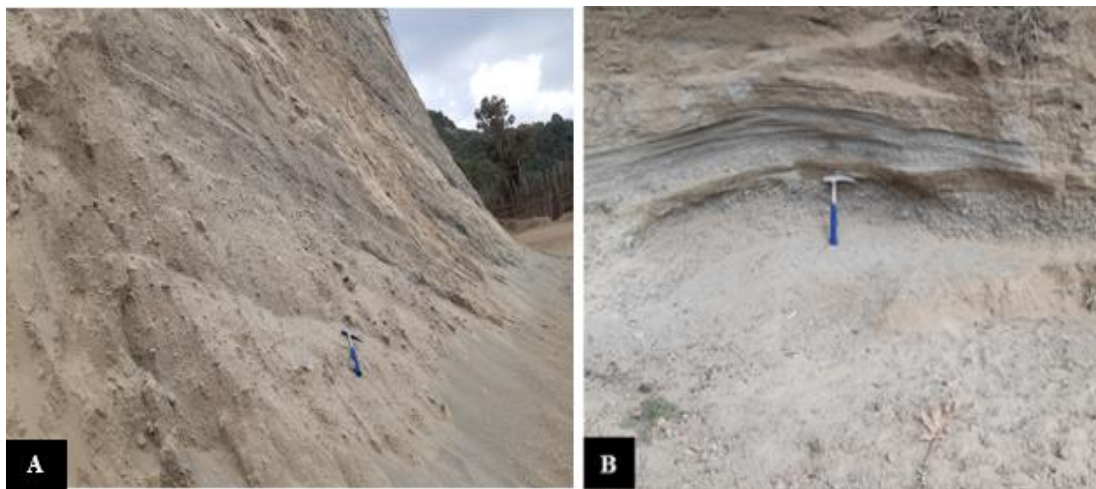


Figure 4. 2 Pumice along Wonchi Lake road.

4.3.2 Scoria

This rock unit is dominantly found around Woliso town (Figure 4.3). It has a vesicular texture and is whitish to reddish-brown in color. In some parts of the study area, this rock unit has decomposed to a reddish-brown soil deposit. This rock unit is currently being used as construction material for gravel road construction around the study area.



Figure 4. 3 Highly weathered and decomposed scoria.

4.3.3 Tuff

This rock unit is dominantly exposed around Lake Wonchi (Figure 4.10). It has a variable color ranging from light to reddish-brown (Figure 4.4) and has usually a pyroclastic texture. It is weak to slightly welded and highly weathered as it disintegrates into pieces with one or two blows of a geological hammer. In some areas, it is also affected by closely spaced fractures whilst systematic joint sets are not observable on the surface of this rock. In most areas, this rock unit has exposed forming escarpments with sharp-pointed hills and steep-sided slopes. Moreover, this rock unit is also outcropped in the majority of the study via intercalation with thin layers of pumice which facilitated slope failure (Figure 4.4).



Figure 4. 4 Tuff exposed along the road section.

4.3.4 Ignimbrite

This rock unit is dominantly outcropped around the western part of Haro Wonchi town (Figure 4.10). It has a dominantly grey color and aphanitic to eutaxitic texture. It is

dominantly strong and characterized by low to medium degree of weathering as it requires numerous blows of a geological hammer to disintegrate into pieces. Moreover, it is also dominantly affected by open to partially filled and gentle to steeply dipping highly persistent systematic joint sets (Figure 4.5). It is outcropped forming a steep escarpment and all structural controlled slope sections are situated with this rock type.

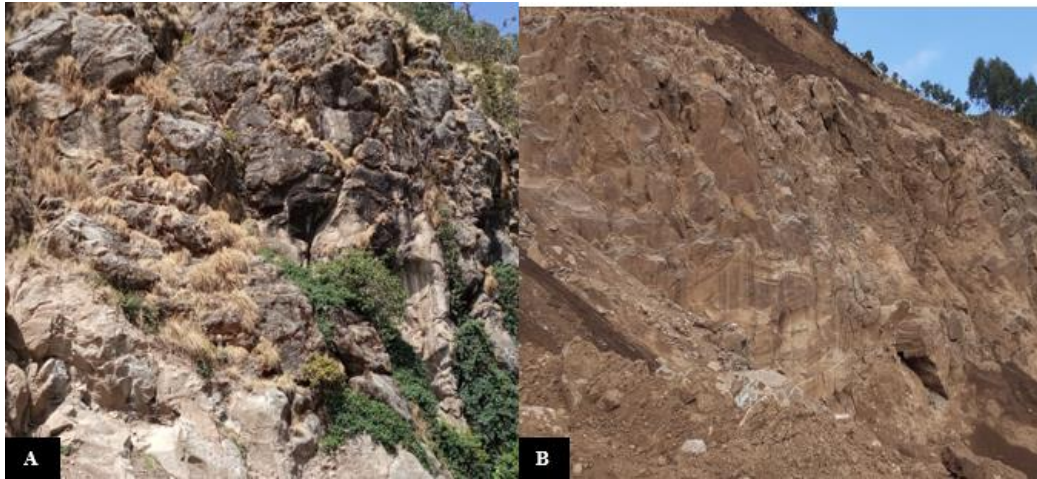


Figure 4. 5 A) Ignimbrite rock with discontinuities and B) ignimbrite rock unit where the discontinuity is partially filled with soil.

4.3.5 Basalt

This rock unit is dominantly outcropped from Citu area to Woliso town (Figure 4.10). Based on the field manifestation, the basaltic rocks of the study area are classified into two namely Vesicular and Aphanitic Basalt. The vesicular basalt is dominantly exposed as a patchy and discontinuous outcrop in the form of a boulder and granulated basalt (Figure 4.6). It is predominantly dark to dark green in color, vesicular in texture, and is characterized by a medium to a high degree of weathering based on response to blow by a geological hammer. Wide and persistent joints are rarely observable on this rock unit as it is exposed in the form of patchy.

The aphanitic basalt, on the other hand, is exposed continuously along relatively gentle topography. It is characterized by dark grey color, fine-grained texture, and a medium degree of weathering. Contrary to the vesicular basalt, this rock unit is dominantly affected by two to three sets of highly persistent joint sets.

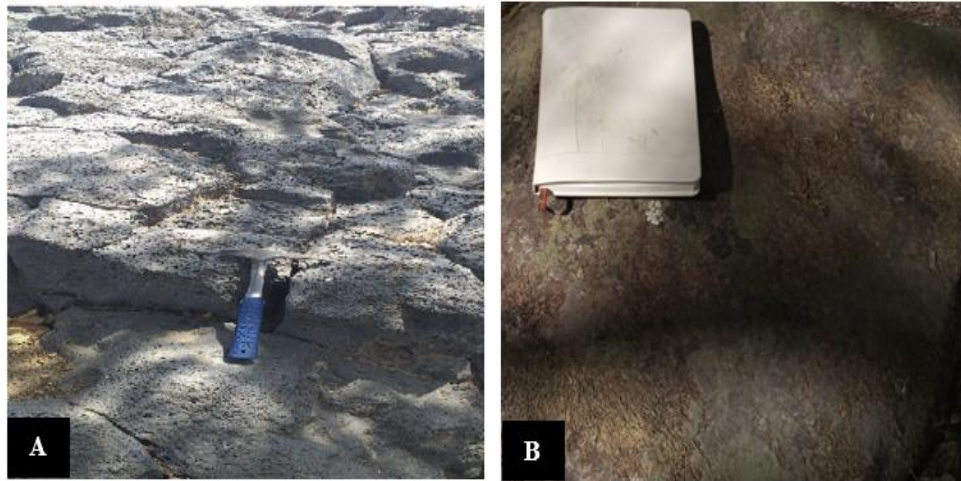


Figure 4. 6 A) vesicular basalt and B) aphanitic basalt.

4.3.6 Soil Deposits

Based on the field manifestation, three types of soil deposits are identified in the study area. These include residual, colluvial and alluvial soil deposits. Residual soils are types of soils formed by physical and/or chemical processes and which are not transported away from the place of origin (Duarte & Rodrigues, 2018). In the study area, this soil unit is exposed around Darian and South of Haro Wonchi Town (Figure 4.10) and shows variable colors (i.e. grey, black, and reddish color) with variation in the location of exposure. In some areas, erosional surfaces are visible on the surface of this soil unit (Figure 4.7). Moreover, residual soils are the type of soil that outcropped almost in all non-structural controlled slope sections that contain soil.

Colluvial deposits, on the other hand, are weathered materials that are transported by gravity and occasionally by water, and that are deposited on and at the toe of the slope. In the study area, this deposit is occasionally deposited at the toe of slope cuts along the road from Haro Wonchi town to Lake Wonchi. It dominantly consists of soils of variable grain sizes ranging from clay to gravel and boulders. This soil unit covered a very small area and hence, it is not mapped taking the scale of the geological map used in this study.

Alluvial soil deposits are those materials transported by river water and usually deposited along the river pathway either in the river bed or in its flood plain. In the study area, this deposit is mainly exposed along the center and banks of rivers of the study area such as Citu River, etc. This deposit mainly consists of clay soil and sub-angular to rounded boulders. Likewise colluvial deposit, this deposit also covered a very small area, and hence, it was not mapped.

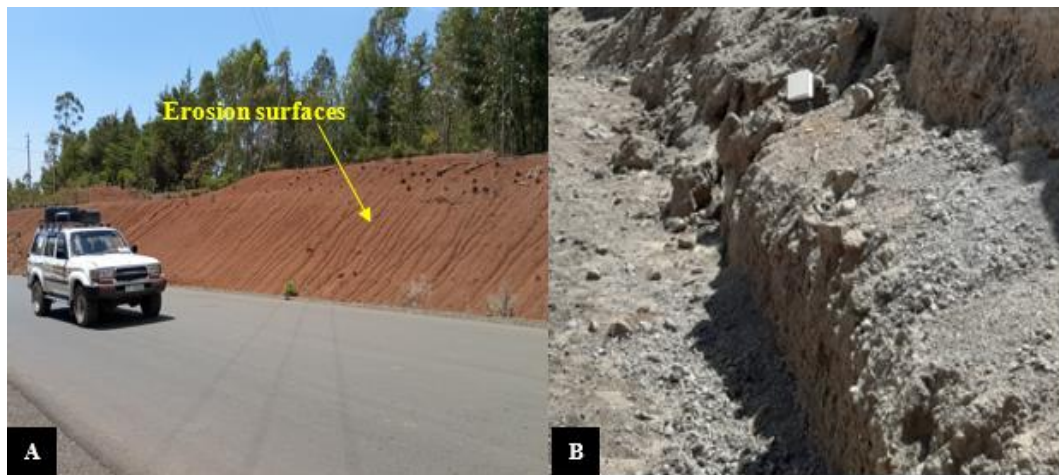


Figure 4. 7 A) Residual soil with visual erosion surface along the road section and B) residual soil (tuff derived soil) deposited around Lake.

4.4 Geological Structures

Geological structures such as joints and faults are a common cause of slope failure. In the study area, secondary geological structures such as joints, lineament, and dykes are identified.

4.4.1 Joints

Joints are the most common secondary geological structure identified in the study area. The analysis of orientation data in terms of the Rosset diagram using Dips 6.0 (Dip6.0 rocsience, 2004) showed that the study area is characterized by three joint sets oriented in NNE, ESE, SSW, and WNW directions (Figure 4.8). The characteristics of these joint sets vary from close to wide spacing, open to tight opening, open to partially filled infill property, and high persistence. Among the rocks of the study area, this structure is dominantly observable in ignimbrite and aphanitic basalt rock units.

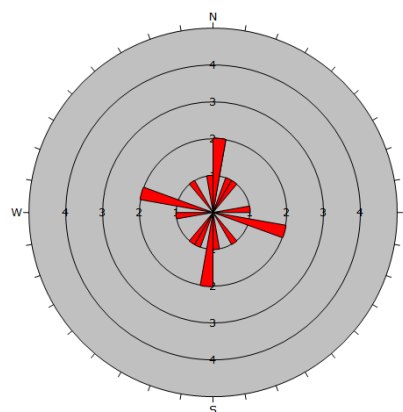


Figure 4. 8 Rosset diagram which illustrated the orientation of joint sets along the section.

4.4.2 Dyke

A basaltic dyke was also identified in the study area along the road from Haro Wonchi to Woliso town. This dyke cuts through tuff intercalation with pumice at an angle of 20° and is characterized by its dark color and fine-grained texture (Figure 4.9). Moreover, it has relatively strong strength as compared to the intruded rock (i.e. tuff intercalation with pumice).



Figure 4. 9 Dyke in the tuff-pumice intercalation rock unit.

4.4.3 Lineament

The lineaments in the study area are distinguished based on the information from satellite imagery (i.e. Google Earth Image) and 30m resolution SRTM DEM. Moreover, bodies of water such as rivers, lakes, and streams were also used for the identification of lineaments. Accordingly, some lineaments with the orientation of NNE are identified in the study area (Figure 4.10).

Finally, a geological map of the study area was produced on a scale of 1: 50,000 (Figure 4.10). Moreover, the geological cross-section was also produced along with lines A to B.

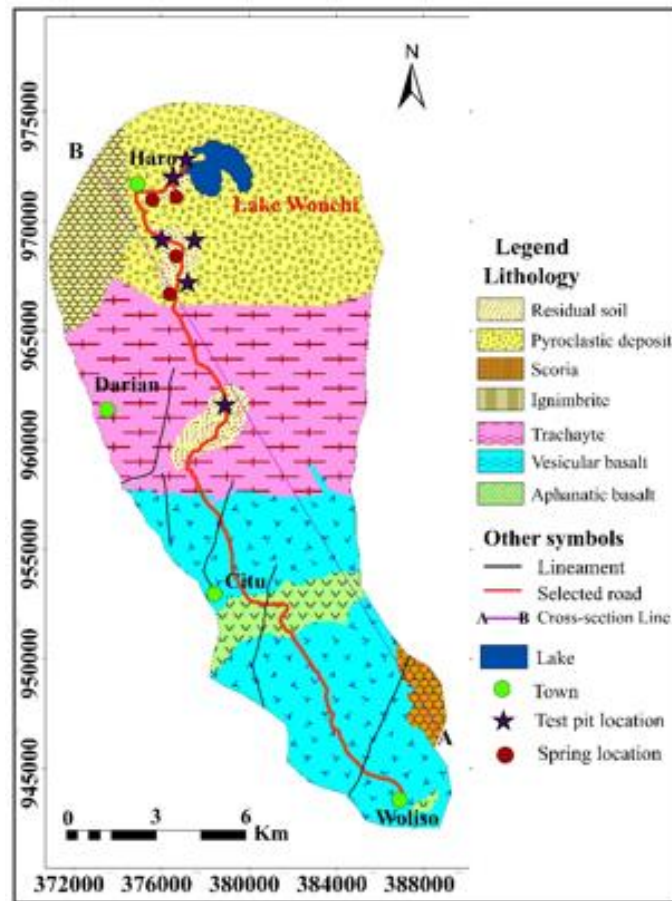


Figure 4. 10 Geological map of the study area.

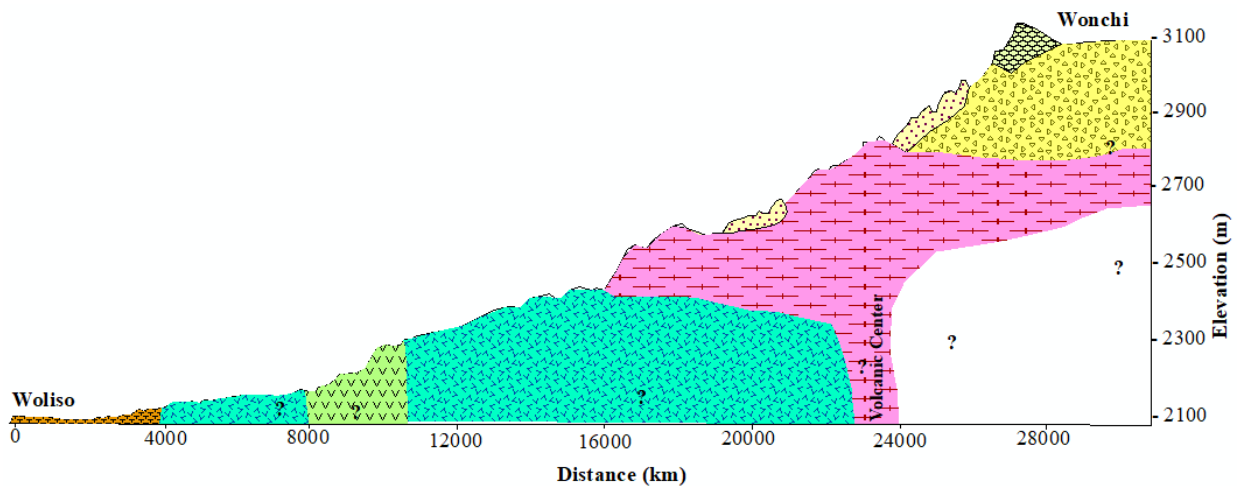


Figure 4. 11 Geological cross-section

4.5 Seismicity of the Study Area

Seismicity is one of the major triggering of slope failure (Keefer, 2000; Parise and Jibson, 2000) and hence, it is an essential component of slope stability analysis. In this study, an attempt was made to analyze the seismicity of the study area based on the Seismic Hazard

Map of Ethiopia produced by EBCS (1995). Based on this map, the study area lies in zone 3 (Figure 4.12) and hence, it is characterized seismic acceleration coefficient of 0.007g (Table 4.1). Hence, this value was used for slope stability analysis during dynamic conditions.

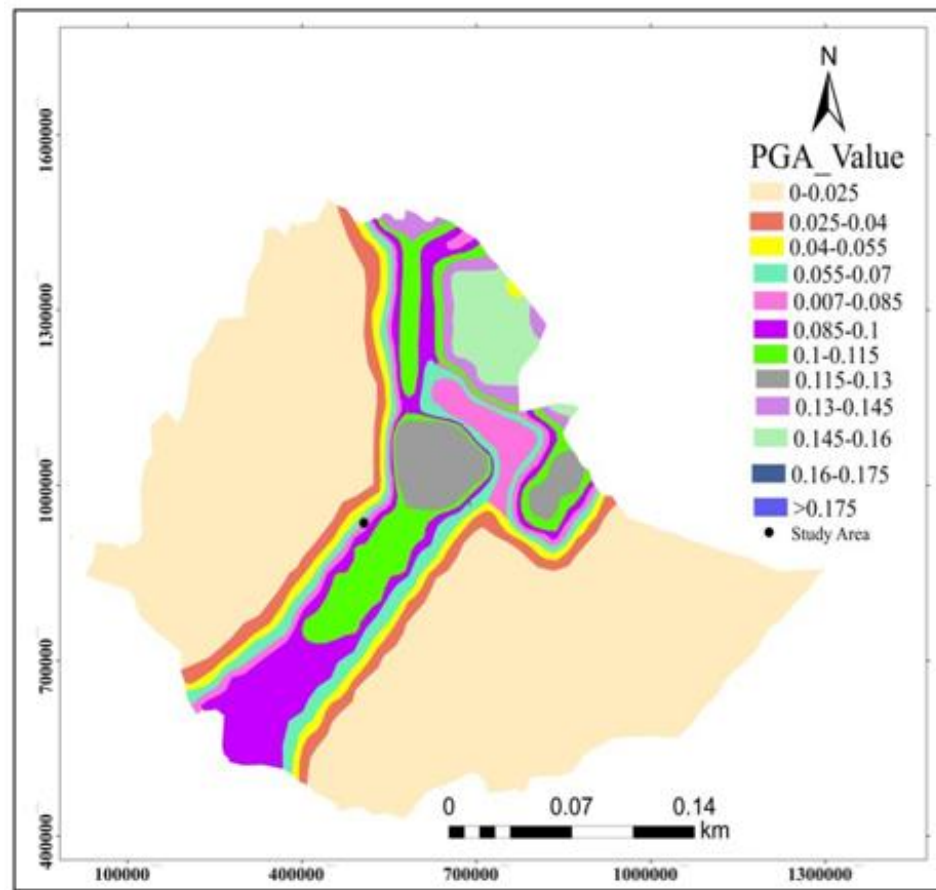


Figure 4. 12 Seismic risk map of Ethiopia in terms of peak ground acceleration (Rawat & P, 2019)

CHAPTER FIVE

RESULTS AND DISCUSSIONS

5.1 Identification of Critical Slope Sections

A detailed geological field investigation was carried out to identify critical slope sections and acquire primary data that can be used for slope stability analysis. The critical slope sections are identified based on field appearance such as steep slope cut, cracks observed on the slope face, failed materials at the toe of the slope, the geological structure orientation data, and the presence of springs on the slope face. The spring, in particular, is observable in most part of the study area especially along the road sections from Haro Wonchi town to Wonchi Lake showing that groundwater act as the main triggering factor for slope failure in the study area. Accordingly, about six non-structural controlled slope sections (i.e. heterogeneous slopes composed of soils and highly weathered rocks) and three structural controlled rock slope sections were identified based on the above-mentioned field manifestations (Table 5.1).

Table 5. 1 Critical slope sections identified during a detailed field investigation.

Slope Section	Location			Characterization of the slope according to field manifestation.
	E	N	Elevation	
Structural Controlled Rock Slope Section				
RSS1	372788	972647	3103	Critical slope section constituted by moderately weathered Ignimbrite
RSS2	372790	972652	3112	Critical slope section constituted by Slightly weathered Ignimbrite
RSS3	372795	972701	3100	Critical slope section constituted by Slightly weathered Ignimbrite
Non-Structural Controlled Slope Sections				
SSS ₁	376863	971620	2995	Critical slope section is constituted by grey silty sand and poorly-graded silty sand.
SSS ₂	376803	971378	3095	Critical soil sections composed of silty sand and pumice-tuff intercalation (medium and slightly weathered tuff).
SSS ₃	376659	971312	3062	Critical slope section composed of low plastic silt at the top and slightly welded tuff at the bottom.
SSS ₄	376633	971476	3096	Critical slope section constituted by slightly and medium welded tuff with pumice intercalation.
SSS ₅	376987	971556	2975	Critical slope section composed silty sand and highly weathered basalt.
SSS ₆	377378	959518	2490	Critical slope section composed of high plastic silt and intercalation of pumice and slightly welded tuff.

Upon identification of critical slope sections, relevant data pertinent to slope stability analysis were directly measured at the corresponding slope sections. The geological units constituting the slope sections are described in terms of their color, texture, and degree of weathering. Similarly, discontinuity parameters such as orientation, spacing, aperture, persistence, roughness, infill material, and wall weathering were also directly determined in the field. Moreover, the strength of intact rocks (i.e. Uniaxial Compressive Strength (UCS)) was directly determined in the field using a Schmidt hammer (Table 5.2). The RQD of the rocks was also determined in the field from the spacing of discontinuities using Palmstrom's (2005) Volumetric Joint Count Method (Table 5.3). Rock mass characterization and slope stability analysis were then done based on data obtained from such detailed survey and laboratory testing of samples.

5.2 Engineering Geological Characterization of Rocks

5.2.1 Uniaxial Compressive Strength (UCS)

The Uniaxial Compressive Strength (UCS) of the rocks of the study area was determined from rebound values acquired from the N-type Schmidt hammer test in the field. The rebound values acquired from this test were converted into UCS using Eq.3.5 forwarded by Barton and Choubey (1977). Table 5.2 below presents some representative UCS of rocks in the study area.

Table 5. 2 UCS value of intact rocks at critical slope sections and their corresponding Schmidt hammer rebound value.

Slope section		Schmidt hammer rebound value	Representative rebound value (R)	UCS (MPa)
Rebound and UCS Values at Structural Controlled Slope Sections				
RSS ₁		48, 50, 50, 52, 52, 54, 54, 56, 56, 58,	53.0	100.146
		24, 28, 30, 32, 32, 32, 34, 34, 36, 38	32.9	42.170
RSS ₂		42, 43, 44, 46, 48, 48, 50, 51, 51, 52	47.5	91.075
		38, 39, 41, 42, 42, 43, 44, 44, 46, 48.	42.7	73.013
RSS ₃		52, 54, 54, 55, 56, 58, 58, 59, 60, 60	56.6	136.490
		48, 50, 52, 52, 52, 54, 56, 60, 60, 60	54.4	123.424
		46, 50, 54, 52, 52, 48, 50, 56, 54, 58	52.0	110.586
Rebound and UCS Values at Non-Structural Controlled Slope Sections				
SSS ₂	SWT	11, 12, 12, 12, 12, 13, 14, 14, 14, 14	12.8	17.2
	CT	12, 16, 16, 16, 18, 18, 18, 20, 20, 22	17.6	21.61
SSS ₃	SWT	12, 12, 13, 13, 13, 14, 14, 14, 16, 18	13.8	17.9
SSS ₄	SWT1	11, 11, 11, 12, 12, 12, 12, 14, 14, 16	12.5	16.98
	SWT2	11, 11, 11, 12, 12, 12, 12, 13, 13, 14	12.1	16.7
	SWT3	14, 14, 15, 16, 16, 16, 16, 17, 18, 18	16	19.57
SSS ₅	HWFB	20, 21, 22, 23, 24, 24, 24, 24, 25, 26	23.3	28.9
SSS ₆	SWT	14, 14, 15, 15, 16, 16, 17, 17, 18, 22	16.4	19.8

The analysis of the results showed that the UCS of rocks of the study area (for structural controlled slope sections) ranges from 42.170MPa to 136.490MPa for the ignimbrite rock at slope section RSS₁ to RSS₃, respectively and for non-structural controlled slope section ranges from 16.7 MPa to 28.9 MPa (Table 5.2). Moreover, based on the classification of rocks from UCS forwarded by ISRM (1978) (Appendix 6), the rocks in the structural controlled slope section are classified as medium strong rock to very strong rock and the rocks in the nonstructural controlled slope section are classified as weak rock to medium-strong rock.

5.2.2 Rock Quality Designation (RQD)

The RQD of rocks in the study area was estimated by adopting the Volumetric Joint Method. In this study, RQD was only determined for rocks outcropped in the structural controlled where joint sets are persistent. The rocks in the non-structural controlled sections, on the other hand, are dominantly affected by irregular fractures rather than systematic joint sets. Accordingly, for rocks of structural controlled slope sections, Volumetric Joint Count (Jv)

was first determined using Eq. 2.6. Afterward, the RQD of rocks was determined using Eq. 2.7 suggested by Palmstrom (2005). The result showed that the ignimbrite at the slope section RSS₁, RSS₂, and RSS₃ have RQD values in the order of 80.68 %, 70.10 %, and 79.50 %, respectively (Table 5.3).

Table 5. 3. RQD of rocks of the study area at structural controlled slope sections.

Slope Section	Jv	RQD (%)	Deer and Deere(1988) RQD Classification
RSS ₁	11.72	80.68	Good
RSS ₂	15.97	70.10	Good
RSS ₃	12.22	79.5	Good

5.2.3 Unit Weight of Rocks

Unit weight of the rock is one of the major parameters in slope stability analysis. Moreover, it is one of the parameters during the determination of UCS of intact or from Schmidt hammer rebound value. In this study, the unit weight of rocks in the study area was determined using the buoyancy technique in the laboratory. Accordingly, the test result showed that the unit weight of rocks in the study area varies from 22.07 kN/m³ to 27.96 kN/m³ (Table 5.4).

Table 5. 4 A sample of the Unit weight of rocks in the study area (Remaining data are given in Appendix1)

Slope section	Rock Type	Structural Controlled						
		Unit weight	Mass(g)	V1(ml)	V2(ml)	V2-V1 (ml)	$\rho =$ (g/cm ³)	Average γ (N/m ³)
RSS1	Ignimbrite	γ_{dry}	133.480	300.000	360.000	60.000	2.225	21.240
			145.650	300.000	368.000	68.000	2.1419	
		γ_{sat}	135.010	350.000	398.000	48.000	2.813	25.783
			122.160	350.000	400.000	50.000	2.4432	
RSS2	Ignimbrite	γ_{dry}	162.920	300.000	370.000	70.000	2.310	22.712
			150.200	298.000	367.000	69.000	2.320	
		γ_{sat}	85.460	350.000	380.000	30.000	2.850	28.505
			115.460	380.000	419.000	39.000	2.961	
RSS3	Ignimbrite	γ_{dry}	142.600	320.000	381.140	67.140	2.125	22.589
			121.690	315.690	401.149	86.459	2.460	
		γ_{sat}	123.685	322.984	367.141	44.157	2.801	27.813
			116.230	364.100	404.600	40.500	2.869	

5.2.4 Rock Mass Rating (RMR)

To determine the quality of rock mass at structural controlled slope sections, Bieniawski's (1989) basic Rock Mass Rating (RMR) system was adopted. For the rocks of non-structural

controlled slope sections, however, RMR was not determined due to the inability to determine RQD and other discontinuity parameters in the field. The Schmidt hammer-based UCS of intact rocks (Table 5.2), Volumetric Joint Method derived RQD (Table 5.3), and field determined condition and spacing of discontinuities, and groundwater was used as input parameter following the guidelines of this system. The analysis of the result showed that ignimbrite rock at the slope sections RSS₁ and RSS₂ are classified as fair rock and the rock at RSS₃ is classified as good rock (Table 5.5).

Table 5. 5 Rock mass classification based on RMR system at structural controlled slope sections.

Parameters		Structural Controlled Slope Sections					
		RSS ₁	Rating	RSS ₂	Rating	RSS ₃	Rating
UCS (MPa)		111.52	12	113.49	12	198.72	12
RQD (%)		80.68	17	70.10	13	79.50	17
Joint spacing (Cm)		26.00	10	19.30	8	27.33	10
Condition of Joints	Persistence(m)	15.00	1	> 20 m	0	> 20 m	0
	Aperture(mm)	26.45	0	8.30	0	3.00	1
	Filling	Fine soil >5mm	0	Fine soil >5mm	0	Fine soil <5mm	2
	Roughness	Rough	5	Rough	5	Rough	5
	Degree weathering	Moderately Weathered	3	Slightly Weathered	5	Slightly Weathered	5
Groundwater Condition.		Damp	10	Damp	10	Damp	10
RMR		58		53		62	
Class		Fair Rock		Fair Rock		Good Rock	

5.3 Slope Stability Analysis of Rock Slope (Structural Controlled)

5.3.1 Kinematic Analyses

Based on field investigation, three slope sections namely RSS₁, RSS₂, and RSS₃ are classified as structural controlled slope sections. The stability analysis of these slope sections is first determined via kinematic analysis using pole data stereographic projections to determine their corresponding mode of failure. This analysis was done using Dips 6.0 software (Dips 6.0 Rocscience, 2004) based on procedures forwarded by Hoek & Bray (1981) and Wyllie & Mah (2004) from input parameters listed in Table 5.6. The input data for the analysis such as the orientation of discontinuities and slope face were determined in

the field while the friction cone (Appendix 4) was determined using Rockdata 3.0 software (Rocdata 3.0 Rocscience, 2004).

Table 5. 6 Input parameters for kinematics analysis.

Slope sections	Joint set and its parameter						Slope face parameters		Friction cone
	JS1		JS2		JS3				
	D	DD	D	DD	D	DD	D	DD	
RSS1	55	064	25	344	44	82	80	75	19
RSS2	42	010	78	210	45	174	85	105	28
RSS3	63	036	20	343	65	79	80	068	32
D= Dip Amount, DD= Dip Direction									

From the analysis of the results, only two modes of failure namely planar and wedge mode of failures were identified. The planar mode of failure was identified in the Dip 6.0 software by determining the pole of joint sets that fall in the critical zone of planar failure as highlighted in red color in Figure 5.1. According to the analysis of the results, the planar mode of failure was only identified at slope section RSS1 where it occurred due to JS1 and JS3 (Figure 5.1 A).

The wedge mode of failure, on the other hand, was identified in the Dips 6.0 software based on the notion that the intersection of two joints should fall in the critical zone of wedge failure. Accordingly, this mode of failure was identified in slope sections RSS₂ and RSS₃. At RSS₂, the wedge has occurred due to the intersection of JS2 and JS3 along the trend direction of 128° (Figure 5.1 B and Table 5.7). Similarly, at RSS₃ the wedge mode of failure has occurred due to the intersection of JS2 and JS3 along 357° trend direction, and due to the intersection of JS1 and JS3 with potential failure toward 044°.

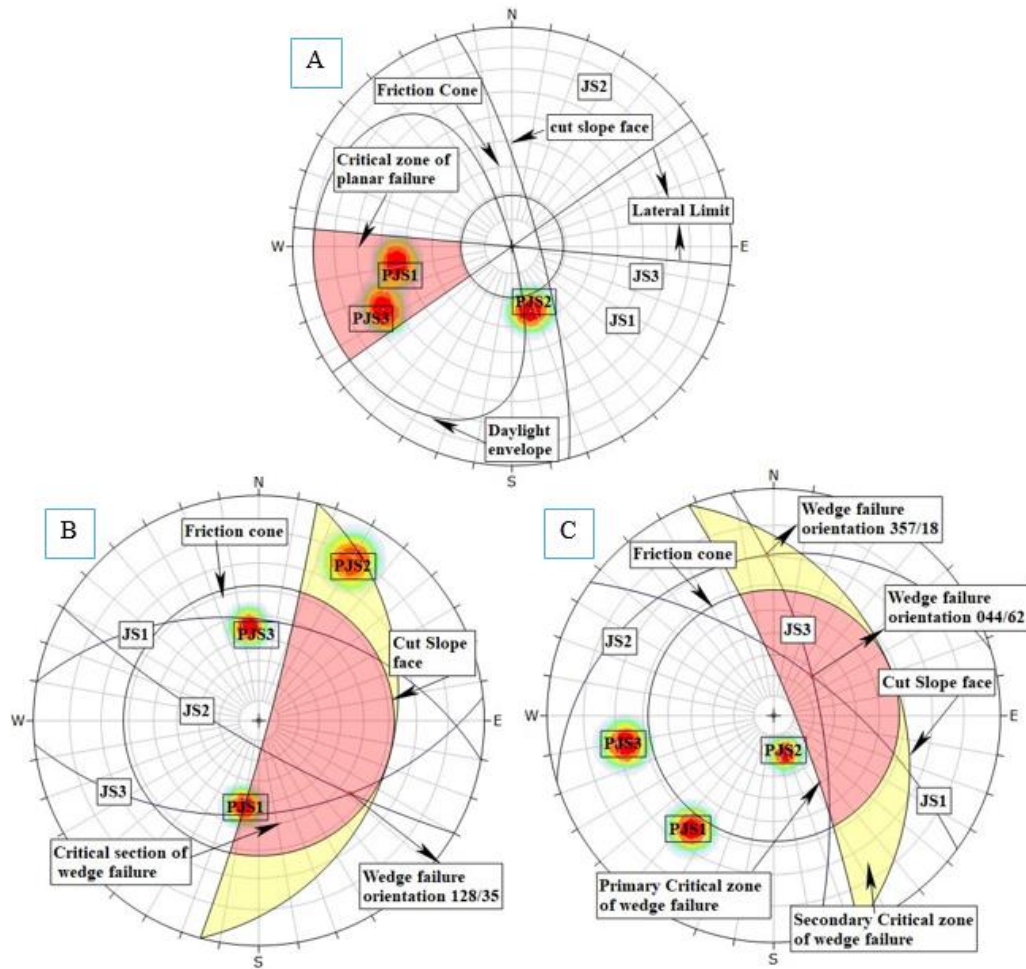


Figure 5. 1 Stereographic projection which shows a different mode of rock slope failures. A) Planar mode of failure at slope section RSS1 and, B and C) wedge mode of failure at slope section RSS2 and RSS3, respectively.

Table 5. 7 The result of the kinematic analysis showing the possible mode of failure.

Slope Sections	Possible mode failure based on kinematic analysis
RSS1	Planar mode of failure due to joint set 1 (JS1)
	Planar mode of failure due to joint set 3 (JS3)
RSS2	Wedge mode of failure due to the intersection of joint set JS2 and JS3
RSS3	Wedge mode of failure due to the intersection of joint set JS2 and JS3
	Wedge mode of failure due to the intersection of joint set JS1 and JS3

5.3.2 Rock Slope Stability Analysis Using LEM

The kinematic analysis revealed that the critical slope sections of the study area are unstable for planar and wedge modes of failure (Figure 5.1). This method, however, only provides the possible mode of failure/s without determining the stability of the slope in terms of FOS (Park and West, 2001). Based on this notion, further stability analysis was conducted by adopting LEM to determine the stability in terms of FOS.

5.3.2.1 Stability Analysis for Planar Mode of Rock Slope Failure

The result of the kinematic analysis (Figure 5.1 A and Table 5.7) showed that slope section RSS₁ is unstable for a planar mode of failure due to JS1 and JS3. Further stability analysis of this mode of failure in terms of FOS was then determined using LEM-based on Rocplane 2.0 software (Rocplane 2.0 Rocscience, 2004) based on the study by Hoek & Bray (1981) and Sharma et al. (1995). The analysis was conducted using input parameters listed in Table 5.8. The analysis was conducted under four anticipated conditions namely static dry, dynamic dry, static saturated, and dynamic saturated conditions. During the saturated conditions, a 100% filled mid-height peak pressure scenario was applied. Similarly, for the dynamic conditions, horizontal seismic acceleration of the study area (0.07g) was adopted from (Rawat & P, 2019).

Table 5. 8 Input parameters for RocPlane software based on Barton- Bandis (1990) failure criterion.

Slope section	Input parameters for RocPlane V. 2.0										
	JS	Slope geometry						Strength		Force	
		Sa (⁰)	Sh (m)	γ (t/m ³) d/s	Fpa (⁰)	W (⁰)	Ufa (⁰)	C (t/m ²)	Φ (⁰)	γ_w (t/m ³)	α_s
RSS1	JS1	80	55	2.166/2.629	55	2	10	1.734	34.03	1	0.07
	JS3	80	55	2.166/2.629	44	1.8	25	2.753	30.91	1	0.07
	JS= Joint set, Sa= slope angle, Sh= slope height, γ = unit weight, Fpa= failure plane angle, W= waviness, Ufa=upper face angle, c= cohesion, Φ = friction angle, γ_w = unit weight of water,s=saturated, d=dry α_s = seismic coefficient.										

The analysis of the result showed that JS1 of slope section RSS1 has a FOS of less than 1 under all anticipated conditions. The calculated FOS for this joint as determined via Rocplane 2.0 software is in the order of 0.58, 0.14, 0.51, and 0.09 during static dry, static saturated, dynamic dry, and dynamic saturated conditions, respectively (Table 5.9). Under the worst scenario (i.e. dynamic saturated condition) in particular, the horizontal seismic force with a magnitude of 161.19 tons and mid-height peak water pressure of 1128.05 ton/m was applied to the failure plane (Figure 5.2). These forces have reduced the resisting force of the failure plane to 171.78 ton/m and increased the driving force to 1978.69 ton/m. As a result of such application of seismic and water pressure forces, the FOS has reduced from 0.58 during the static dry condition to 0.09 during this dynamic saturated condition. Furthermore, the analysis of the results also showed that the FOS of the saturated condition

is much lower than for the dry condition showing that pore water pressure significantly contributes to destabilizing this slope.

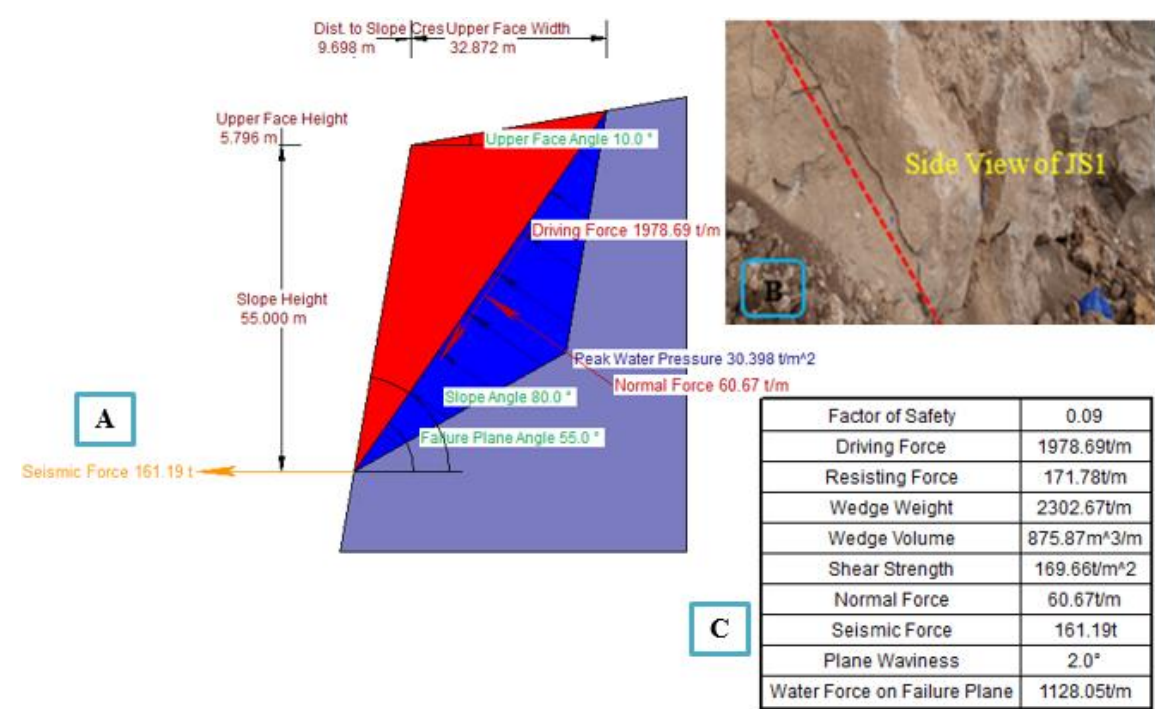


Figure 5. 2 A) 2D view of the planar mode of failure due to JS1 dipping 55° along RSS1 slope section, B) field photo showing discontinuity set and C) summary table of RocPlane 2.0 software for dynamic saturated condition.

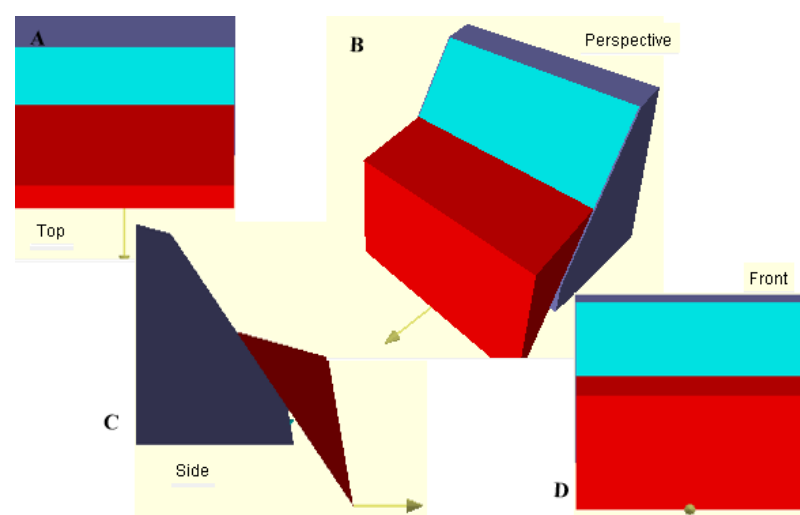


Figure 5. 3 Main 3D view of the planar mode of failure in RSS1 slope section due to JS1 under dynamic saturated

Similarly, the analysis of the results also manifest that JS3 of this slope section is also unstable under all static dry, static saturated, dynamic dry, and dynamic saturated conditions with FOS of 0.76, 0.23, 0.67, and 0.17, respectively (Table 5.9). Under the dynamic condition, the failure plane was subjected to 100 % saturation along the mid-height of the slope with a magnitude of 3429.09 ton/m and horizontal seismic force with a magnitude of 424.45 ton. This has resulted in a decrease in FOS from 0.76 during static dry to 0.17 during the dynamic dry condition with corresponding resisting and driving forces of 788.475 ton/m and 4517.39 ton/m, respectively (Figure 5.4).

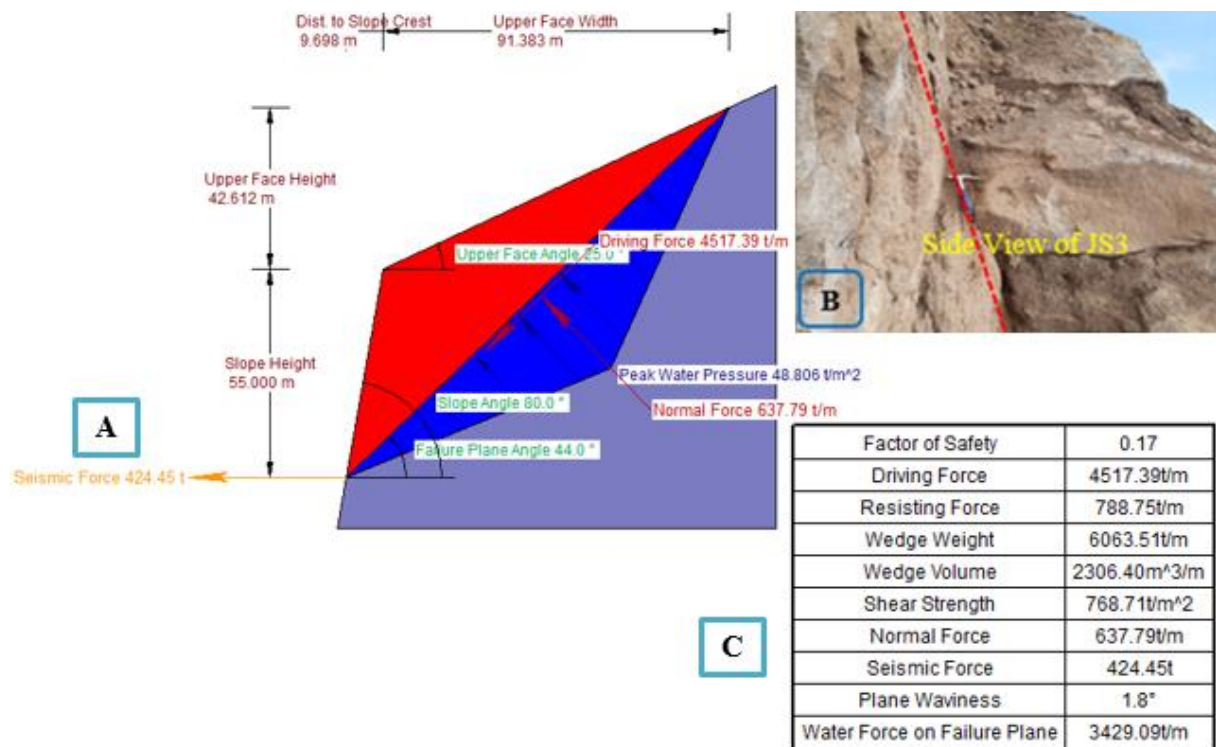


Figure 5. 4 A) 2D view of the planar mode of failure due to JS3 dipping 44° along RSS1 slope section under dynamic saturated condition, B) field photo showing discontinuity set and C) summary table of RocPlane 2.0 software.

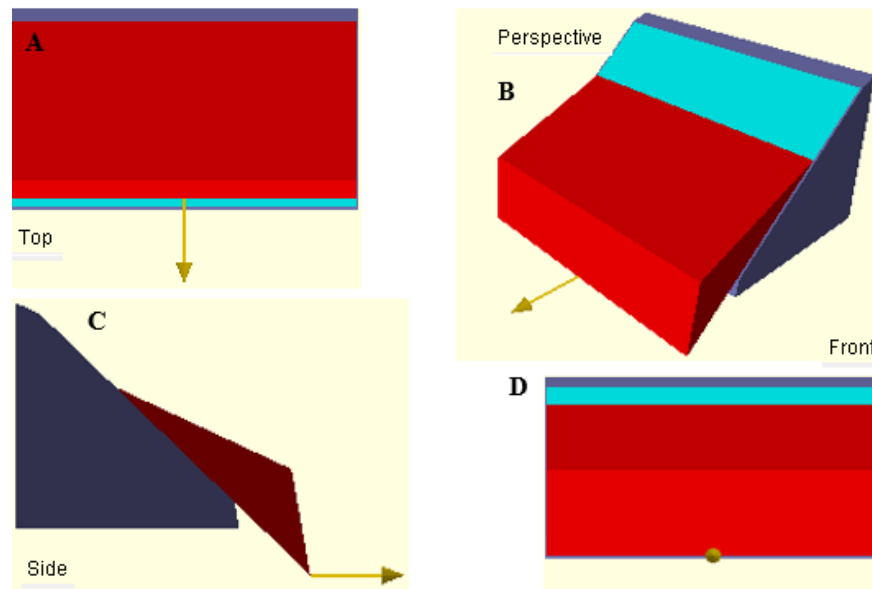


Figure 5. 5 Main 3D view of the planar mode of failure in RSS1 slope section due to JS3 under dynamic saturated condition.

Table 5. 9 Summary of FOS obtained for RSS1 under different anticipated conditions.

Slope section	JS	FOS result under different anticipated conditions			
		Static dry	Static saturated	Dynamic dry	Dynamic saturated
RSS1	JS1	0.58	0.14	0.51	0.09
	JS3	0.76	0.23	0.67	0.17

As shown in Table 5.9 above, the evaluated FOS of both JS1 and JS2 of RSS1 is less than 1 under all anticipated conditions. According to Wyllie and Norrish (1996), Wyllie & Mah (2004), Christian (2004), and Pauzi et al. (2014), FOS less than 1, greater than 1, and equal to 1 indicate unstable, stable, and slope in the brink of failure, respectively. Hence, based on these studies slope section RSS1 is unstable under all anticipated conditions of the study area for a planar mode of failure due to JS1 and JS3. This showed RSS1 is unstable under the current condition/or static dry conditions (i.e. without application of any amount of horizontal seismic acceleration and pore water pressure). According to Johnson & Degraff (1991) and Pauzi et al. (2014), the stability of such slope is dominantly controlled by factors such as failure plane shear strength parameters and slope geometry. Therefore, the low FOS of RSS1 is attributable to the low shear strength parameters of the discontinuities as compared to higher slope geometry and failure plane angle.

5.3.2.2 Stability Analysis for Wedge Mode of Rock Slope Failure

The stability analysis of structural controlled slope sections via kinematic analysis (Figure 5.1) showed that two slope sections namely RSS2 and RSS3 are kinematically admissible

for wedge mode of failure. Hence, further stability of this wedge mode of failure was carried out through the application of LEM using Swedge 4.0 software (Swedge 4.0 Rocscience, 2004). This software utilizes input parameters such as the orientation of discontinuities and slope (determined in the field), unit weight of the rocks (determined in the laboratory (Table 5.10), and shear strength parameters of failure plane (determined using Rockdata software using Barton & Bandis (1990) failure criteria (Appendix 4). Accordingly, the stability analysis was conducted using input parameters shown in Table 5.11 under four anticipated conditions.

Table 5. 10 Input parameters used for determination of FOS for Swedge software.

Slope section	Input parameters for Swedge V. 4.0									
	JS	Slope geometry					Strength		Force	
		JS D/DD(⁰)	Sh (m)	γ (t/m ³) s/d	Sa (D/DD ⁰)	UF (D/DD ⁰)	C (t/m ²)	Φ (⁰)	γ_w (t/m ³)	α_s
RSS2	JS2	78/210	50	2.907/2.316	85/105	25/105	3.977	36.51	1	0.07
	JS3	45/174	50	2.907/2.316	85/105	25/105	3.059	35.17	1	0.07
	JS1	63/36	57	2.836/2.303	80/068	12/195	3.671	36.83	1	0.07
RSS3	JS2	20/343	57	2.836/2.303	80/068	12/195	1.937	33.86	1	0.07
	JS3	65/79	57	2.836/2.303	80/068	12/195	5.608	37.94	1	0.07
	JS= joint set, Sh=slope height, γ =unit weight, s/d=saturated/dry, Sa=slope angle, UF=upper face, c=cohesion, Φ =friction angle, γ_w = unit weight of water, α_s = seismic coefficient.									

As revealed from kinematic analysis, slope section RSS2 is subjected to wedge mode of failure along trend direction of 128⁰ due to intersection of JS2 and JS3 (Figure 5.1). This slope section, in particular, is outcropped over a steeply sloped ridge (DA = 85⁰) and is constituted by a very strong ignimbrite. Further stability analysis of this slope using Swedge 4.0 software showed that this slope has FOS of 2.625, 0.812, 2.340, and 0.723 during static dry, static saturated, dynamic dry, and dynamic saturated, respectively (Table 5.11 and Figure 5.6). Hence, based on the interpretation of FOS given by Wyllie and Norrish (1996), Wyllie & Mah (2004), Christian (2004), and Pauzi et al. (2014), this slope section is unstable during the two saturated conditions. Moreover, as can be noted from the result, saturated conditions have significantly much lower FOS than dry conditions. According to Hossain (2011), Ahmadi & Eslami (2011), and Raghuvanshi et al. (2017), pore water pressure in the discontinuities reduces slope stability by lubricating and decreasing the shear strength of the discontinuities. Therefore, the low FOS of this slope section during saturated conditions is attributable to the water force.

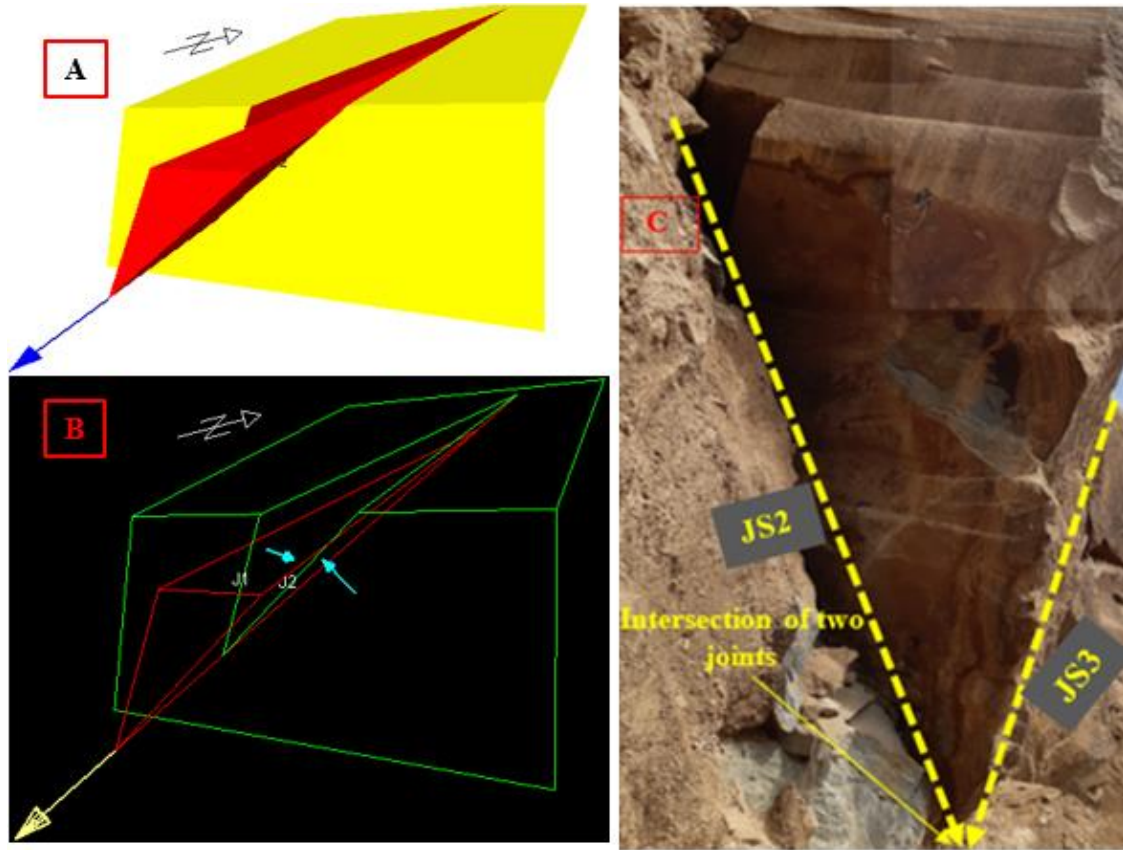


Figure 5. 6 A) 3D view of the slope geometry with shaded draw mode under dynamic saturated condition, B) 3D view of geometry with wireframe draw mode and C) field photo of slope section RSS₂.

Similarly, the kinematic analysis also showed that slope section RSS₃ is subjected to wedge mode of failure due to the intersection of JS₁ and JS₃, and JS₂ and JS₃ (Figure 5.7). Further stability analysis of wedge-formed due to intersection of JS₂ and JS₃ using Swedge software revealed the FOS which is greater than one under all anticipated conditions (Figure 5.7 and Table 5.11). The difference in the FOS between the worst condition (i.e. dynamic dry condition) and the current condition (i.e. static dry condition) is in the order of 0.46 indicating that seismicity has a limited influence on the stability of this wedge.

On the contrary, stability analysis of wedge-formed due to JS₂ and JS₃ has resulted in FOS which is less than under all anticipated conditions (Figure 5.8 and Table 5.11). Hence, this wedge of RSS₃ is unstable under all projected conditions including the current/or static dry condition. Following the works of Johnson & Degraff (1991), Pauzi et al. (2014), and Raghuvanshi et al. (2014), it can be deduced that the stability of this wedge of RSS₃ section is primarily controlled by factors other seismicity and water pressure such as slope geometry and failure plane characteristics.

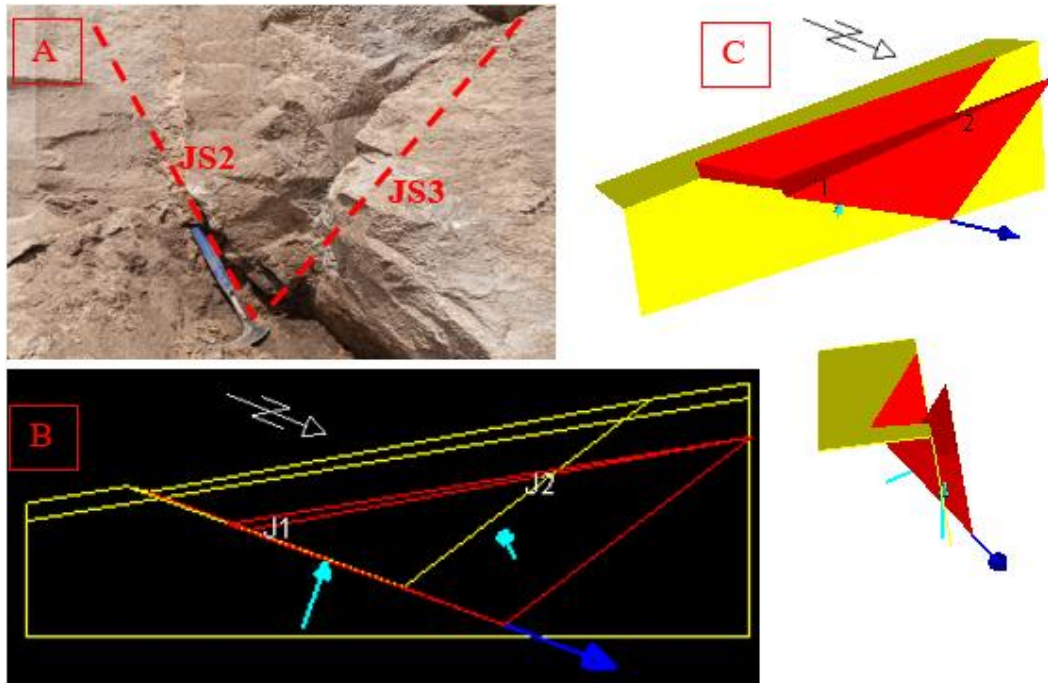


Figure 5. 7 A) Field photo of slope section RSS₃ illustrating wedge due to JS2 and JS3, B) 3D view of geometry with wireframe draw mode and C) 3D view of the slope geometry with shaded mode

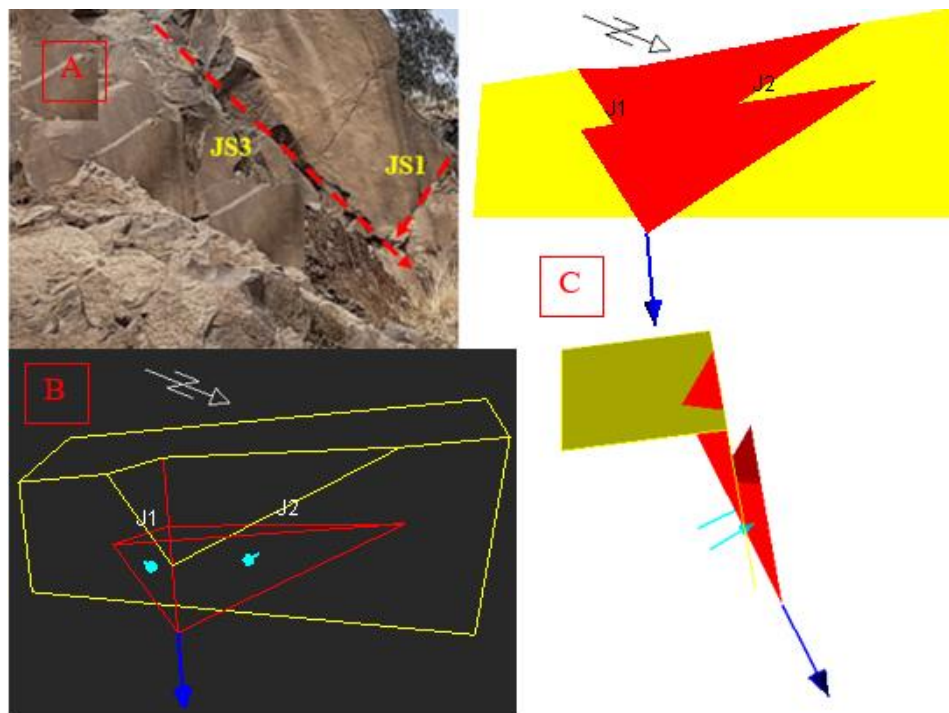


Figure 5. 8 A) Field photo of slope section RSS₃ illustrating wedge due to JS1 and JS3, B) 3D view of geometry with wireframe draw mode under dynamic saturated condition and C) 3D view of the slope geometry with shaded mod under dynamic saturated condition.

Table 5. 11. Factor of safety under different anticipated conditions for wedge mod of failure.

Slope section	JS (intersection)	FOS result under different anticipated conditions			
		Static dry	Static saturated	Dynamic dry	Dynamic saturated
RSS2	JS2 & JS3	2.625	0.812	2.340	0.723
RSS3	JS2 & JS3	2.635	1.524	2.176	1.290
	JS1 & JS3	0.861	0.016	0.798	0.015

5.4 Engineering Characterization of Soil

The study area is mainly covered with a different types of soil with different properties. The variability of their property has a significant role in the stability of the slope. Accordingly, laboratory tests were conducted on the soil samples obtained from the critical slope section.

5.4.1 Unit Weight

The unit weight of soil is an important parameter in slope stability analysis. From this point of view, unit weight was determined for soils and pumice obtained from each test pit. The test result showed that the dry unit weight of soil in the study area ranges from 10.710 to 12.980 and the saturated unit weight of soil varies from 12.025 to 14.5. Similarly, the dry unit of pumice varies from 12.200 to 15.000 and the saturated unit of soil varies from 14.100 to 19.000.

Table 5. 12. Unit weight of soil and pumice of the study area.

Soil sample Code	$\gamma_{\text{dry}}/\gamma_{\text{sat}}$ (kN/m ³)	Pumice Sample	$\gamma_{\text{dry}}/\gamma_{\text{sat}}$ (kN/m ³)
S-1	12.900/14.200	Pumice A at SSS ₂	14.390/15.200
S-2	10.980/12.025	Pumice B at SSS ₃	14.010/15.630
S-3	12.650/13.960	Pumice C at SSS ₄	12.200/14.100
S-4	12.940/14.500	Pumice D at SSS ₄	14.200/15.023
S-5	10.71/13.72	Pumice E at SSS ₆	15.000/19.000
S-6	11.830/14.847		
γ_{dry} = dry unit eight and γ_{sat} = saturated unit weight.			

5.4.2 Grain Size Analysis

Grain size analysis tests were conducted to identify the type of soil for the soil which compose the critical slope section along the road section. The test was done on the different samples which were taken hold of from the test pits. Sieve analyses were done for soil particles that passed 75 μ m (No. 200 sieve) in diameter and hydrometer and hydrometer analysis for soil particles less than 75 μ m (No. 200 sieve) in diameter. Particle size distribution curve was plotted from the analysis result (Figure 5.9) and the percentage of soil particles were computed for each soil sample (Table 5.13).

Table 5. 13.The percentage of soil particles for each soil sample.

Sample code	Depth	Percentage (%)			
		Gravel	Sand	Silt	Clay
S-1	1.4-2.2	0	54.960	45.040	0
S-2	1.5-2.4	3.78	86.97	9.25	0
S-3	1.6-2.5	2.1	52.5	44.44	0.96
S-4	1.1-2.0	0	29.7	70.21	0.09
S-5	0.8-1.6	2.92	56.72	40.36	0
S-6	1.4-2.1	0	34.82	57.28	7.9

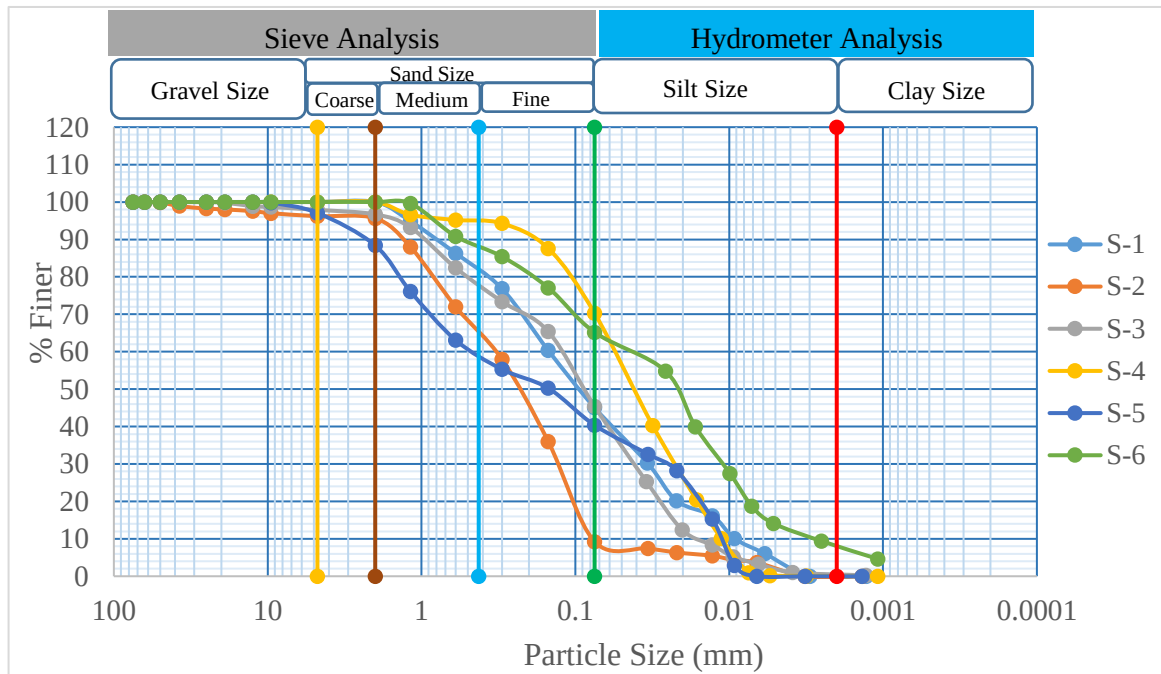


Figure 5. 9 Graph of the particle size distribution for gradation analyses.

5.4.3 Atterberg Limit

Liquid limit and plastic limit tests were conducted for each soil sample which passed 0.425mm sieve size after the sieve analysis test. The plasticity index was then calculated by using Eq.3.4.

Table 5. 14.Atterberg limit result of the soil of the critical slope section.

Sample- Id	Location			LL (%)	PL (%)	PI (%)
	E	N	elevation			
S-1	376863	971620	2995	39.03	29.52	9.51
S-2	37680.3	971378	3095	48.04	38.69	9.35
S-3	376659	971312	3062	41.81	32.43	9.38
S-4	376633	971476	3096	45.36	36.258	10.02
S-5	376987	971556	2975	39.98	26.48	13.5
S-6	377378	959518	2490	55.43	32.87	22.56

LL=liquid limit, PL=plastic limit, PI=plasticity index, N=northing, E= easting

Since the plasticity index of the soil of the slope section one, two, three. And five are between 7 and 17, the soil is categorized as medium plastic, and the soil which constitute the slope section six is categorized under highly plastic soil because the plasticity index is greater than 7. The plasticity of the soil makes the slope makes more problematic.

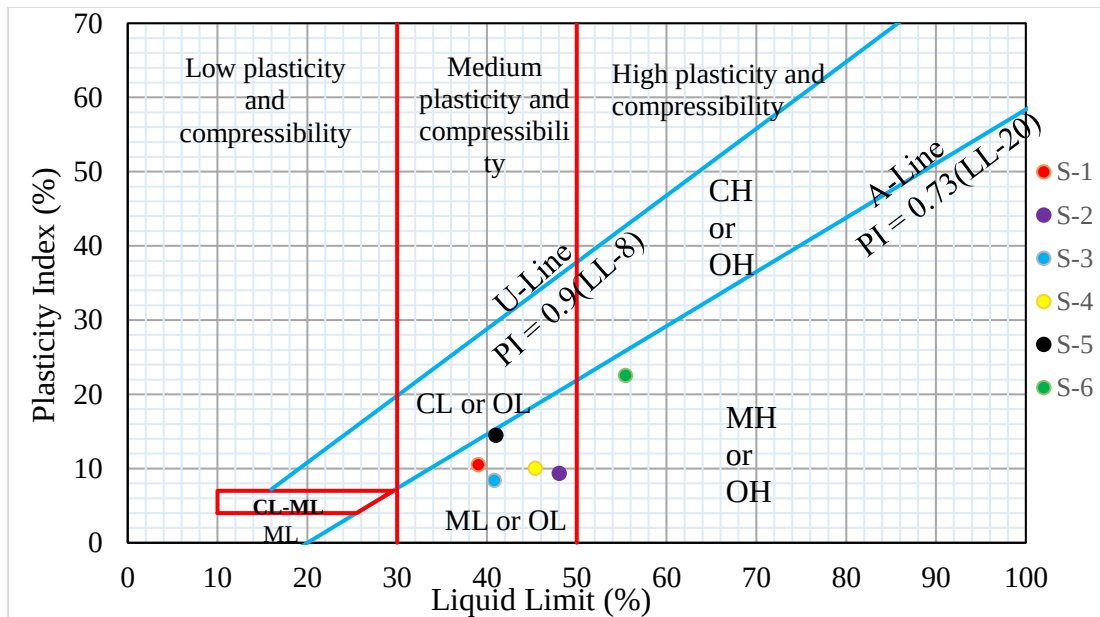


Figure 5. 10 The distribution of soil in the plasticity chart

5.4.4 Shear Strength Test

The shear strength test is very essential in the slope stability analysis because both the LEM and FEM-based software needs cohesion and friction angle as input parameters to calculate the factor of safety and stress reduction factor. The laboratory test implies that the friction angle of the soil ranges from 22.5° to 29° and the cohesion varies from 4.53 to 26 (Table 5.15).

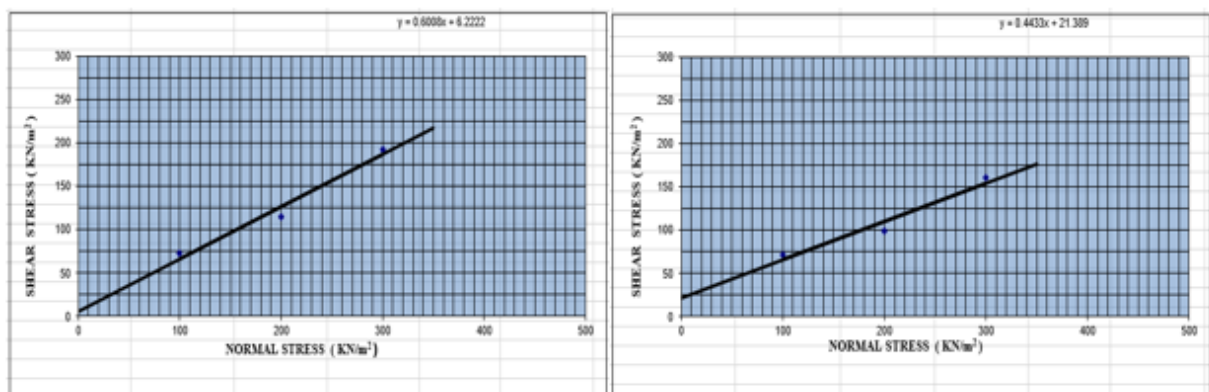


Figure 5. 11 A Sample of normal versus shear stress curve of direct shear test on the soil.

Table 5. 15 Shear strength parameters of the soils and pumice.

Sample ID	Soil Type	Shear Strength Parameters		Pumice ID	Shear Strength Parameters	
		C(kN/m ²)	ϕ (°)		C(kN/m ²)	ϕ (°)
S-1	Silty sand	5.000	29.000	P-A	9.000	34.000
S-2	Poorly-graded silty sand	4.530	27.800	P-B	4.590	32.200
S-3	Silty sand	6.000	29.500	P-C	4.200	30.950
S-4	Low Plastic Silt	5.670	29.000	P-D	4.570	28.300
S-5	Silty sand	21.000	23.910	P-E	4.800	30.200
S-6	High plastic silt	26.000	22.500			

5.4.5 Soil Classification

From grain size analysis and Atterberg limit tests soil of the study area along the road section was classified as shown in Table 5.16. The type of soil was found to be A-3, A-4, A-5, A-6 and A-7-5 according to the AASHTO classification system. Based on the USCS soil classification system, the soil of the slope section SSS₁ and SSS₂ at the top and also at SSS₅ is classified as silty sand, and the soil at the bottom of SSS₁ classified as poorly graded silty sand. The soil at SSS₃ and SSS₆ classified as low plastic silt and high plastic silt respectively. This implies that the percent of clay soil in the study area was small while that of sand and silt was large. In sandy soils pore water pressure quickly develops mainly when the slope was approached failure (Wang and Sassa, 2003; Chen and Lau, 2014). Therefore, failure of soil slope at SSS₁, SSS₃ and SSS₅ may be due to pore water pressure developed in the soil mainly during the rainy season, shallow groundwater level, and poor drainage system maintained along the road sections.

Table 5. 16 Types of soil classified according to AASHTO and USCS.

Slope Section	Sample Name	Sieve No	Percent Passing (%)	Atterberg Limit		coefficient		classification	
				LL	PI	CC	CU	AASHTSO	USCS
SSS ₁	S-1	4	100.000	39.030	9.510	0.674	16.840	A-4 (2)	SM (Silty Sand)
		10	100.000						
		40	82.100						
		200	45.040						
	S-2	4	96.220	48.040	9.350	0.766	4.000	A-3	SP-SM
		10	95.650						
		40	65.800						
		200	9.250						
SSS ₂	S-3	4	97.900	41.810	9.380	0.740	6.320	A-5 (2)	SM
		10	96.700						
		40	78.100						
		200	45.400						
SSS ₃	S-4	4	100.000	45.360	9.102	1.110	4.460	A-5 (8)	ML (low plastic silt)
		10	100.000						
		40	94.250						
		200	70.300						
SSS ₅	S-5	4	97.080	39.98	13.50	0.935	46.150	A-6 (2)	(SM) Silty Sand
		10	88.400						
		40	58.100						
		200	40.360						
SSS ₆	S-6	4	100.000	55.430	22.560	1.540	13.230	A-7-5 (15)	MH (High Plasticity Silt)
		10	100.000						
		40	88.120						
		200	65.180						

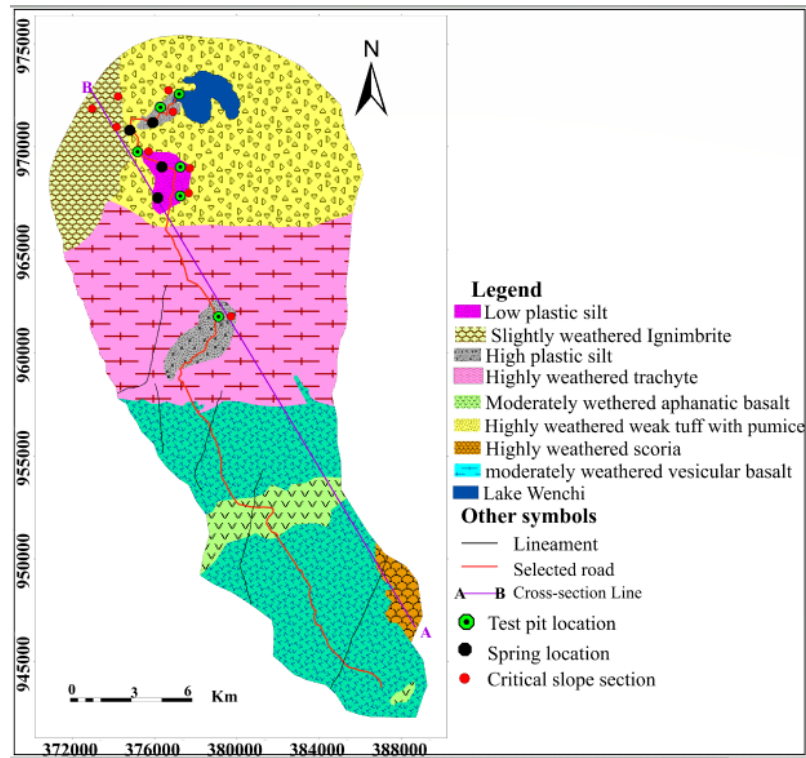


Figure 5. 12 Engineering geological map of the study area.

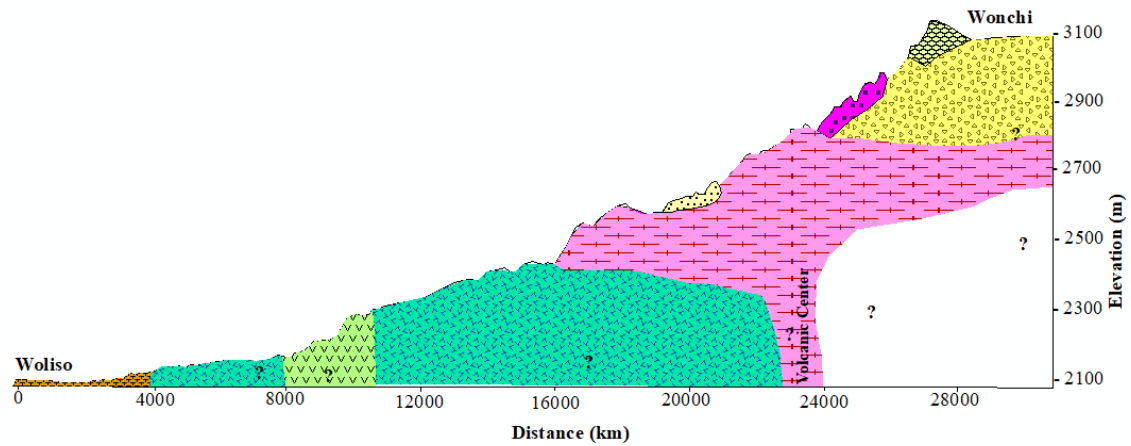


Figure 5. 13 Engineering geological cross-section.

5.5 Stability analysis of Soil and Highly weathered Rock (Non-Structural Controlled)

Based on field observation, about six non-structural controlled critical slope sections were identified. These slope sections were constituted by soil and highly weathered rocks such as tuff, pumice, and basalt. The stability analysis of these slope sections was determined using LEM-based Slide 6.0 and FEM-based Phase 2.0 software under static dry, dynamic dry, static saturated, and dynamic saturated conditions. In addition, PLAXIS software was also utilized for the analysis of saturated. Finally, the results obtained from different methods are compared to each other.

5.5.1 Stability Analysis Using Limit Equilibrium Method (LEM)

As stated earlier, the stability analysis using LEM was done via Slide 6.0 software. To determine the stability of the slope, this software uses slope geometry, unit weight, and shear strength parameters of slope materials as input parameters. In this study, the slope geometry was directly determined in the field, and the shear strength parameters of soils and unit weight of both rocks and soils were determined in the laboratory (Tables 5.4 and 5.15). The shear strength parameter of rocks was, however, determined using Rocdata software (Rocdata 3.0 Rocscience, 2004) following generalized Hoek-Brown Failure criteria (Hoek et al., 2002). Upon determination of these input parameters, the analysis was carried out using the Spencer method under four projected conditions from input parameters listed in Table 5.17. The interpretation of FOS obtained from the analysis was then interpreted as per Wyllie and Norrish (1996), Wyllie & Mah (2004), Christian (2004), and Pauzi et al. (2014) which stated the FOS less than 1, greater than 1 and equal to 1 indicate unstable, stable and slope in the brink of failure, respectively.

Table 5. 17 Input Parameters for the Slide 6.0 Software.

SSS	material	C (KN/m ²)	ϕ (°)	γ _{dry} (KN/m ³)	γ _{sat} (KN/m ³)	Slope height (m)	Slope angle (°)
SSS ₁	SM	5.000	29.000	12.900	14.200	38.000	58.000
	SP-SM	4.530	27.800	10.980	12.025		
SSS ₂	SM	6.000	29.500	12.650	13.960	82.000	42.000
	Pumice A	9.000	34.000	14.390	15.200		52.000
	SWT	201.912	27.510	20.020	21.950		45.000
	Pumice B	4.590	32.200	14.010	15.630		
	MWT	226.93	30.370	19.900	22.300		
SSS ₃	ML	5.670	29.000	12.940	14.500	38.000	39.000
	SWT	130.773	33.590	21.500	22.600		
SSS ₄	SWT	140.000	26.670	24.900	27.000	75.000	79.000
	Pumice C	4.200	30.950	12.200	14.100		45.000
	MWT1	169.880	27.050	23.100	26.350		
	Pumice D	4.570	28.300	14.200	15.023		
	MWT2	217.560	30.700	23.100	26.350		
SSS ₅	SM	21.000	23.910	10.710	13.720	50.000	46.000
	HWB	7407.000	2.820	23.000	24.210		52.000
SSS ₆	MH	26.000	22.500	11.830	14.847	78.000	49.000
	Pumice E	4.800	30.200	15.000	19.000		
	SWT	126.000	19.370	22.760	26.346		
c=cohesion, ϕ=friction angle, γ _{dry} =dry unit weight, γ _{sat} =saturated unit weight, H=height, ML= low plastic silt, SM=silty sand, SP-SM= poorly-graded silty sand, MH=high plastic silt, SWT=slightly welded tuff, MWT = medium welded tuff, HWB=highly weathered basalt.							

The slope section SSS₁ dips at an angle approximately equal to 58° and has a height of 38 m. During the field investigation, two different soils with slightly variable colors and textures were identified and hence, two samples were taken from this slope section. According to the USCS system, these soils were classified as silty sand and poorly-graded silty sand soils. The stability analysis of this slope was executed from input parameters shown in Table 5.17 and the results of the analysis (i.e. factor of safety) and inferred failure surface parameters were presented in (Table 5.18) and (Figure 5.14).

Table 5. 18 LEM-based Slide 6.0 software results for SSS₁ by using the spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic Saturated
FOS	0.649	0.585	0.221	0.169
RM (kN-m)	15472.5	14620.200	3367.22	2683.940
DM(kN-m)	23858.100	25003.100	15245.5	15911.10
RHF(kN)	3033.448	275.336	76.262	63.0404
DHF(kN)	467.909	470.874	345.286	373.720
C	43.566, 45.058	44.823, 46.315	37.283, 36.261	36.026, 36.261
R	32.009	33.575	22.162	22.0210

RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.

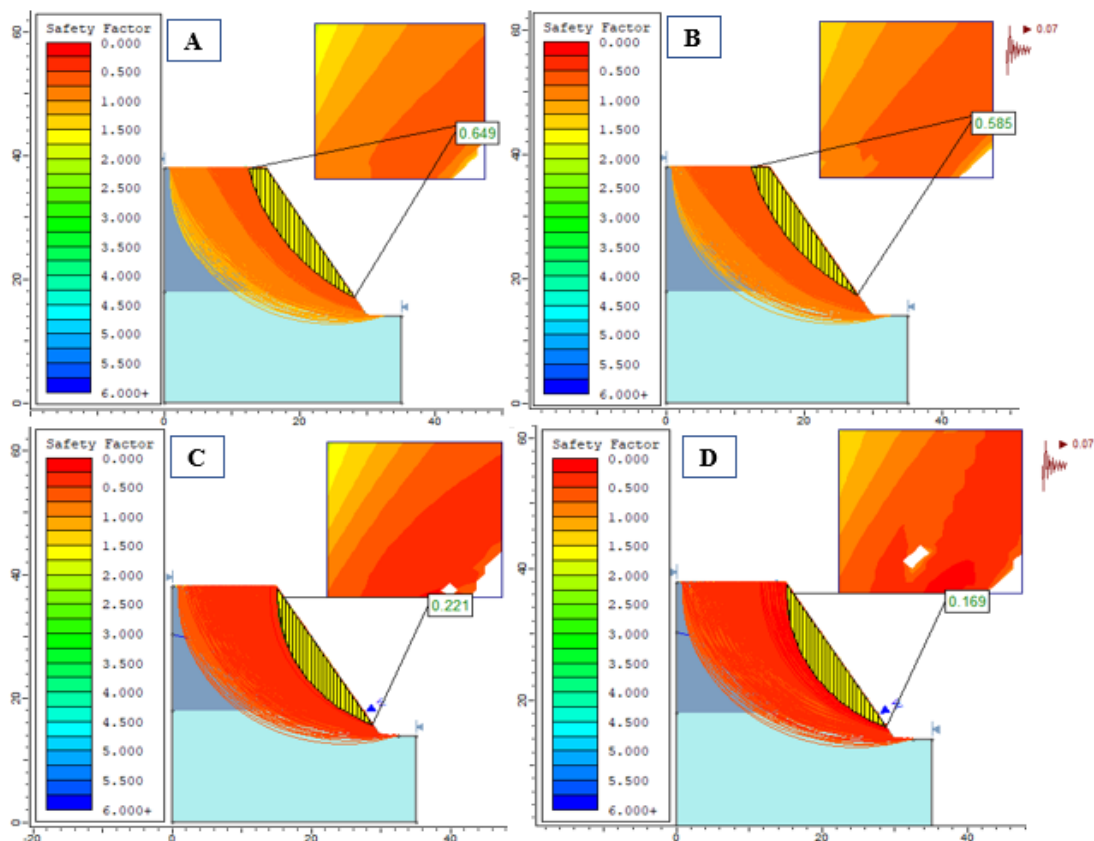


Figure 5. 14 Slope model and FOS of SSS₁ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated.

The analysis of the results showed that the minimum FOS matching the critical slip surface for SSS1 is in the order of 0.649, 0.585, 0.221, and 0.169 under static dry, dynamic dry, static saturated, and dynamic saturated conditions, respectively. This showed that this slope section is unstable under all anticipated conditions. Moreover, as can be noted from the results, the minimum FOS was obtained during a dynamic saturated condition which resulted when the resisting force of 63.04 kN and driving force of 373.72 kN were applied to the slope (Table 5.18).

The slope section SSS₂, on the other hand, has a height of approximately 82 m and is multi-sloped having three slope profiles (i.e. the bottom, middle, and top slope profiles). The bottom profile is sloped at an angle of 45° while the middle and top profiles are sloped at an angle of 52° and 42°, respectively. This slope is successively constituted from bottom to top by medium welded tuff, pumice, slightly welded tuff, pumice, and silty sand. Both pumice and tuff rocks are dipping toward the road cut at an angle of approximately 30°. Upon determination of the requirements for all aforementioned geological materials, the stability analysis was carried out and the corresponding results are given in Table 5.19 and Figure 5.15 below.

Table 5. 19 LEM-based Slide 6.0 software results for SSS₂ by using the spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic Saturated
FOS	1.049	0.969	0.461	0.410
RM (kN-m)	18298.200	17635.700	17244.700	16098.900
DM(kN-m)	17448.600	18387.000	37420.200	40013.900
RHF(kN)	366.486	356.900	293.120	279.482
DHF(kN)	349.470	372.100	636.060	694.652
C	38.801, 82.817	38.801, 82.817	38.801, 91.777	38.801, 91.777
R	34.532	34.532	43.118	43.118
RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.				

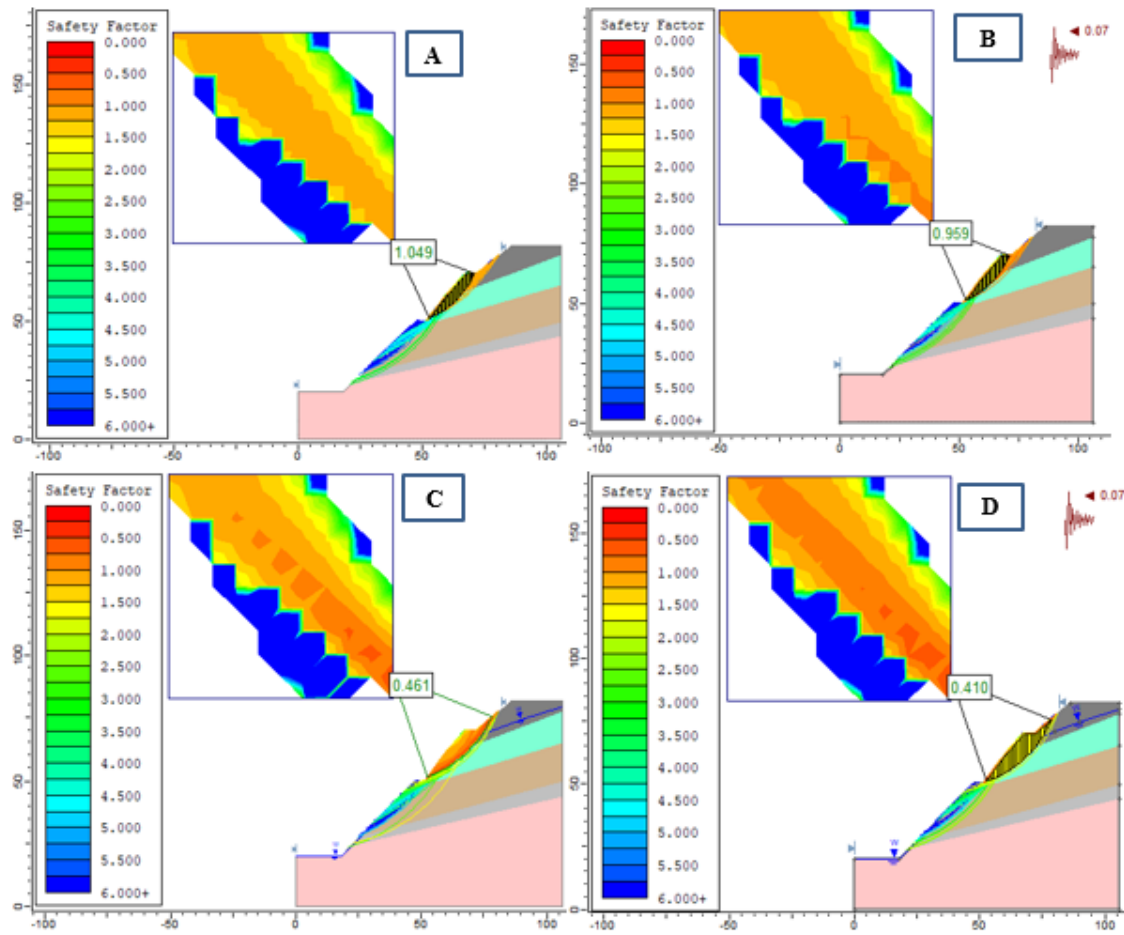


Figure 5. 15 Slope model and FOS of SSS₂ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated.

The analysis of obtained results showed that this slope section (i.e. SSS₂) is marginally stable during static dry conditions with a FOS of 1.049 (Table 5.19 and Figure 5.15A). During the dynamic dry conditions, however, this slope is found to be unstable with a FOS of 0.969 and a potential failure surface only involving the middle slope failure (Figure 5.15 B). Similarly, this slope is found to be unstable during static saturated and dynamic saturated conditions with FOS of 0.461 and 0.410, respectively. During these two conditions, the potential failure affected both the top and middle slope profiles (Figure 5.15 C and D) indicating saturation has a large capacity to destabilize a large number of materials on this slope. As shown in Table 5.19, a staggering 636.03 kN and 694.6 kN driving horizontal forces were applied to this slope during static saturated and dynamic saturated conditions which significantly contributed to destabilizing the slope.

The slope section SSS₃ on the other hand is constituted by low plastic silt soil type and horizontally layered slightly welded tuff rock. This slope has a height of approximately 38

m and dips at an angle of 39° . The results of the stability analysis of this slope are given in Table 5.20 and Figure 5.16 below.

Table 5. 20 LEM-based Slide 6.0 software results for SSS₃ by using the spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic Saturated
FOS	1.099	0.980	0.769	0.684
RM (kN-m)	19509.700	21885.100	9714.220	12458.300
DM(kN-m)	17746.800	22320.400	12633.800	18216.400
RHF(kN)	483.681	450.258	411.336	482.702
DHF(kN)	439.976	459.213	534.964	705.805
C	41.062, 53.795	44.821, 60.060	32.290, 40.011	32.290, 42.517
R	32.493	39.443	18.237	20.693

RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.

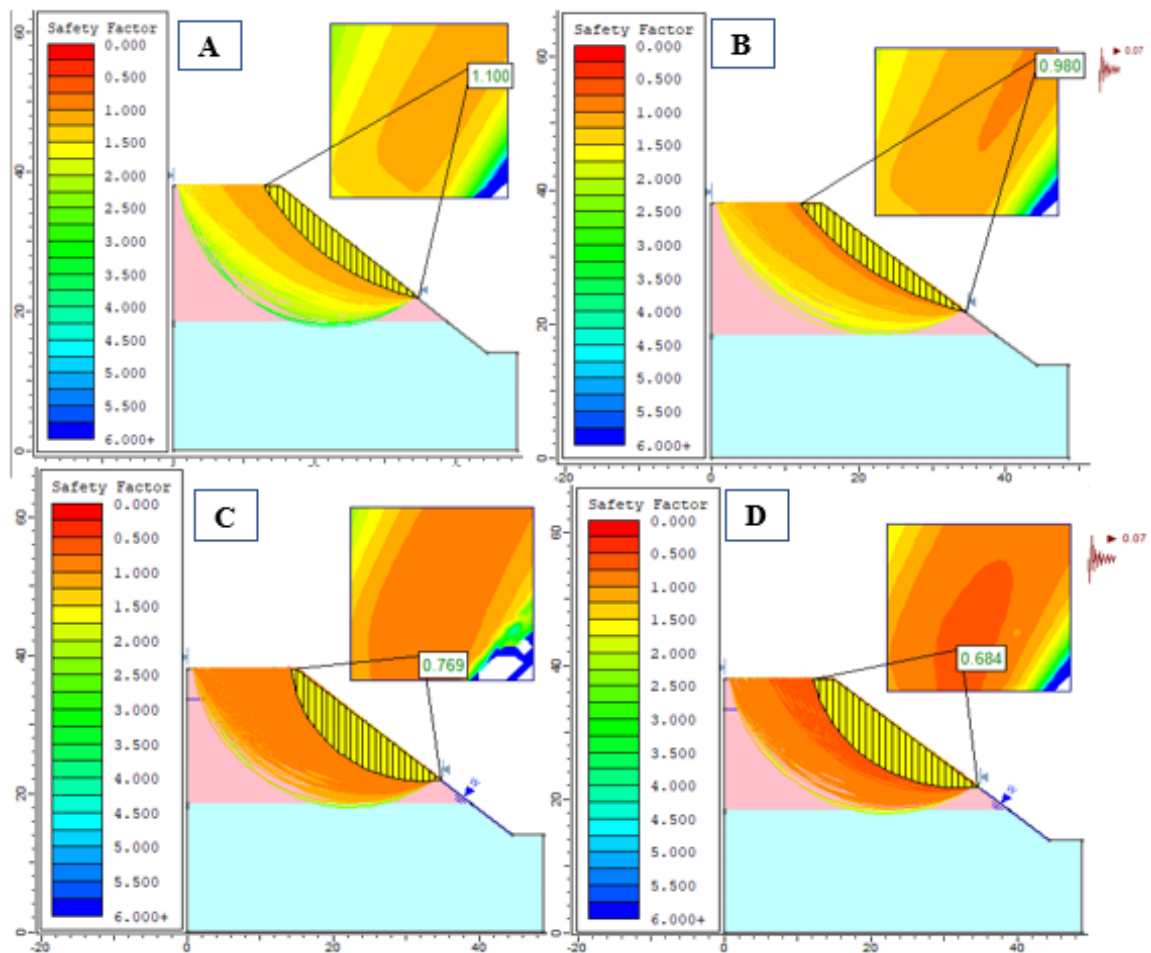


Figure 5. 16 Slope model and FOS of SSS₃ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated.

According to the results, this slope section is stable only during static dry conditions with a FOS of 1.100. (Table 5.20 and Figure 5.16 A). During the remaining conditions, this slope

is found to be unstable with potential failure only affecting the low plastic silt exposed at the top of the slope. As can be noted from the results, the FOS is significantly lower during saturated conditions showing that water pressure plays a great role in destabilizing the slope in the study area.

Another critical slope section, SSS₄, is located along the road from Haro Wonchi town to Wonchi Lake and is characterized by its 75 m height and two slope profiles (namely the top and bottom slope profiles). The top slope profile dips at an angle of 79° and is entirely constituted by slightly welded tuff which dips approximately at an angle of 24°. Similarly, the bottom slope profile dips at an angle of 45° and it is successively composed of pumice, slightly welded tuff, pumice, and slightly welded tuff from top to bottom which is all dipping approximately at an angle of 20°. After the determination of input parameters (Table 5.17), the stability analysis was carried out and the results are given in Table 5.21 and Figure 5.17 below.

Table 5. 21.LEM-based Slide 6.0 software results for SSS₄ by using the Spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic Saturated
FOS	1.069	0.877	0.591	0.414
RM (kN-m)	1.04147e+006	842385	553030	196070
DM(kN-m)	1.133e+006	960303	935441	473993
RHF(kN)	10747.300	9220.680	6270.87	2120.23
DHF(kN)	9808.240	10511.400	10607.87	5125.600
C	3.810, 86.835	21.116, 73.856	16.789, 73.856	29.768, 73.856
R	75.954	57.846	60.271	45.455
RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.				

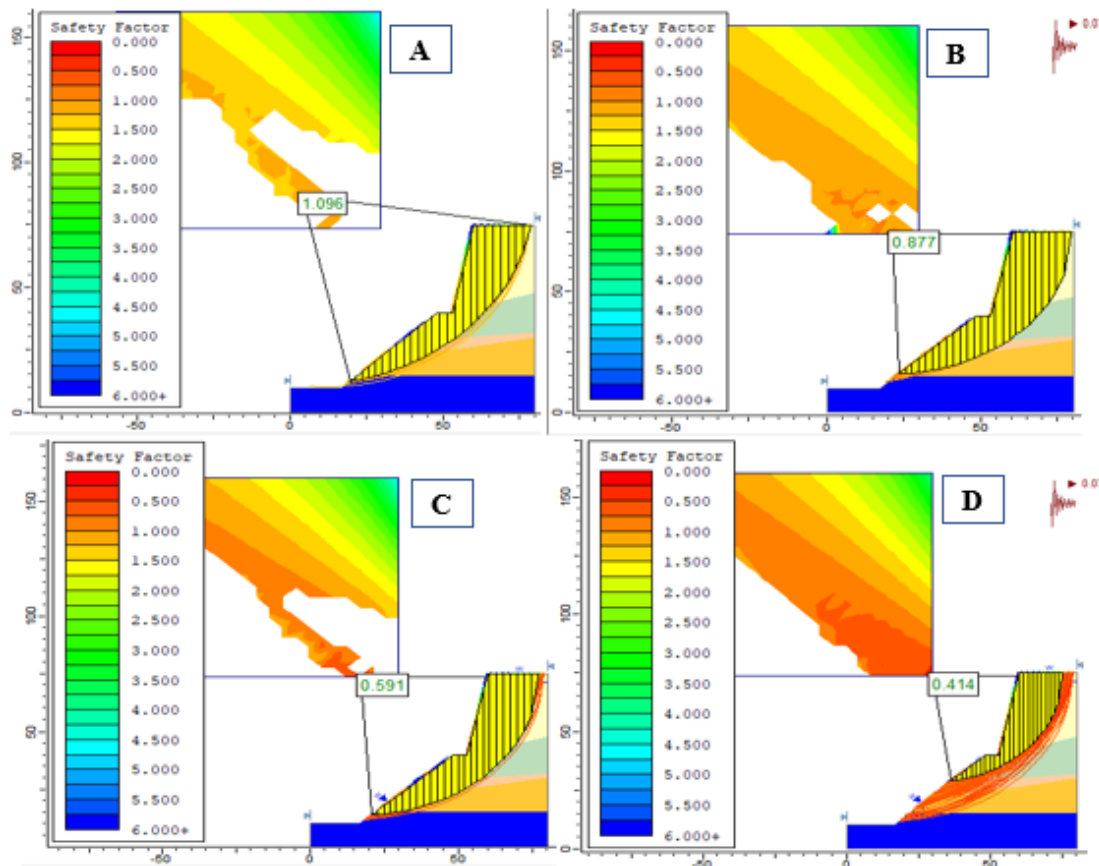


Figure 5. 17 Slope model and FOS of SSS₄ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated.

Based on the field manifestation, this slope section is one of the most critical with a steeply sloping top slope profile. According to the results, this slope is found to be stable during the static dry conditions and unstable during all remaining conditions (Table 5.21 and Figure 5.17). The potential failure corresponding to the minimum slip surface/or FOS affects both slope profiles during all anticipated conditions. This showed that the pumice layer (considered a weak layer and exposed on both slope profiles) has significantly contributed to destabilizing both slope profiles as shown in Figure 5.17 above.

Slope section SSS₅ is located along the road from Haro Wonch to Woliso town and has a height of 50 m. It has two slope profiles (i.e. multi-sloped). The top slope profile of this slope is dipped at an angle of 46° and is constituted by silty sand while the bottom slope profile is dipped at an angle of 52° and is composed of horizontally layered highly weathered basalt. The stability analysis results of these slope sections are also illustrated in Table 5.22 and Figure 5.18 below.

Table 5. 22 LEM-based Slide 6.0 software results for SSS₅ by using the spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic saturated
FOS	1.296	1.181	0.681	0.614
RM (kN-m)	35972.700	35421.300	18690.5	18245.100
DM(kN-m)	27761.600	29999.700	27440.0	29710.300
RHF(KN)	976.650	960.485	598.810	591.206
DHF(KN)	753.720	813.472	879.130	962.716
C	31.31, 43.66	31.31, 43.66	27.23, 39.58	27.23, 39.58
R	28.824	28.824	24.470	24.470

RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.

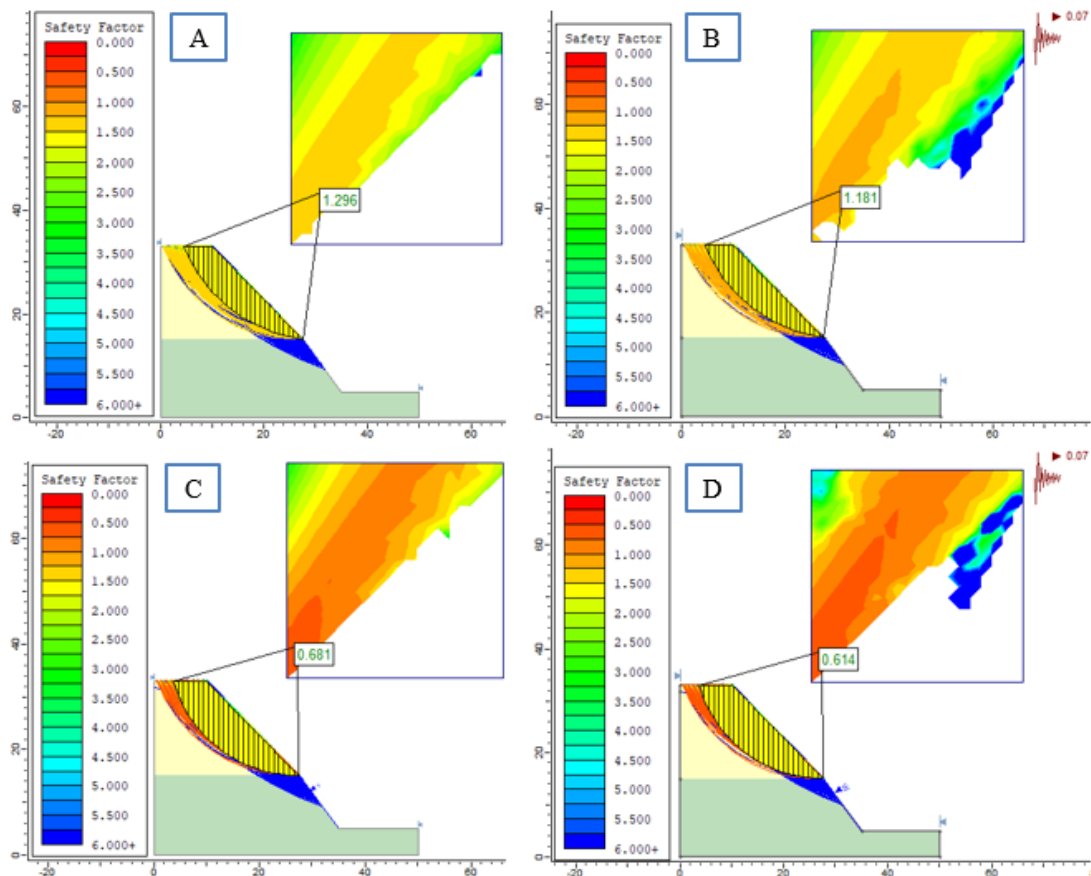


Figure 5. 18 Slope model and FOS of SSS₅ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated, and D) dynamic saturated.

The analysis of the results showed this slope section is stable during dry conditions (i.e. static dry and dynamic conditions) with corresponding FOS of 1.296 and 1.181 (Table 5.22 and Figure 5.18). During the static and dynamic saturated conditions, however, the slope becomes unstable with corresponding FOS as low as 0.681 and 0.614 showing that the stability of this slope is dominantly controlled by water pressure. The analysis also showed that the failure of this slope only involves the top slope profile (i.e. silty sand) as highlighted

by yellow in Figure 5.18 above. As shown in Table 5.22, 27440.0 kN-m and 29710.3 kN-m driving moment forces were applied to this slope during static saturated and dynamic saturated conditions, respectively which contributed to instability during these conditions.

The last slope section considered for the analysis, SSS₆, has approximately a height of 78 m and is single sloped at an angle of 49°. It consists of a pumice layer that is dipped at an angle of 34° and sandwiched between the topmost high plastic silt and bottom slightly welded tuff. The results of the stability analysis of this slope are given in Table 5.23 and Figure 5.19.

Table 5. 23 LEM-based Slide 6.0 software results for SSS₆ by using the Spencer technique.

Global minimums	Spencer			
	Static Dry	Dynamic Dry	Static Saturated	Dynamic Saturated
FOS	1.235	1.064	0.777	0.687
RM (KN/m)	54790.700	32241.800	27516.300	26178.600
DM(KN/m)	44338.900	30309.200	35435.800	38080.200
RHF(KN/m)	989.860	729.898	591.664	564.766
DHF(KN/m)	801.025	686.148	761.952	821.526
C	45.261, 73.708	42.367, 62.130	42.367, 62.130	42.367, 62.130
R	43.611	32.003	32.003	32.003
RM=resisting moment, DM=driving moment, RHF=resisting horizontal force, DHF=driving horizontal force, C=center, R=radius.				

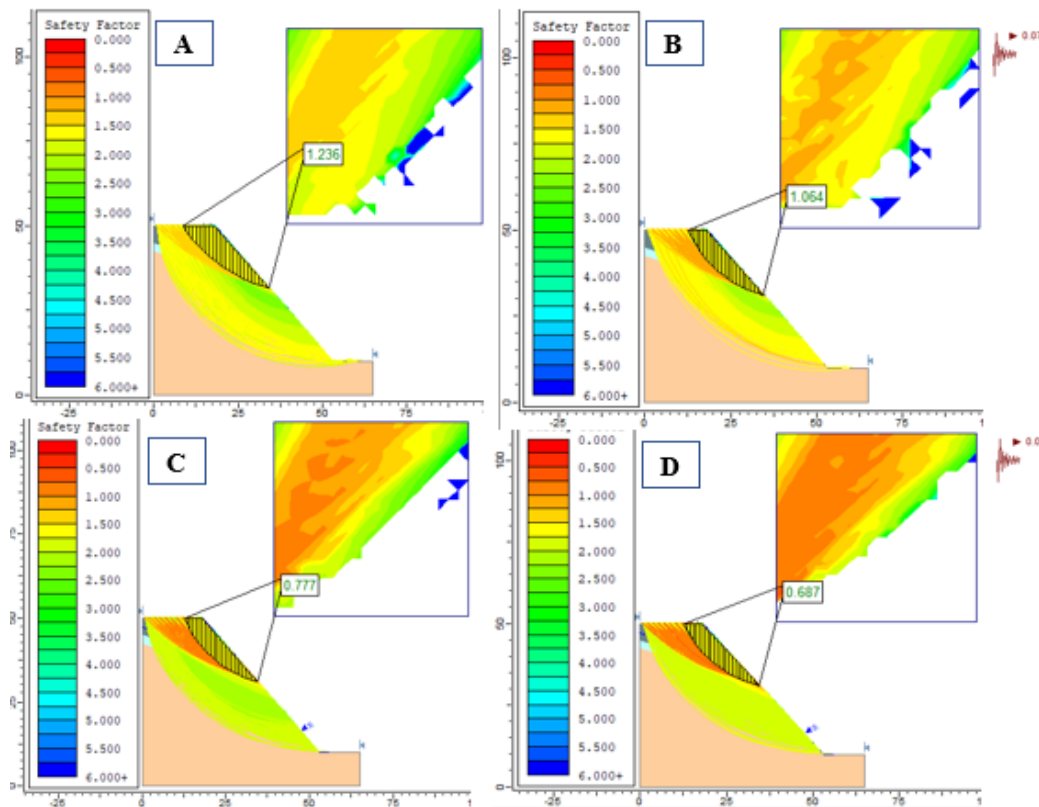


Figure 5. 19 Slope model and FOS of SSS₆ using Slide 6.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated.

Likewise SSS₅, the analysis results of this slope section showed that it is stable during dry conditions and unstable during saturated conditions (Table 5.23 and Figure 5.19). Similar to the results of other slope sections, the results of this slope section also showed the dominant influence of water pressure on the slope failure. Moreover, the analysis of this slope also revealed that the potential failure only involves layers above the pumice indicating that the pumice layer acts as a slipping surface.

Table 5. 24 Summary of FOS under different anticipated conditions.

Slope section	FOS result under different anticipated conditions			
	Static Dry	Static Saturated	Dynamic Dry	Dynamic Saturated
	Spencer	Spencer	Spencer	Spencer
SSS ₁	0.649	0.221	0.585	0.169
SSS ₂	1.049	0.969	0.461	0.410
SSS ₃	1.099	0.769	0.980	0.684
SSS ₄	1.069	0.591	0.877	0.414
SSS ₅	1.296	1.810	0.681	0.614
SSS ₆	1.236	0.777	1.064	0.687

In general, slope stability analysis of non-structural controlled sections using Slide 6.0 software revealed that all slope sections are unstable under saturated conditions. This

illustrates that rainfall can be a major triggering factor for slope failure and hence, any remedial should consider water pressure. The analysis also revealed that seismicity has little effect on the stability of slopes of the study area as FOS is only reduced in a very small amount from static dry to dynamic dry conditions (Table 5.24).

5.5.2 Stability Analysis Based on the FEM method

The stability of non-structural controlled slope sections is also further evaluated by using FEM-based Phase 2.0 software. During this analysis, the same slope geometry used in the Slide 6.0 software was modeled in this Phase 2.0 software. Moreover, the same input parameter used in slide software was used with exception of the addition of two parameters namely the Poisson ratio and Young's modulus. In this study, the Poisson ratio and Young's modulus of rocks and soils were determined as stated in sections 3.4.2 and 3.4.3 and are given in Table 5.25 below. Finally, the analysis was executed from the input parameters listed in Table 5.17 and Table 5.25 under four anticipated conditions to determine the Stress Reduction Factor (SRF). Similar to the LEM scenario, the interpretation of SRF was done based on the study forwarded by Wyllie & Mah (2004) and Christian (2004) & Pauzi et al. (2014).

Moreover, FEM-PLAXIS software was also briefly used to determine the stability of non-structural controlled slope sections for saturated conditions. The same input parameters used for Phase 2.0 software were used in this PLAXIS software.

Table 5. 25 Input parameters for finite element stability analysis to calculate critical SRF.

Section	Material	V	E (KPa)	SSS	Material	V	E (KPa)
SSS ₁	SM	0.160	22880.000	SSS ₄	SWT1	0.360	799617.330
	SP-SM	0.142	20304.000		Pumice1	0.1893	17152.200
SSS ₂	SM	0.20455	19838.440		SWT2	0.354	948105.590
	Pumice1	0.1675	24020.400		Pumice2	0.1495	21351.200
	SWT	0.467	1054101.850		SWT3	0.342	1432284.470
	Pumice2	0.208	20467.600	SSS ₅	SM	0.08365	25407.048
	CT	0.338	1516223.970		HWFB	0.336	890490.000
SSS ₃	ML	0.160	22880.000	SSS ₆	MH	0.0625	11700.000
	SWT	0.352	1116561.310		Pumice	0.178	15283.200
					SWT	0.394	302530.000

γ_{dry} = dry unit weight, γ_{sat} =saturated unit weight, ML=low plastic silt, SW=well-graded sand
SWT=slightly welded tuff, CT=consolidated tuff, HWFB=highly weathered and fractured basalt,
V=poission ratio, E=elastic modulus.

5.5.2.1 Stability Analysis Result of Phase 2.0 Software

As stated in section 5.6.2, FEM analysis was first executed using Phase 2.0 software from input parameters listed in Table 5.17 and Table 5.25. The analysis revealed that slope section SSS₁ is unstable during all anticipated conditions (Figure 5.20). The unstable nature of this slope even under static dry conditions showed that the shear strength parameters and slope geometry have considerable influence on the slope failure as illustrated by Pauzi et al. (2014). Moreover, the SRF of this slope is abruptly decreased to 0.198 and 0.143 during static saturated and dynamic saturated conditions demonstrating further aggravation of slope failure during saturation conditions (Figure 5.20).

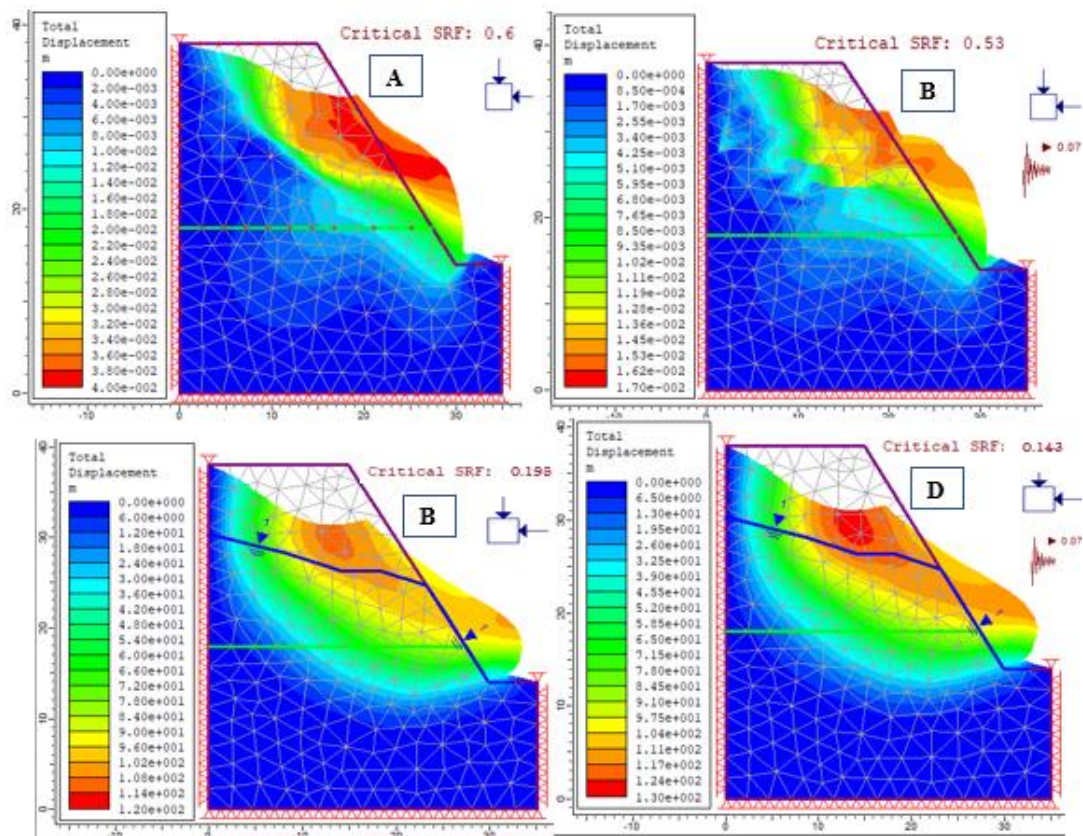


Figure 5. 20 SRF and slope mode of failure SSS₁ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

Moreover, the stability analysis results of SSS₂ showed that this slope is marginally stable under static dry conditions (i.e. SRF = 1.04) and unstable under all remaining conditions (Figure 5.21). There is a notable reduction in SRF under saturated conditions which demonstrates the influence of water pressure/or rainfall on the stability of this slope. Likewise, the results obtained via Slide 6.0 software, the Phase 2.0 software analysis of this slope also revealed that the critical slip surface is likely to affect only the top slope profile

during the dynamic dry conditions (Figure 5.21 B), and both the top and middle slope profile during the two saturated conditions (Figure 5.21 C and D). Hence, based on the results obtained from the two software, it can be deduced that the pumice layer (weak layer) found on these two slope profiles can act as a slipping surface for the slope failure.

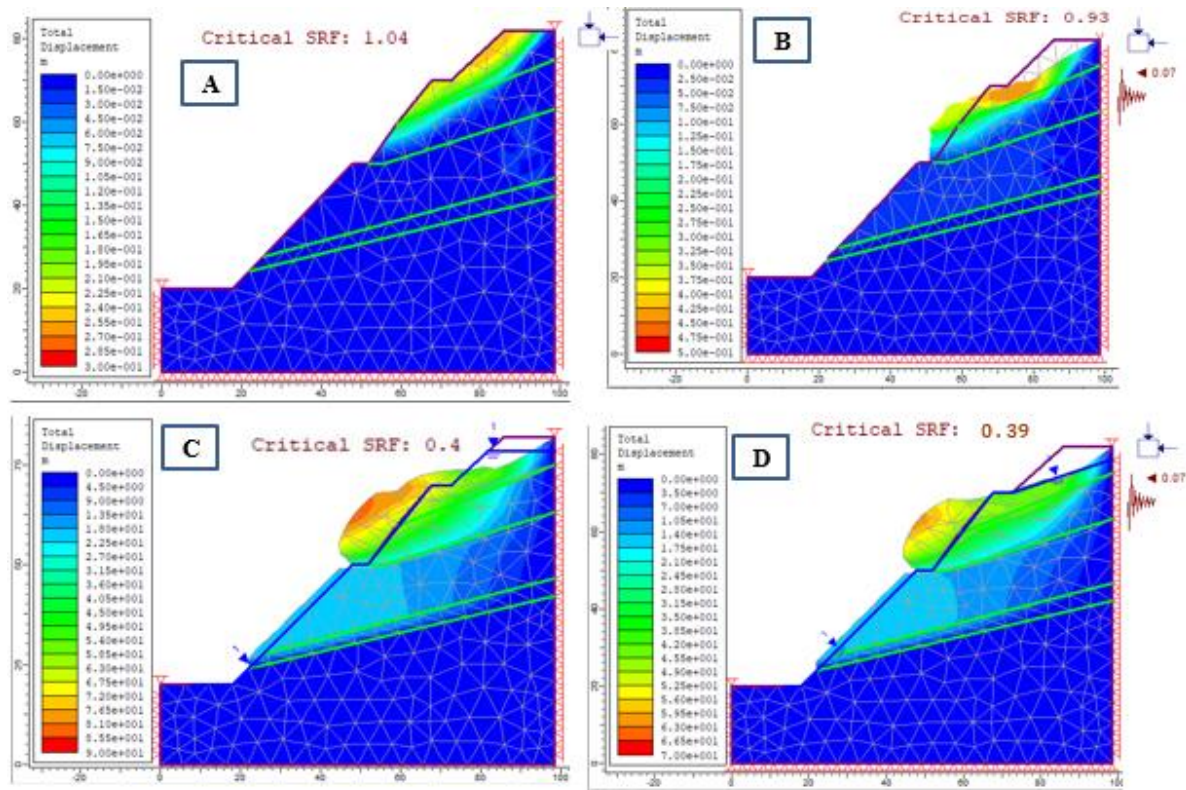


Figure 5. 21.SRF and slope mode of SSS₂ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

The FE-based Phase 2.0 software analysis of slope section SS3 also revealed it is characterized by marginally stable slope conditions during the static dry and unstable conditions during all remaining conditions (Figure 5.22). As can be observed from Figure 5.22 below, the potential failure surface of this slope tends to affect only the top layer (i.e. low plastic silt soil type)

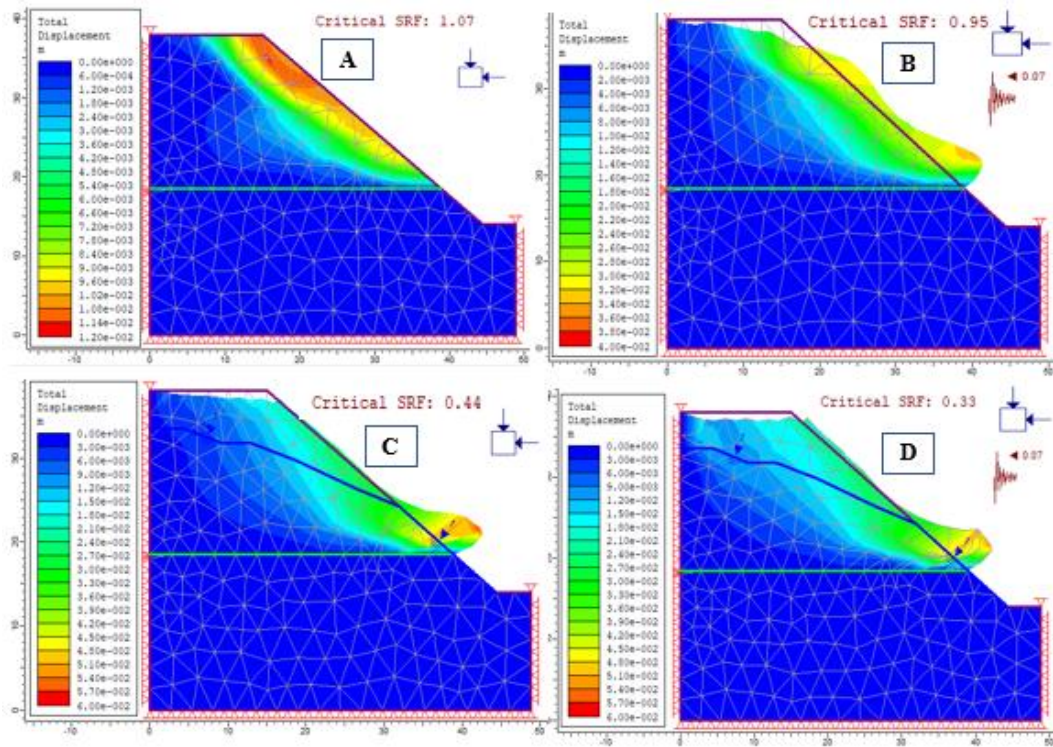


Figure 5. 22 SRF and slope mode of SSS₃ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

Similarly, Phase 2.0 software analysis of the SSS₄ slope revealed unstable slope conditions during all projected conditions (Figure 5.23). This result is consistent with the actual field manifestation of this slope which is constituted by highly weathered slightly welded tuff with pumice intercalation. As can be noted from Figure 5.23, the potential slip surface has crossed through both top and middle slope profile (Figure 5.23) demonstrating the displacement of a large mass of geological materials. Considering the geology and material property of this slope, the weak pumice layer located on both slope profiles has likely contributed to such a large slipping surface.

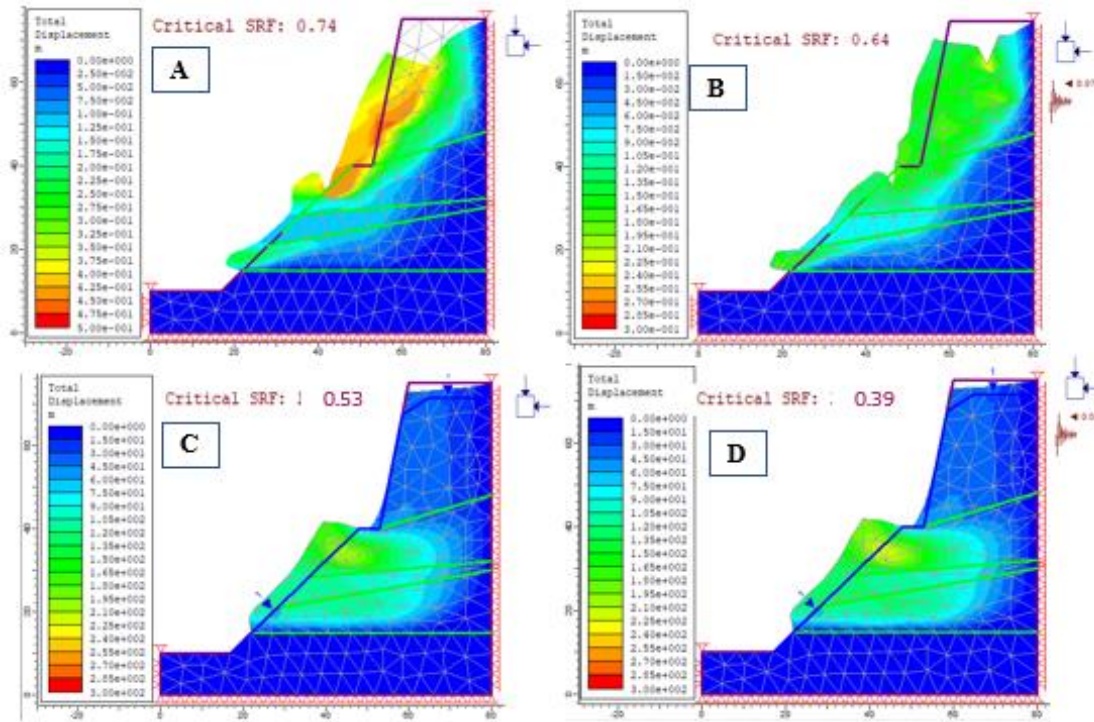


Figure 5. 23 SRF and slope mode of SSS₄ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

The stability analysis of slope section SSS₅ using this Phase 2.0 software also illustrated various stability conditions for this slope. According to the results, this slope is found to be stable during dry conditions and unstable during saturated conditions which are consistent with results obtained from Slide 6.0 software (Figure 5.24). As can be distinguished from the results, there is a significant reduction in SRF during saturated (i.e. 1.13 during static saturated and 0.45 during dynamic saturated) and hence, the stability of this slope is dominantly controlled by water pressure.

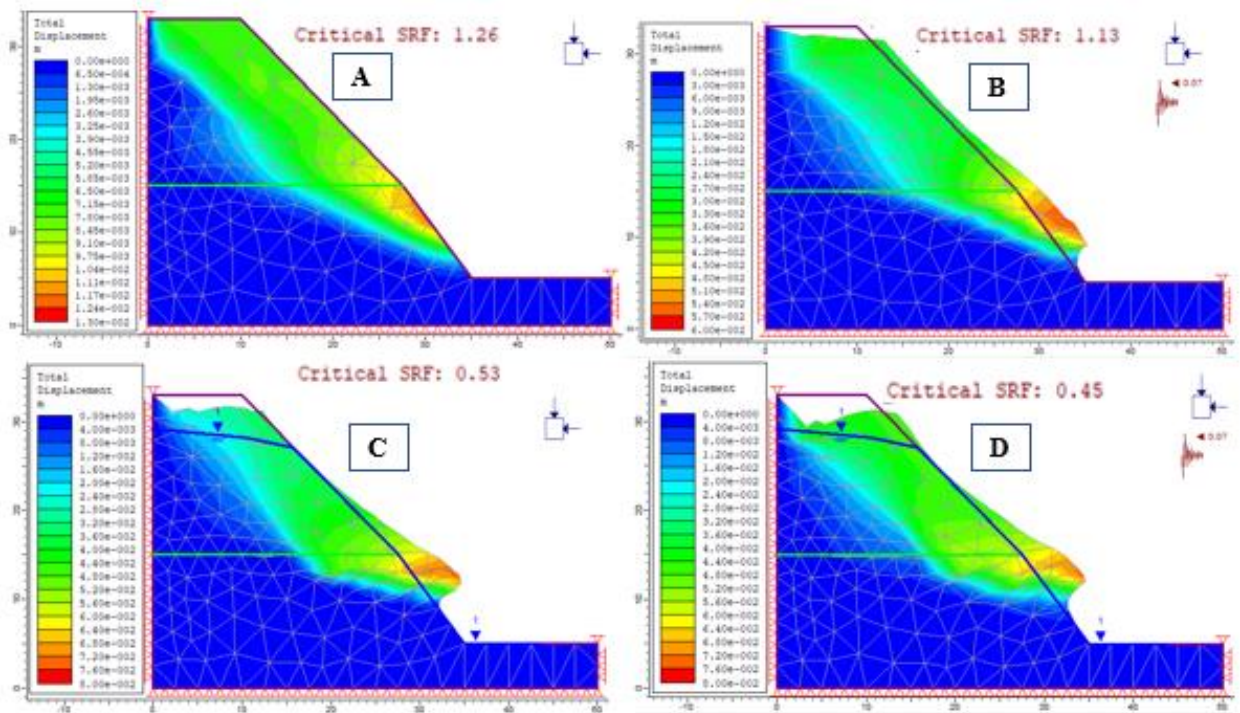


Figure 5. 24 SRF and slope mode of SSS₅ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

The FEM analysis of the last slope section, SSS₆, using Phase 2.0 software also revealed various slope stability conditions. According to the results, this slope is stable during static dry (SRF = 1.18), marginally stable during dynamic dry (SRF = 1.05), and unstable condition during the saturated conditions (Figure 5.25). Likewise analysis results of this slope using Slide 6.0 software, it can be noted from the slope model below (Figure 5.25) that the potential failure tends to follow the weak pumice layer. This demonstrates that the weak pumice layer significantly contribute is destabilizing this slope during saturated conditions by acting as a slipping surface.

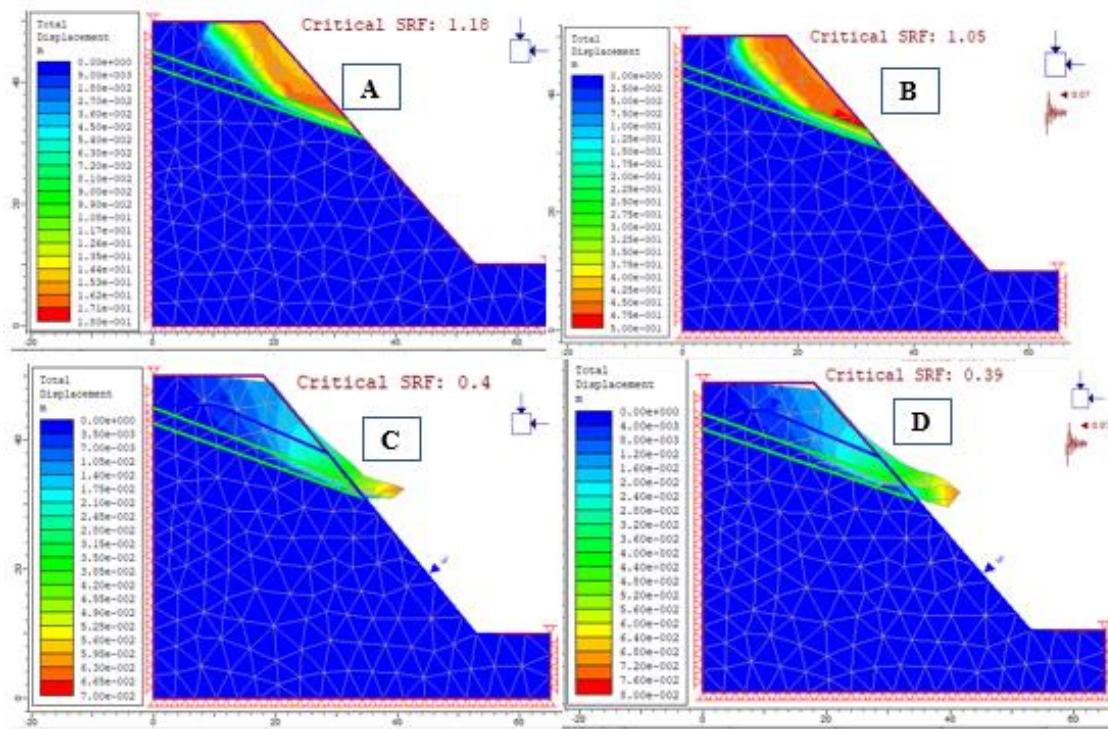


Figure 5. 25 SRF and slope mode of SSS₆ as determined by Phase 2.0 software under A) static dry, B) dynamic dry, C) static saturated and D) dynamic saturated conditions.

Table 5. 26 Summary of SRF obtained via Phase 2.0 software under different anticipated conditions.

Slope Section	Anticipated conditions			
	Static dry	Static saturated	Dynamic dry	Dynamic saturated
SSS ₁	0.600	0.198	0.530	0.143
SSS ₂	1.040	0.400	0.930	0.390
SSS ₃	1.070	0.440	0.950	0.330
SSS ₄	0.740	0.530	0.640	0.390
SSS ₅	1.260	0.530	1.130	0.450
SSS ₆	1.180	0.40	1.050	0.390

In general, the results obtained from Phase 2.0 software showed that all slope sections are unstable under saturated conditions. According to Hossain (2011) and Ahmadi & Eslami (2011), this scenario illustrates the dominant influence of surface and/or groundwater on the slope stability of the study area. On the other hand, two slope sections namely SSS₁ and SSS₄ were found to be unstable under all anticipated conditions including the current static scenario.

5.5.2.2 Stability Analysis Result by Using Plaxis 2D

The stability of non-structural controlled slope sections was also evaluated using another FEM-based PLAXIS software. This analysis was conducted from the same input parameters

used for Phase 2.0 software which are listed in Table 5.17 and Table 5.25. This analysis was, however, conducted only for dry and saturated conditions. This is due to the stability analysis using both Slide 6.0 and Phase 2.0 demonstrated that dynamic condition has little effect on the slope stability of the study area. Therefore, PLAXIS 2D software was used for further demonstration of the contribution of water in the destabilization of the slope of the study area.

Accordingly, the analysis of the SRF factor obtained from slope section SSS₁ showed unstable condition during both static dry (SRF = 0.471) and static saturated conditions (SRF = 0.100). Similarly, the analysis also showed slope section SSS₂ is marginally stable during the dry condition and unstable during saturation. Moreover, it can be distinguished from the results of both slope sections that SRF is significantly lower during saturation (Figure 5.26).

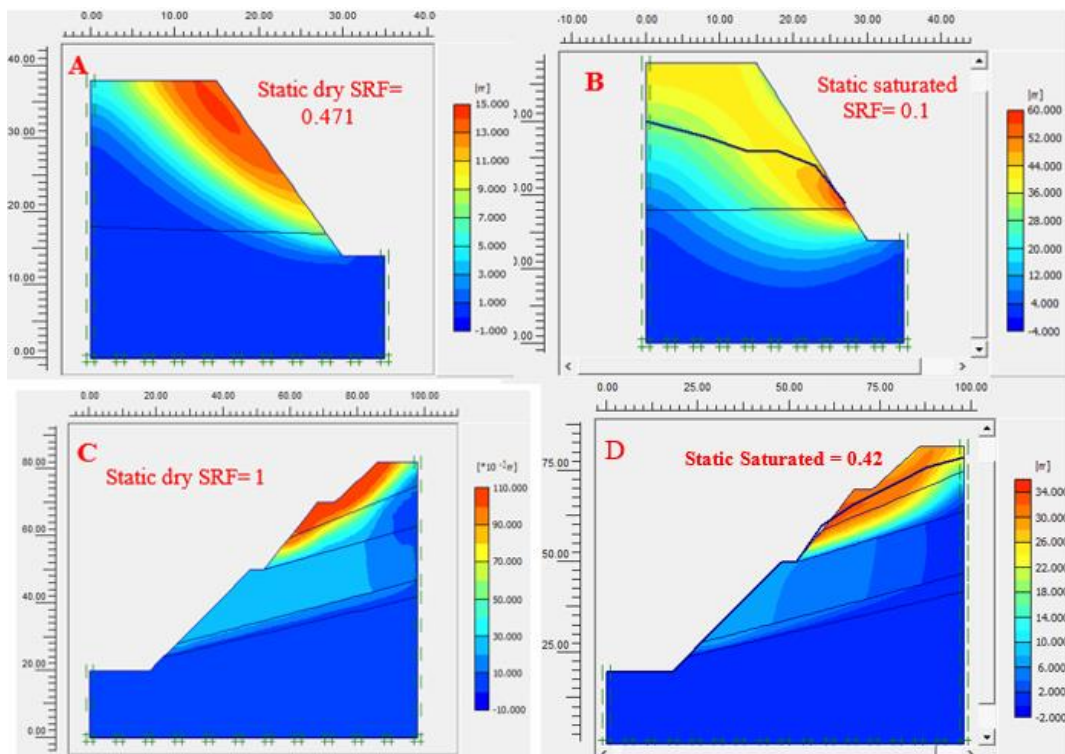


Figure 5. 26 Slope model and SRF as determined through PLAXIS software A and B) for SSS₁, and C and D) for SSS₂.

Similarly, PLAXIS slope stability analysis slope sections SSS₃ and SSS₄ also revealed that both slope sections are marginally stable during the dry condition and unstable during saturation (Figure 5.27). Likewise, analysis results obtained from Slide and Phase software also demonstrate that potential slip surface tends to follow the upper layer for SSS₃, and the top and middle slope profile for SSS₄.

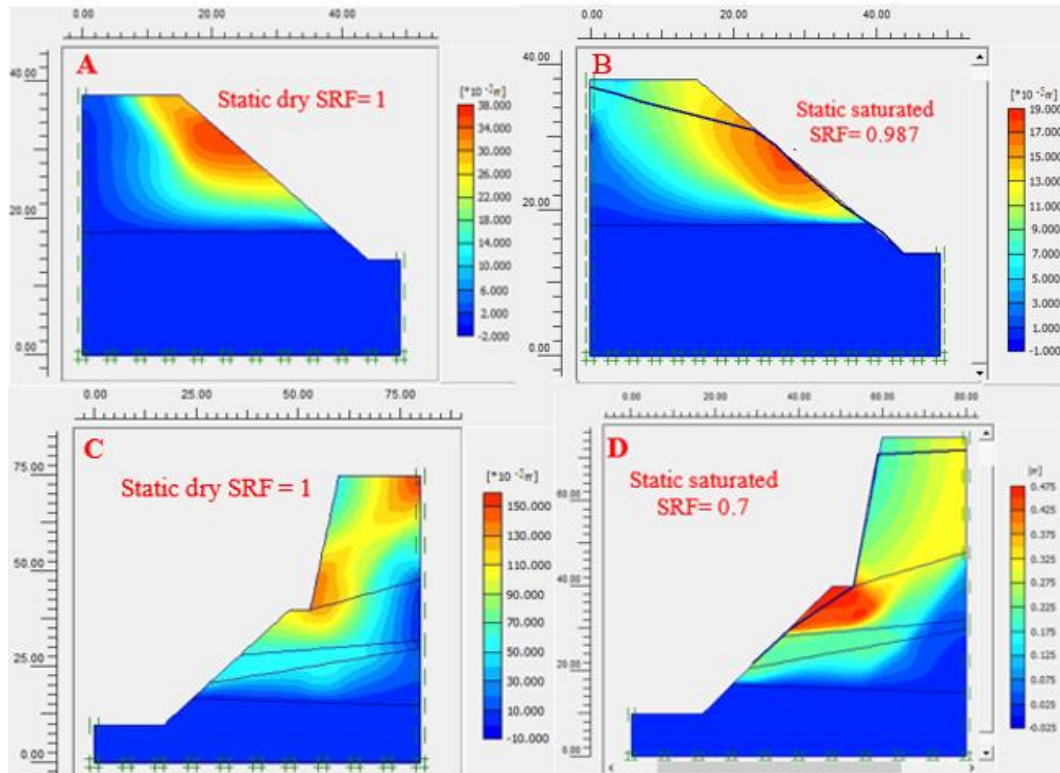


Figure 5. 27 Slope model and SRF as determined through PLAXIS software A and B) for SSS₃, and C and D) for SSS₄.

On the other hand, PLAXIS slope stability of the two remaining sections namely, SSS₅ and SSS₆ revealed marginally stable condition during the static dry condition and unstable slope condition during the static saturated condition (Figure 5.28).

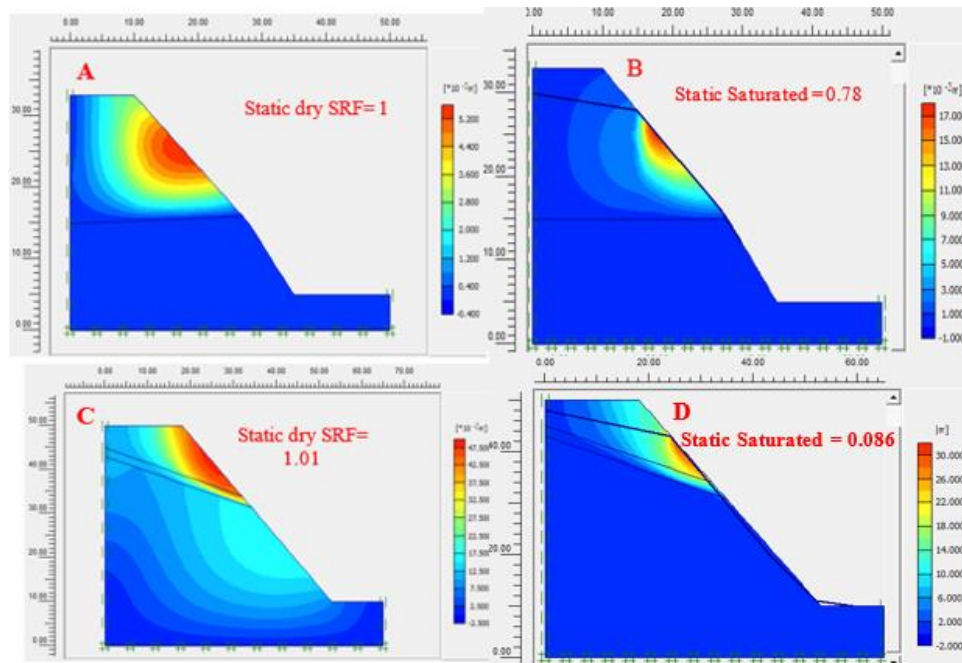


Figure 5. 28 Slope model and SRF as determined through PLAXIS software A and B) for SSS₅, and C and D) for SSS₆.

5.6 Comparison of FOS/SRF for Non-Structural Controlled Slope Sections

Upon determination of FOS and SRF through LEM and FEM-based software, the results were then compared to each other. The comparison showed that the FOS and SRF obtained LEM-based Slide 6.0 and FEM-based Phase 2.0 software are in close agreement (Table 5.27). However, a detailed examination of the results also demonstrated that SRF obtained from Phase 2.0 is slightly lower than the corresponding FOS obtained from Slide 6.0 software. Moreover, the potential slip surface illustrated by both Slide 6.0 and Phase 2.0 software is also in very close agreement. In addition, the analysis of results obtained from both methods also showed that seismicity has little effect on the stability of the slope of the study area.

The comparison of SRF obtained from PLAXIS software, however, shows some slight difference from those obtained using Slide 6.0 and Phase 2.0 software. The SRF obtained from PLAXIS software is slightly lower than corresponding values determined via Slide software for all slopes dry conditions whilst it yielded higher SRF under static saturated conditions for SSS₃, SSS₄, SSS₅, and SSS₆. Similarly, it also yielded SRF which are lower than those determined by Phase software under static dry conditions except for SSS₄ (Table 5.27).

Table 5. 27 Correlation of FOS/SRF obtained between Slide 6.0, Phase 2.0, and PLAXIS software.

Slope Section	LEM (Slide 6.0 Software)				FEM (Phase 2.0 Software)				FEM (PLAXIS)	
	SD	SS	DD	DS	SD	SS	DD	DS	SD	SS
SSS ₁	0.649	0.221	0.585	0.169	0.600	0.198	0.530	0.143	0.471	0.100
SSS ₂	1.049	0.461	0.969	0.410	1.040	0.400	0.930	0.390	1.000	0.420
SSS ₃	1.099	0.769	0.980	0.684	1.070	0.440	0.950	0.330	1.000	0.987
SSS ₄	1.069	0.591	0.877	0.414	0.740	0.530	0.640	0.390	1.000	0.700
SSS ₅	1.118	0.728	0.992	0.505	1.260	0.53.0	0.130	0.450	1.000	0.780
SSS ₆	1.236	0.772	1.064	0.679	1.180	0.400	1.050	0.390	1.010	0.118

In general, the results obtained from Phase 2.0 software showed that all slope sections are unstable under saturated conditions (Table 5.27). According to Hossain (2011) and Ahmadi & Eslami (2011), this scenario illustrates the dominant influence of surface and/or groundwater on the slope stability of the study area. On the other hand, two slope sections namely SSS1 and SSS4 were found to be unstable under all anticipated conditions including the current static scenario. Based on studies by Johnson & Degraff (1991), Price (2009), and Hamza and Raghuvanshi (2017), slope geometry and shear strength parameters of materials play a significant in destabilizing slopes of such conditions. Moreover, the analysis of Phase results also revealed that seismicity has little effect on the slope stability condition of the study area as can be noted by the difference in SRF between static dry and dynamic dry conditions.

5.7 Possible Remedial Measures

5.7.1 Possible Remedial Measures for Structural Controlled Slope Sections

As stated in section 5.4.2, the LEM analysis of structural controlled slope sections showed that slope section RSS₁ is unstable for the planar mode of failure under all projected conditions. Similarly, the analysis also revealed that RSS₃ is unstable for wedge failure due to the intersection of JS1 and JS3 under all scenarios whilst RSS₂ is only stable for wedge mode of failure under dynamic saturated condition. Hence, appropriate remedial measure/s is/are very crucial for these slope sections so that they can withstand failure even if they are subjected to the worst-case scenario (i.e. dynamic saturated condition).

Accordingly, the stabilization analysis of the planar mode of failure was carried out by adopting Rocplane 2.0 software (Rocplane 2.0 Rocscience, 2004) through the application of

Rock Bolts which were implanted across the failure plane to relatively strength rock. Similarly, the stabilization analysis for the wedge mode of failure was done with the help of Swedge 4.0 software (Swedge 4.0 Rocscience, 2004) by using Rock Bolts and Shotcrete to the face of the slope. All the stabilization analyses were carried out considering the worst (i.e. dynamic saturated) condition of the slope. The designed bolt length were considered to pass the failure plane to increase the factor of safety.

Subsequently, four rock bolts were applied to RSS₁ at an angle of 10° to the slope each with 490 ton bolt capacity to stabilize planar failure due to JS1 (Figure 5.29). As a result of this stabilization, the FOS of this slope due to JS1 has increased from 0.090 to 1.25. Similarly, four rock bolts each of them with a 1000 ton bolt capacity were installed to RSS₁ at an angle of 15° to the slope to stabilize planar failure due to JS3 (Figure 5.30). This stabilization measure has increased the FOS of this slope due to JS3 to 1.22. According to Hoek & Bray (1981) and Duncan (1995), FOS close to 1.2 is recommended for road-slope design. Agreeing to these studies, the FOS for both JS1 and JS3 after installation of rock bolts becomes greater than 1.2 illustrating stable slope condition.

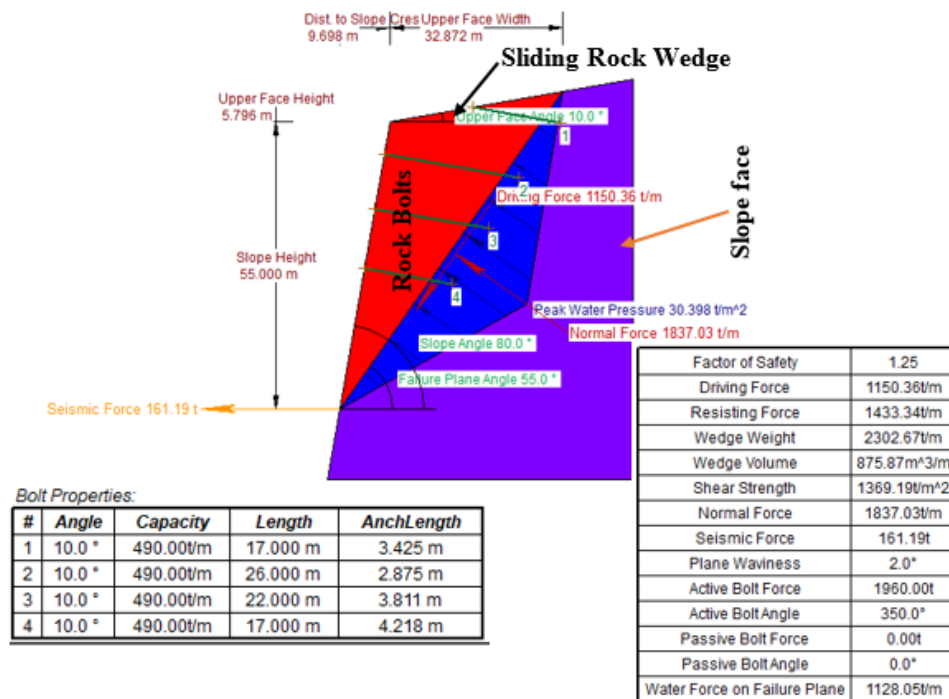


Figure 5. 29 A designed and installed rock bolt for JS1 of RSS₁

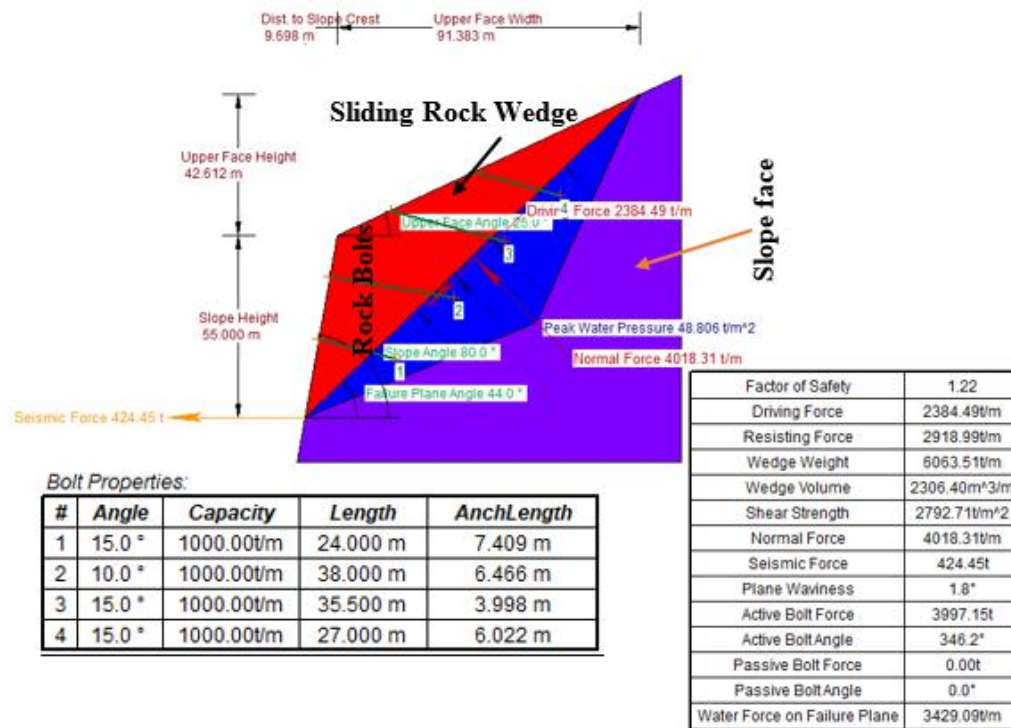


Figure 5. 30 A designed and installed rock bolt for JS3 of RSS₁

Table 5. 28 Stabilization of planar mode of failure at RSS1 through application of Rock Bolt.

Slope section	Joint set	Bolt Parameters					FOS
		Bolt angle (°)	Average Bolt Length (m)	Single Bolt Capacity (ton/m)	Number of Bolt applied	Total Bolt Capacity (ton/m)	
RSS1	JS1	10	20.5	490	4	1960	1.25
	JS3	15	31	1000	4	4000	1.22

Similarly, about five rock bolts with plunge direction of 20° and 750 ton single bolt capacity and shotcrete with shear strength capacity of 1600 t/m² were designed and applied to RSS₂ to stabilize the likely wedge due to the intersection of JS2 and JS3 (Figure 5.31 A). This remedial measure has increased the FOS of this slope from 0.723 to 1.251. Moreover, four rock bolt with a single bolt capacity of 7800 ton with a plunge direction of 10° and shotcrete with a shear strength of 2950 t/m² was applied to RSS₃ to stabilize the wedge due to the intersection of JS1 and JS3 (Figure 5.31 B). According to the results, this countermeasure technique has increased the FOS of this slope to 1.201. According to Hoek & Bray (1981) and Duncan (1995), the FOS became greater than 1 due to the applied stabilization measure, and hence the slope will be stable.

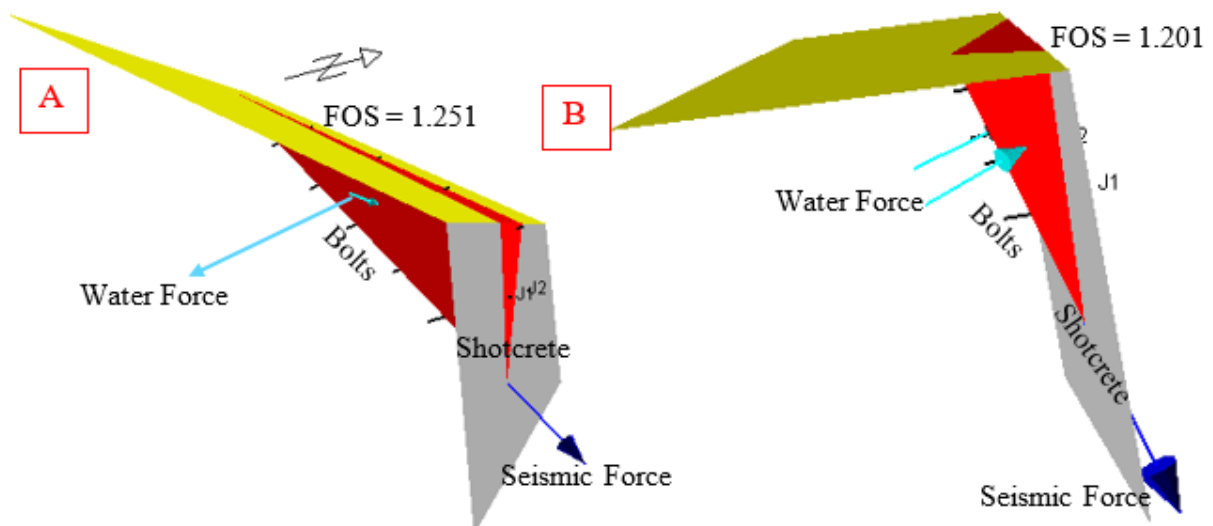


Figure 5. 31 A designed and installed rock bolt and shotcrete for slope section A) for RSS₂ and B) for RSS₃ (wedge-formed due to intersection of JS1 and JS3).

Table 5. 29 Stabilization of wedge mode of failure at RSS2 and RSS3 through application of Rock Bolt and Shotcrete.

Slope section	Bolt Parameters								FOS
	Joint	Bolt plunge direction (°)	Bolt trend direction (°)	Average Bolt Length (m)	Single Bolt (ton/m) capacity	Number of Bolt applied	Total Bolt Capacity (ton/m)	Shotcrete shear strength (t/m ²)	
RSS2	JS2 & JS3	20	285	22	750	5	3750	1600	1.251
RSS3	JS1 & JS3	10	248	16	7800	4	31200	2950	1.201

5.7.2 Possible Remedial Measures for Non-Structural Controlled Slope Sections

The stability analysis/or modeling of non-structural controlled slope sections using SLIDE, PHASE, and PLAXIS software showed that all slope sections are unstable during saturated and dynamic conditions. Taking these results and field observation into consideration, flattening slope angle, multi-sloping, and benching were identified and tested for remedial measures. Accordingly, three slope sections namely SSS₂, SSS₄, and SSS₅ were selected for such remedial measure analysis based on their field manifestation and will be discussed in the next sections.

5.7.2.1 Flattening Slope Angle

Steeply sloped sections are highly susceptible to slope failure (Hamza and Raghuvanshi, 2017; Raghuvanshi et al., 2014). With increasing slope angle, the rock and soil mass will

exert additional load on the weight component of the slope, and thus, shear stress will increase, and slope failure will be aggravated (Sharma et al., 1995).

Based on this notion, flattening slope angle analysis was carried out for three slope sections such as SSS₂, SSS₄, and SSS₅ from their worst condition (i.e. dynamic saturated condition). The original slope angle was modeled as (1H: 1V) and the analysis was then carried out by subsequently reducing the slope angle from (1H: 1V) to (1.5H: V), (2H: 1V) and (2.5H: 1V) by keeping the height (V) constant. This analysis was carried out using Slide 6.0 software (Slide 6.0 Rocscience, 2004) using the Spencer technique. Finally, the interpretation of results (i.e. FOS) was made based on work by Duncan (1996) which suggested the FOS of close to 1.2 for road-slope stability. Table 5.30 below presents the results of this slope flattening analysis.

Table 5. 30 The results of slope flattening analysis for slope section SSS₂, SSS₄ and SSS₅ as determined from the dynamic saturated conditions.

Slope Section and Failure Slope Face		1H:1V		1.5H:1V		2H:1V		2.5H:1V	
		Eqv. Angle	FOS	Eqv. Angle	FOS	Eqv. Angle	FOS	Eqv. Angle	FOS
SSS ₂	Top Slope face	42	0.410	29.75	0.990	23.19	1.267	21.80	1.854
	Middle Slope face	52		41.63		33.69		28.07	
SSS ₄	Top layer	79	0.414	73.3	0.774	68.20	0.774	63.43	1.157
SSS ₅	Top Slope face	46	0.614	34.44	0.842	27.22	1.201	22.36	1.769

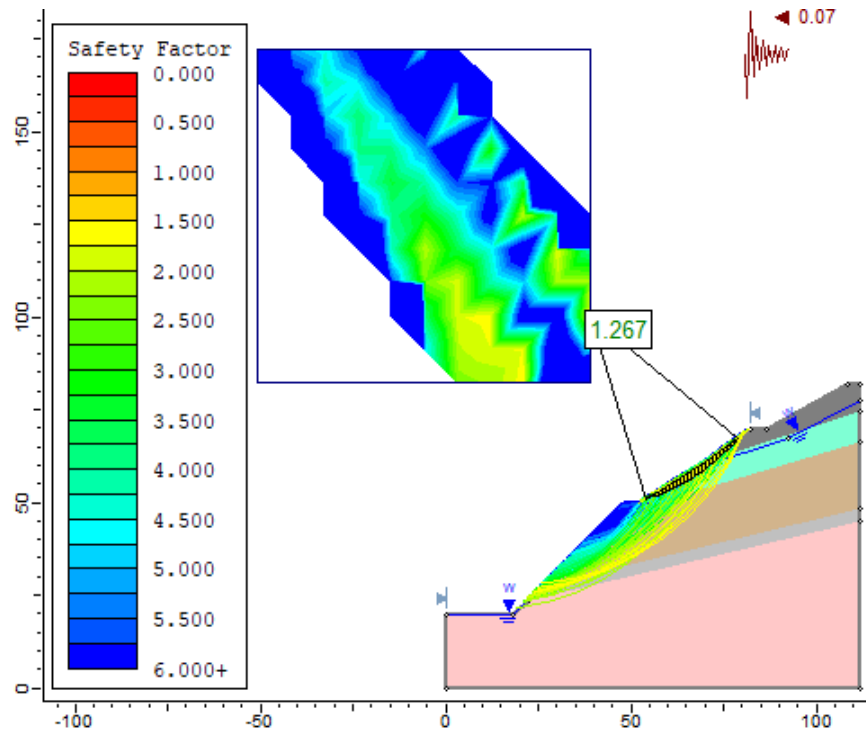


Figure 5. 32A sample of a slope model after slope flattening to 2H: 1V for slope section SSS2.

The modeling of slope sections SSS₂ using Slide 6.0 and Phase 2.0 software under saturated conditions showed that the potential failure involves both top and middle slope face (Figure 5.15 A &B and Figure 5.21). Hence, the slope flattening analysis of this slope section was carried out for both top and middle slope faces. The analysis showed that this slope will be stabilized by reducing the upper slope angle from 42° to 23.09° and the middle slope angle from 52° to 33.69° which increases FOS from 0.410 to 1.267 (Table 5.30 and Figure 5.32). This stabilization is obtained by reducing the original slope angle (1H: 1V) to (2H: 1V) which increased FOS by 77.26%.

Similarly, slope flattening analysis showed that slope section SSS₄ will be stabilized at an angle as low as 63.43° rather than 79° (Table 5.30). Moreover, the modeling also showed that slope section SSS₅ will become stable with a reduction in slope angle from 1H: 1V (46°) to 2H: 1V (27.22°). For this slope section, the reduction in slope angle from 1H: 1V (46°) to 1H: 1.5V (34.44°) and 2H: 1V (27.22°) has increased FOS by 34.28 % and 48.90 %, respectively.

5.7.2.2 Multi-Sloping

Multi-sloping is another mechanism of reducing slope angle in which a single slope profile is divided into two or more combinations of slope angle profiles. Accordingly, in this study,

the effect of multi-sloping on the stability of the slope was evaluated by dividing the failure surface profile into two successively lower slope angles slope namely (θ and β). Multi-sloping analysis was then carried out using Slide 6.0 software (Slide 6.0 Rocscience, 2004) as per the Spencer technique for SSS2, SSS4, and SSS5 from their worst condition. The results of the analysis are presented in Table 5.31.

Table 5. 31 The results of multi-sloping analysis for slope section SSS2. SSS4 and SSS5 as determined from the dynamic saturated conditions.

Multi-Sloping Angles		SSS2		SSS4	SSS5
		Top slope profile	Middle slope profile	Top slope profile	Top slope profile
Original angle		42	52	79	46
MSA1	θ1	32	45	70	35
	β1	38	48	75	40
	FOS	0.591		0.758	0.741
MSA2	θ2	28	40	65	30
	β2	32	44	70	35
	FOS	0.921		0.962	0.997
MSA3	θ3	25	38	60	25
	β3	28	35	65	30
	FOS	1.234		1.189	1.573
MSA = Multi-Slope Angle					

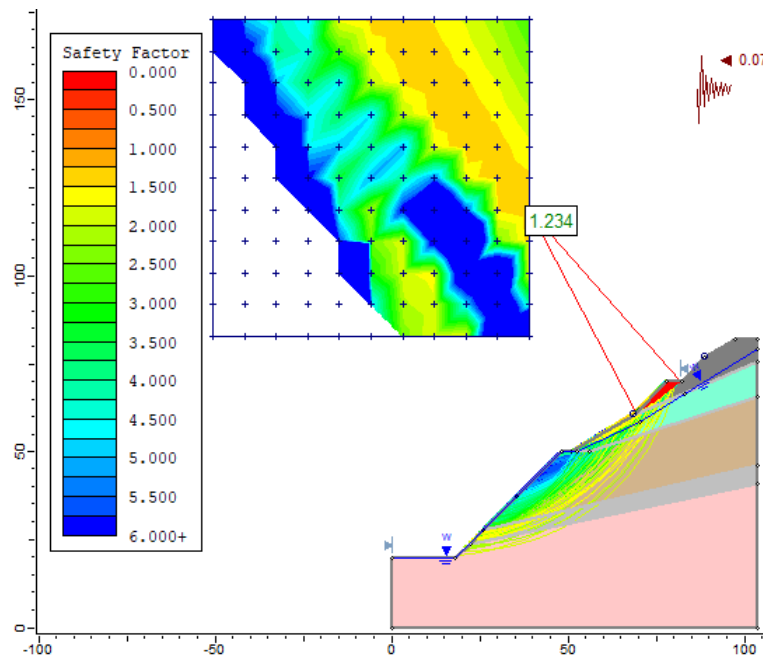


Figure 5. 33 A sample of a slope model after multi-sloping for SSS₂

As mentioned earlier, the potential failure surface of slope section SSS₂ includes two slope angles (which were previously designated as top and middle slope faces). Hence, during the multi-sloping analysis of this slope section, both the top and middle slope faces were

separately divided into two slope angle profiles. The analysis showed that SSS₂ will be stabilized with a FOS of 1.234 when its top slope profile is divided multi-sloped into ($\theta = 25^0$ and $\beta = 28^0$) and the middle into ($\theta = 25^0$ and $\beta = 28^0$) (Table 5.31 and Figure 5.33). This multi-sloping of this slope section has increased FOS by 75.24 % from its original dynamic saturated condition. Similarly, the multi-sloping analysis revealed that SSS₄ and SSS₅ will become stable when their failure surface profile is divided into ($\theta = 60^0$ and $\beta = 65^0$) and ($\theta = 25^0$ and $\beta = 30^0$), respectively (Table 5.31).

5.7.2.3 Combination of Slope Flattening and Benching

Benching coupled with slope flattening is one of the most effective slope stabilization mechanisms (Jampani and Bhupathi, 2017). Accordingly, in this study, this analysis was carried out by successively reducing the slope angle from (1H: 1V) to (1.5H: V), (2H: 1V), and (2.5H: 1V), and providing a 3 m width bench at different heights. Based on field manifestation, this analysis was carried out for SSS₂, SSS₃, and SSS₅ for their worst condition using Slide 6.0 software (Slide 6.0 Rocscience, 2004) via the Spencer technique. Table 5.32 below presents the results of this combined slope flattening and benching.

Table 5. 32 The results of combined slope flattening and benching analysis for slope sections SSS₂, SSS₄ and SSS₅ as determined from the dynamic saturated conditions.

Slope Section and Failure Slope Face		1H:1V			1.5H:1V			2H:1V			2.5H:1V		
		Eqv. Angle	Bench Height	FOS	Eqv. Angle	Bench Height	FOS	Eqv. Angle	Bench Height	FOS	Eqv. Angle	Bench Height	FOS
SSS2	Top Slope face	42	55	0.606	29.75	60	1.104	23.19	70	1.621	21.80	75	1.964
	Middle Slope face	52	35		41.63	45		33.69	55		28.07	60	
SSS4	Top layer	79	25	0.592	73.3	30	0.797	68.20	45	0.855	63.43	55	1.846
SSS5	Top Slope face	46	15	0.747	34.44	19	1.049	27.22	23	1.599	22.36	27	1.812

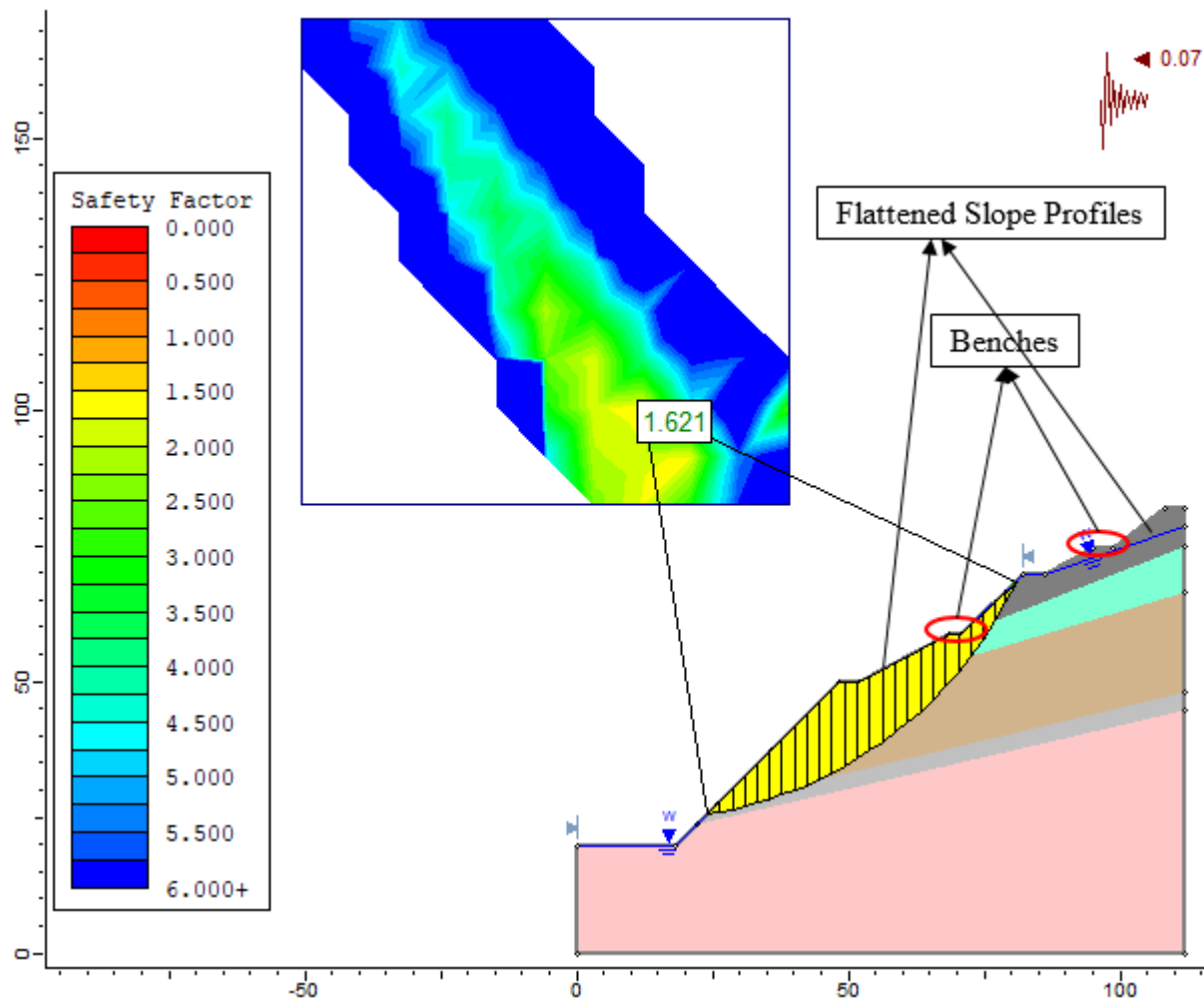


Figure 5. 34 A sample of a slope model after combined slope flattening to 2.5H: 1V and Bench at heights of 50 m and 70 m for slope section RSS2.

The results of the analysis showed that combined slope flattening and benching have a significant influence on the stabilization as compared to just slope flattening alone. For example, the slope flattening analysis has shown that the reduction of slope angle from 1H: 1V to 2H: 1V has resulted in an increase in FOS from 0.410 to 1.267 for SSS₂ (Table 5.32 and Figure 5.34). On the other hand, when a 3 m width bench is provided at a height of 50 m (on middle slope profile) and 70 m (on top slope profile) with the same slope flattening angle, the FOS has increased from 0.410 to 1.621 (Table 5.32 and Figure 5.34). This showed this combined slope flattening and benching has increased FOS by 99.02 % illustrating the effectiveness of this stabilization mechanism.

Similarly, stabilization analysis of SSS₄ with original slope angle (1H: 1V) and application of 3 m width bench at height of 25 m has increased FOS to 0.592. However, combined slope flattening of this slope from 1H: 1V to 2.5H: 1V and 3 m width bench at the height of 55 m

has significantly increased FOS to 1.846 illustrating the effectiveness of combined slope flattening and benching (Table 5.32). Furthermore, combined slope flattening and benching analysis of SSS_5 at 1H: 1V and 15 m bench height, 1.5H: 1V and 19 m bench height, 2H: 1V and 23 m bench height, and 2.5H: 1V and 27 m bench height has resulted in FOS of 0.747, 1.049, 1.599 and 1.812, respectively (Table 5.32). This showed that combined slope flattening and benching have a significant contribution to stabilizing this slope. Moreover, this analysis also showed that FOS increases with an increase in bench height (Table 5.32).

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This study evaluated the stability of the slope along the road section based on a detailed field survey followed by a laboratory test, rock mass rating, kinematic analysis, and LEM and FEM methods. Finally, the following concluding remarks were made.

- ✚ The study area is mainly constituted by pyroclastic deposit, ignimbrite, basalt, scoria, trachyte, and residual soil deposits.
- ✚ Based on field investigation, three structural controlled and six non-structural controlled slope sections were identified.
- ✚ The RQD values of the structural controlled slope sections classified as good rock according to Deer and Deere(1988).
- ✚ The UCS of rocks of the study area (for structural controlled slope sections) ranges from 42.170MPa to 136.490MPa for the ignimbrite and for non-structural controlled slope section ranges from 16.7 MPa to 28.9 MPa.
- ✚ The structural controlled slope sections are constituted by medium strong rock to very strong rock ignimbrite as per ISMR (1978) rating and fair to good rock mass quality as per RMR rating.
- ✚ The rocks in the nonstructural controlled slope section are classified as weak rock to medium-strong tuff rock rock as per ISMR (1978).
- ✚ The kinematic analysis of structural controlled slope sections revealed kinematic admissibility for planar failure due to JS1 and JS3 at RSS₁. Similarly, this analysis also showed the occurrence of wedge mode of failure due to the intersection of JS2 and JS3 at RSS₂ and JS1 and JS3, and JS2 and JS3 at RSS₃.
- ✚ Further stability analysis of planar mode using Rocplane software showed that RSS₁ is unstable (i.e. FOS < 1) under all projected conditions. Similarly, stability analysis of wedge mode of failure through swedge software revealed that slope section RSS₂ is stable only during the static dry condition. In addition, it was deduced that slope section RSS₃ is stable due to the intersection of JS2 and JS3, and unstable due to the intersection of JS1 and JS3 under all anticipated conditions.
- ✚ The non-structural controlled slope section is mainly constituted by soil, tuff, pumice, and basaltic rocks. The tuff is mostly intercalated with pumice and has weak to

medium strength as per ISRM (1978) rating. The basaltic rock at that found at the bottom of SSS₅ is also highly weathered and has medium strength.

- ✚ Grain size analysis and Atterberg limit tests imply that the soils obtained from the critical slope section are dominantly composed of coarse-grained soil with plasticity index ranging from 9.102 to 22.560 and liquid limit varies from 39.030 to 55.430. Generally, according to the USCS system, the soils of the study area were classified as silty sand, poorly-graded silty sand, low plastic silt, and high plastic silt.
- ✚ Both LEM-based and FEM-based software showed that all six non-structural controlled slope sections are unstable (i.e. $FOS < 1$) during saturated conditions. Hence, it is deduced that saturation is one of the major factor contributing to slope failure in the study area.
- ✚ A comparison of FOS obtained from LEM-based Slide 6.0 software and SRF obtained from FEM-based Phase 2.0 software is in very close agreement. However, a detailed examination showed that FEM yielded slightly lower SRF attributing its consideration of stress and strain deformation.

Recommendations

For the most part, this study was concerned with the slope stability analysis along the selected rock section from woliso Town to wonchi Lake. Different methods and approaches were used in order to analyze the slope section. Therefore, satisfactory and proper remedial measures are essential in order to stabilize the slope sections are recommended based on the analysis result.

- ✚ To control the effects of water saturation, surface drainage should be planned for non-structural controlled sections by redesigning the slope face to control water flow and runoff, providing a flow line to remove rainwater, constructing toe drains, and planting vegetation on the slope face. And also using subsurface drainage is also recommended because the occurrence of springs along the slope surface indicates that the groundwater is shallow. Furthermore, surface drainage system like like covering the rock with an impervious material to control the water that enters into the joints is also recommended for structural controlled slope sections.
- ✚ Rock bolt with an orientation of 10^0 to the slope face with a total bolt capacity of 1960 tons and another series of bolts with a total bolt capacity of 4000 tons at an angle of 15^0 to the slope face is designed to stabilize the planar mode of failure due to JS1 and JS3 at RSS₁, respectively.

- ✚ A total bolt capacity of 3750 tons with a plunge direction of 20^0 and shotcrete with a shear strength capacity of 1600 t/m^2 is designed to stabilize the likely wedge due to the intersection of JS2 and JS3 at RSS₂.
- ✚ A rock bolt with a total bolt capacity of 31200 tons with a plunge direction of 10^0 and shotcrete with a shear strength of 3950 t/m^2 are designed to stabilize the wedge due to the intersection of JS1 and JS3 at RSS₃.
- ✚ However, considering the budgets of the bolts and the current stage of this road construction, removing unstable rock wedges is recommended for structural controlled slope sections.
- ✚ For structural controlled slope sections, slope flattening (if desired) should consider a reduction in slope angle of the top slope profile of SSS₂ to less than 23.19^0 and the middle slope profile to less than 33.69^0 to stabilize this slope through slope flattening. Moreover, the top slope profiles of slope sections SSS₄ and SSS₅ should be reduced to an angle less than 63.43^0 and 27.22^0 , respectively to be remediated by slope flattening.
- ✚ Multi-sloping remedial measure (if desired) should divide the upper slope profile of SS4 into two profiles with an angle of angle less than ($\theta = 25$ and $\beta = 28$) and the middle slope profile into ($\theta = 38$ and $\beta = 35$). Similarly, multi sloping SSS4 and SS5 should be at an angle of less than ($\theta = 60$ and $\beta = 65$), and ($\theta = 25$ and $\beta = 30$), respectively.
- ✚ This study has recommended combined slope flattening and benching as this mechanism proved to be effective for slope sections of the study area.

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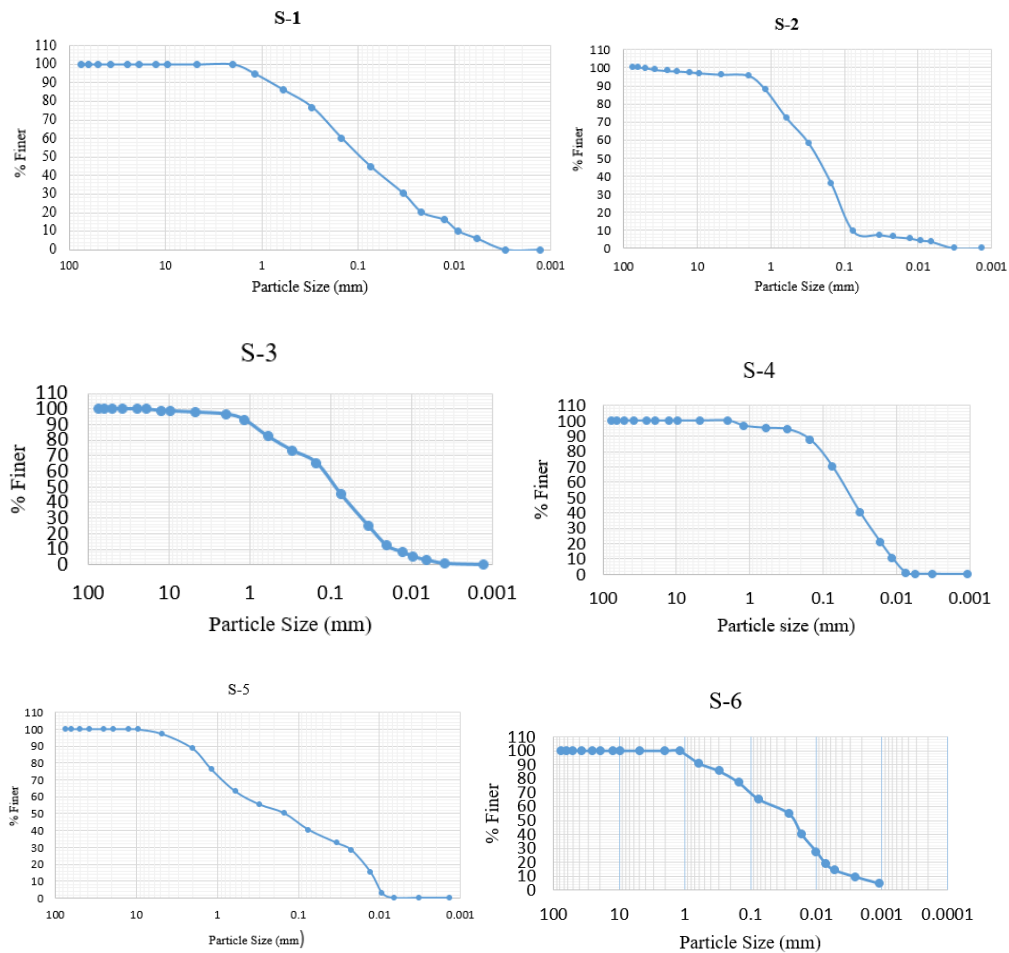
APPENDICES

Appendix 1: Unit weight of rocks at structural controlled and nonstructural controlled slope section

	Structural Controlled							
Slope section	Rock Type	Unit weight	Mass(g)	V1(ml)	V2(ml)	V2-V1 (ml)	ρ=(g/cm³)	Average γ (N/m³)
RSS1	Ignimbrite	γ _{dry}	133.480	300.000	360.000	60.000	2.225	21.240
			145.650	300.000	368.000	68.000	2.1419	
		γ _{sat}	135.010	350.000	398.000	48.000	2.813	25.783
			122.160	350.000	400.000	50.000	2.4432	
RSS2	Ignimbrite	γ _{dry}	162.920	300.000	370.000	70.000	2.310	22.712
			150.200	298.000	367.000	69.000	2.320	
		γ _{sat}	85.460	350.000	380.000	30.000	2.850	28.505
			115.460	380.000	419.000	39.000	2.961	
RSS3	Ignimbrite	γ _{dry}	142.600	320.000	381.140	67.140	2.125	22.589
			121.690	315.690	401.149	86.459	2.460	
		γ _{sat}	123.685	322.984	367.141	44.157	2.801	27.813
			116.230	364.100	404.600	40.500	2.869	
	Non-structural Controlled							
SSS2	Slightly welded tuff	γ _{dry}	79.000	354.000	392.146	38.146	2.071	20.020
			82.000	361.000	401.796	40.796	2.010	
		γ _{sat}	91.000	351.200	394.992	43.792	2.078	21.950
			89.630	311.000	348.408	37.408	2.396	
	Medium Welded tuff	γ _{dry}	86.236	312.260	357.528	45.268	1.905	19.900
			93.654	315.630	359.169	43.539	2.151	
		γ _{sat}	99.600	318.100	357.577	39.477	2.523	22.300
			98.650	311.256	360.016	48.760	2.023	
SSS3	Slightly welded tuff	γ _{dry}	86.360	310.600	346.140	35.540	2.430	21.500
			92.159	321.260	368.521	47.261	1.950	
		γ _{sat}	98.300	290.000	344.824	54.824	1.793	22.600
			94.250	295.000	328.505	33.505	2.813	
SSS4	Slightly welded tuff	γ _{dry}	81.560	294.500	330.288	35.788	2.279	24.900
			86.258	315.400	346.239	30.839	2.797	
		γ _{sat}	89.300	312.600	346.890	34.290	2.604	27.000
			95.400	322.852	355.748	32.896	2.900	
	Medium Welded tuff	γ _{dry}	83.423	312.300	350.065	37.765	2.209	23.100
			88.951	326.500	362.066	35.566	2.501	
		γ _{sat}	92.630	320.000	357.869	37.869	2.446	26.350
			95.251	318.000	351.005	33.005	2.866	
SSS5	Highly Weathered Basalt	γ _{dry}	154.257	290.350	367.325	76.975	2.004	23.000
			149.223	284.960	340.557	55.597	2.684	
		γ _{sat}	161.025	300.500	382.739	82.239	1.958	24.210
			154.360	305.400	357.233	51.833	2.978	
SSS ₆	Slightly welded tuff	γ _{dry}	86.489	300.500	342.282	41.782	2.070	22.760
			83.654	307.600	340.150	32.550	2.570	
		γ _{sat}	94.258	302.900	340.220	37.320	2.526	26.346
			96.558	300.000	333.930	33.930	2.846	

Appendix 2: Sieve and hydrometer analysis

S-1 at Slope section SSS ₁		S-2 at slope section SSS ₁		S-3 at slope SSS ₂	
Sieve opening (mm)	%finer	Sieve opening (mm)	%finer	Sieve Opening (mm)	% Finer
75	100	75	100	75	100
63	100	63	100	63	100
50	100	50	99.89	50	100
37.5	100	37.5	98.91	37.5	100
25	100	25	98.25	25	100
19	100	19	98.02	19	99.8
12.5	100	12.5	97.56	12.5	98.9
9.5	100	9.5	97.01	9.5	98.65
4.75	100	4.75	96.216	4.75	97.9
2	100	2	95.65	2	96.7
1.18	94.86	1.18	88	1.18	93.2
0.6	86.3	0.6	72	0.6	82.4
0.3	76.84	0.3	58	0.3	73.4
0.15	60.34	0.15	36	0.15	65.4
0.075	45.04	0.075	9.25	0.075	45.4
0.03402	30.28126	0.033679	7.45	0.03456	25.3251
0.02201	20.18751	0.021946	6.35	0.02015	12.3625
0.012853	16.15001	0.012853	5.45	0.01287	8.32544
0.009247	10.09375	0.009247	4.29	0.00945	5.23541
0.005864	6.056252	0.006605	3.68	0.00645	3.21534
0.002985	0	0.003215	0	0.00386	1.02354
0.001288	0	0.001371	0	0.00129	0.23514
S-4 at slope section SSS ₃		S-5 at slope section SSS ₅		S-6 at slope section SSS ₆	
Sieve opening (mm)	% Finer	Sieve Opening (mm)	% Finer	Sieve Opening (mm)	% Finer
75	100	75	100	75	100
63	100	63	100	63	100
50	100	50	100	50	100
37.5	100	37.5	100	37.5	100
25	100	25	100	25	100
19	100	19	100	19	100
12.5	100	12.5	100	12.5	100
9.5	100	9.5	100	9.5	100
4.75	100	4.75	100	4.75	100
2	100	2	100	2	100
1.18	96.6	1.18	99.6	1.18	99.6
0.6	95.2	0.6	97.84	0.6	90.84
0.3	94.3	0.3	95.42	0.3	85.42
0.15	87.6	0.15	93.08	0.15	77.08
0.075	70.3	0.075	90.18	0.075	65.18
0.03145	40.3	0.04125	74.84214	0.025926	54.84
0.01628	20.4	0.02648	72.97108	0.01658	39.97
0.01125	10.1	0.01462	65.48687	0.009883	27.48
0.00746	0.9	0.00842	61.74476	0.007131	18.74
0.00543	0.2	0.00584	56.1316	0.005166	14.13
0.00315	0.1	0.00385	52.38949	0.002508	9.38
0.00108	0	0.00248	48.64739	0.001084	4.64

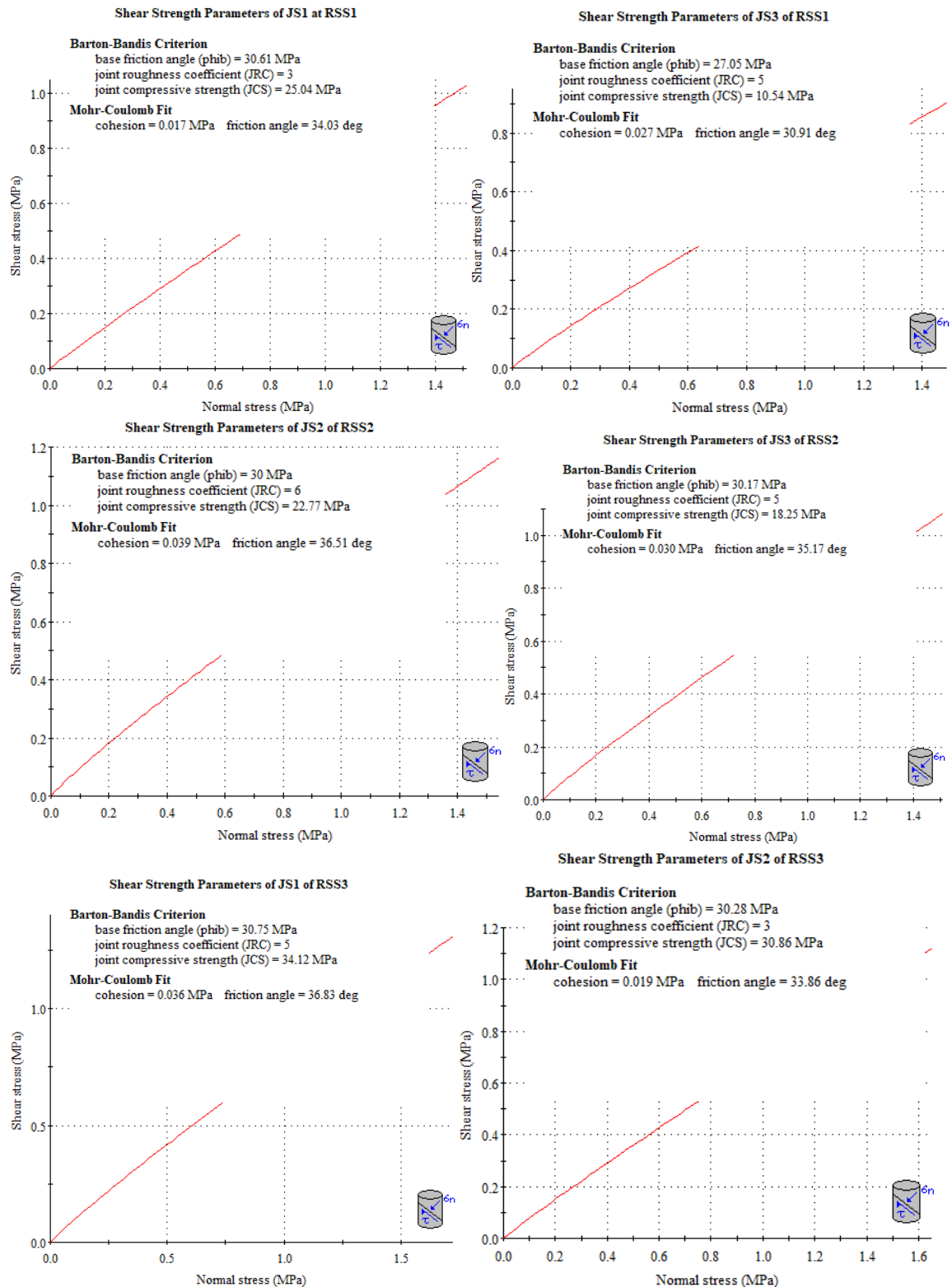


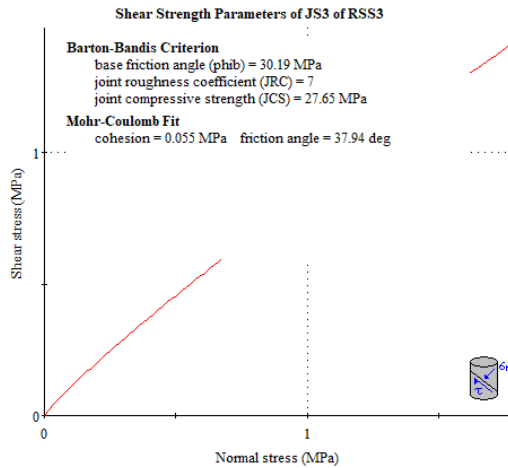
Appendix 3: Basic Rock mass rating System (Beinswski, 1989)

			Range of values						
1	Strengt h of intact rock	PL MPa	>10	10•4	4•2	2•1	For this low range UCS is preferred		
		UCS ,MPa	>250	100-250	50-100	25-50	25• 5	5• 1	< 1
	Rating		15	12	7	4	2	1	0
2	RQD		90-100	75-90	50-75	25-50	<25		
	Rating		20	17	13	8	3		
3	Spacing of		>2m	0.6-2m	200-600mm	60-200mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces, not continuous, no separation, weathered wall rock	Slightly rough surface, separation<1m m,highly weathered wall	Slightly rough surface, separation<1m m,highly weathered wall	Slicken sided surfaces, or gouge<5mm thick or separation 1-5mm continuous	Soft gouge >5mm thick, or separation >5mm		
	Rating		30	25	20	10	0		
5	GW	General	Completely dry	damp	wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
RMR rating value			100-81	80-61	60-41	40-21	<20		
Class no			V	IV	III	II	I		
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		

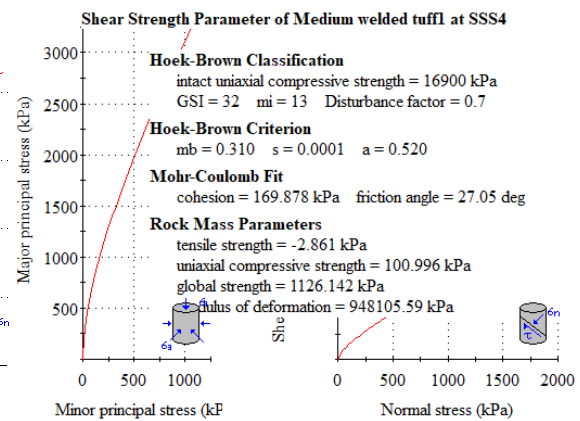
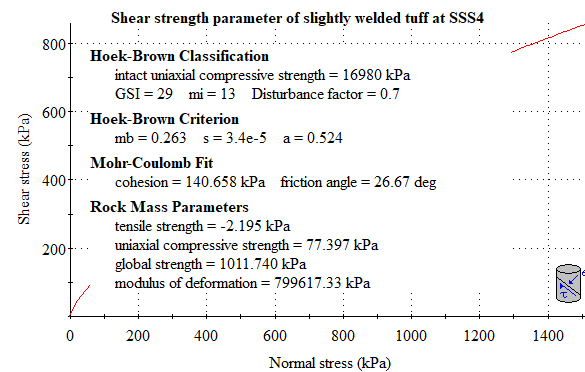
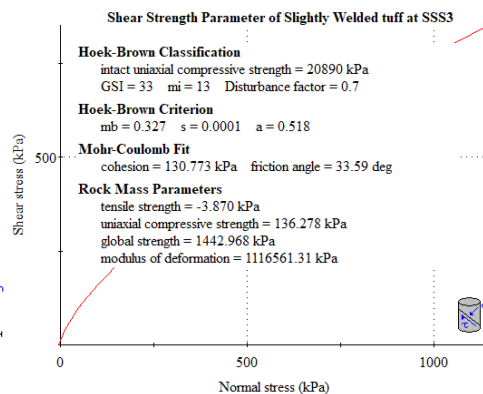
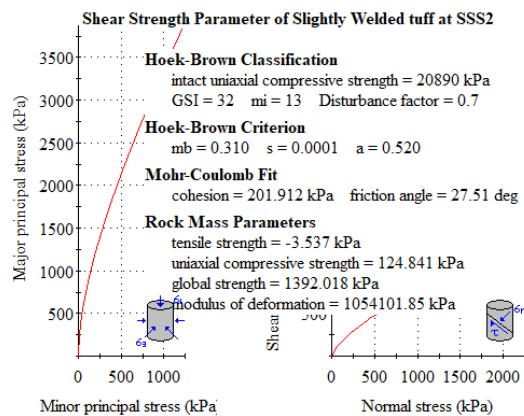
Appendix 4: The shear strength parameters of structural controlled slope sections as determining using Rockdata software using Barton & Bandis (1990) non-linear failure criterion.

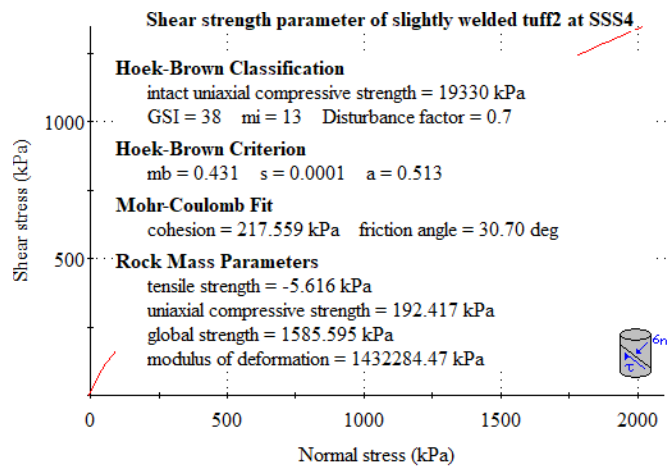
Slope section	Input parameters								Output result		
	Joint set	JRC	JCS (Mpa)	γ (MN/m ³)	phir	Sh (m)	R	r	Cohesion		Φ (°)
									C (Mpa)	t/m2	
RSS1	JS1	3	25.04	0.02124	30.61	55	53.0	48.0	0.017	1.734	34.03
	JS3	5	10.54	0.02124	27.05	55	32.9	24.1	0.027	2.753	30.91
RSS2	JS2	6	22.77	0.022712	30.00	50	47.5	42.0	0.039	3.977	36.51
	JS3	5	18.25	0.022712	30.17	50	42.7	38.0	0.030	3.059	35.17
RSS3	JS1	5	34.12	0.022589	30.75	57	56.6	52.0	0.036	3.671	36.83
	JS2	3	30.86	0.022589	30.28	57	54.4	48.0	0.019	1.937	33.86
	JS3	7	27.65	0.022589	30.19	57	52.0	46.2	0.055	5.608	37.94
JRC= joint roughness coefficient, JCS= joint compressive strength, phib= basic friction angle, γ = unit weight of rock, Sh= slope height, R= dry Schmidt rebound value, r= wet Schmidt rebound value, c= cohesion. Φ = friction angle											





Appendix 5: shear strength parameter of the nonstructural controlled slope section





Appendix 6: Approximate range of rock strength and their corresponding description ISRM (1978).

UCS range (MPa)	Description	UCS (MPa)	Description
0.25-1.0	Extreamly weak rock	50.0-100.0	Strong rock
1.0-5.0	Very weak rock	100.0-250.0	Very strong rock
5.0-25.0	Weak rock	>250.0	Extreamly strong rock
25.0-50.0	Medium strong rock		

Appendix 7: GSI built in rock data software.

Rock Type: General

GSI Selection: 50 OK

SURFACE CONDITIONS				
VERY GOOD	GOOD	FAIR	POOR	VERY POOR
STRUCTURE INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities				
BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets				
VERY BLOCKY- interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets				
BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity				
DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces				
LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes				

DECREASING INTERLOCKING OF ROCK PIECES ↓

DECREASING SURFACE QUALITY →

Appendix 8: Pck mi value for different rock types built in rock data software.

Pick Mi Value

List of Mi Values

Migmatite 29 ± 3
Norite 20 ± 5
Obsidian 19 ± 3
Peridotite 25 ± 5
Phyllites 7 ± 3
Porphyries 20 ± 5
Quartzites 20 ± 3
Rhyolite 25 ± 5
Sandstones 17 ± 4
Schists 10 ± 3
Shales 6 ± 2
Siltstones 7 ± 2
Slates 7 ± 4
Sparitic Limestones 10 ± 5
Tuff 13 ± 5

Selected Mi Value

Mi Value: 10

Filter List

☐ Rock Type

☒ Sedimentary
☐ Igneous
☐ Metamorphic

☐ Texture

☒ Coarse
☐ Medium
☐ Fine
☐ Very Fine

OK

Cancel

Appendix 9: Disyurbance factor built in rock data software.

Disturbance Factor D

Application: ☐ Tunnels ☒ Slopes



Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.

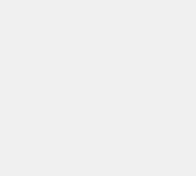
D=0.7
Good Blasting

D=1.0
Poor Blasting



Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal.

D=1.0
Production Blasting



In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.

D=0.7
Mechanical Excavation

Disturbance Factor: 0.7

OK

Cancel