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School of Graduate Studies

Collage of Natural Science

ASSESSMENT AND MITIGATION OF SLOPE STABILITY HAZARDS ALONG KOMBOLCHA- DESSE ROAD, NORTHERN ETHIOPIA

A Thesis Submitted to
The School of Graduate Studies of Addis Ababa University
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Engineering Geology

By

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ABSTRACT

The Kombolcha to Desse road, linking Addis Ababa with Northern Ethiopia towns traverses through one of the most difficult mountainous ranges in Ethiopia. Slope instability problems in the form of rock fall, rotational failure of colluvial material and debris slides are the common events in the area on the sides of the main road. The presence of loose unconsolidated materials (colluvial materials), highly weathered and fractured basalt rocks high relief, steep natural slopes, nature of geologic formations exposed along the road section, poor drainage conditions, occurrence of high seasonal rains, and seismically active nature of the region created favorable condition for slope instability in the area. Thus, keeping in mind all above points the present study was conceived to study in detail the slope stability condition of the area. It was realized that detailed slope stability studies along this road section are very necessary to identify critical slopes and to provide the best remedial measures to minimize the slope instability problems which frequently disrupt and endanger the traffic movement on this important road.

For the present study based on the field manifestation of instability two most critical slope sections were identified for detailed slope stability analysis. The deterministic slope stability analysis approach was followed to perform the detailed slope stability analysis of the selected slope sections. Factor of safety for the selected slope sections was determined for the different anticipated conditions (i.e., static and dynamic with varied water saturations) using Slope/W and Slide software. Both static and seismic slope stability analysis was carried out and factor of safety was deduced for each anticipated conditions. In general, detailed slope stability analysis of the two critical slope sections reveals that for only static dry condition both the slopes sections would be stable. However, for the rest anticipated conditions defined by static and dynamic situations with varied water saturations both critical slope sections would be unstable.

Moreover, the causes of slope instability in the study area are governed by different factors; therefore integrated approaches of remedial measures are more appropriate to mitigate the possible slope instability in the study area. Depending on site condition and slope stability analysis result four types of suitable preventive and remedial measures are recommended namely; proper managements of drainages, retaining structures, gabions, and managing steeply cut slopes.

Key words: Slope stability analysis, Sensitivity analysis, Factor of safety, Remedial measures, Static and Dynamic condition.

CHAPTER ONE INTRODUCTION

1.1 Introduction

Slope stability problems and associated catastrophes have been faced throughout history when human or nature has disrupted the delicate balance of natural soil and rock slopes (Abramson, 2001). Although, improvement in identification, prediction and mitigation measures is advanced, slope stability problem still trigger economic and environmental crises in mountainous region. This is partly due to the complicity of the processes driving slope failure and our inadequate knowledge for prediction ground condition. Nevertheless, different researchers and experts have been used various slope stability analysis methods and stabilization techniques in order to minimize the problem related to slope failure.

Amongst the several methods, the Limit equilibrium is one of the methods which is most widely used and accepted for analyzing slope stability problems. Therefore, the current study is entirely focused on Limit equilibrium slope stability analysis. For analysis purpose in the present study Slope/W and Slide software has been utilized to analyze the probability of failure along the road cut slopes in the study area. The minimum factor of safety has been computed or calculated by the software's using several parameters and for analysis values were adopted from the soil and rock investigation reports.

The road which passes on the hilly and mountainous terrains is characterized by variable topographical, geological, and hydrological and land-use conditions and is frequently affected by slope failures. This is because, the road crosses on hilly and mountainous terrains experience both deep and shallow excavations in the construction stage, that disturb the inherent nature of rock and soil slopes. Furthermore, rock or soil cut slope along road fail due to various factors such as; seismic activity, high groundwater pressures (after heavy rainfall), geological factors, and human activities can trigger large rock/soil blocks or even larger assemblages of rock to crash down on to the road surface below (Budetta, 2004).

Therefore, assessment of the slopes instability requires comprehensive information about the geology, groundwater, seismicity and engineering geology of the area. Slope stability analysis is often carried out in order to ensure that the analyzed slope can be made safe and probability of slope failure is minimized (Abramson, 2001).

Hence, analysis technique chosen depends on both site conditions and the potential mode of failure, with careful consideration being given to the varying strengths, weaknesses and limitations inherent in methodology (Abramson, 2001). Analyses must be based upon a model that accurately represents site sub-surface conditions, ground behavior, and applied loads (Abramson, 2001).

1.2 Statement of the problem

Slope failures are always catastrophic due to their large affected areas and great energy, generated by the collapsed soil or rocks with rapid and long run-out movement. This study has very important scientific significance and practical worth.

Most of constructed road in the highland of Ethiopia cross within valleys, hilly and mountainous terrain. Previous studies shown that, the slopes along roads are affected several times by slope instability (landslide) problems (Lulseged Ayalew, 1999; Tenalem Ayenew and Barebieri, 2005). The main triggering factors for the instabilities of the areas are the different environmental factors (e.g. intensive rainy summer), variable geological and structural elements (weak rocks, slide debris weak soils, shear zones, and faults) difficult road characteristics (narrow roads with tight horizontal and vertical curvature (Kifle Woldearegay, 2013).

Road plays very important role for development of any country. Kombolcha-Desse road is the main route which connects Northern Ethiopian cities (Mekel, Aksum, Desse and others) with capital city of Addis Ababa. Like other roads and highways that were constructed on highland areas, the road cut slopes of study area are very susceptible to failures especially during rainy season. The rock falls, debris flows and colluvial material slides hinder the traffic along these road sections. Therefore, in order to provide a safe access for transportation and economic development the proper functioning of this road is very essential. Indeed, construction of road is often accompanied with rock and soil slope cut. For that reason, the stability of the slope is always of paramount importance during the lifetime of the structures (Tenalem Ayalew et al., 1989).

Due to slope instability problem Ethiopian Roads Authority (ERA) used different remedial measures (retaining wall, surface drainage and gabions) in order to prevent damage along the road.

Even though, counter-measures applied on some of the critical portions of road cuts, but most of slopes are not mitigated and analyzed. The current study has embark on a comprehensive evaluation of the stability of all cut slopes along road, with a major objective to identify the most hazardous zones. Besides, the present study is also forwarding the general remedial measures which may be adopted to minimize or to eliminate the possible hazard along the road. This will be accomplished by using different methodologies and techniques based on slope stability result and site conditions.

1.3 Description of Study Area

1.3.1 Location and Accessibility

The study area is located in the in Northern Ethiopia, Amhara regional state along Kombolcha to Desse road. It connects Northern Ethiopian cities (Mekel, Aksum, Desse, Kombolcha and others) with capital city of Addis Ababa (Fig.1.1).

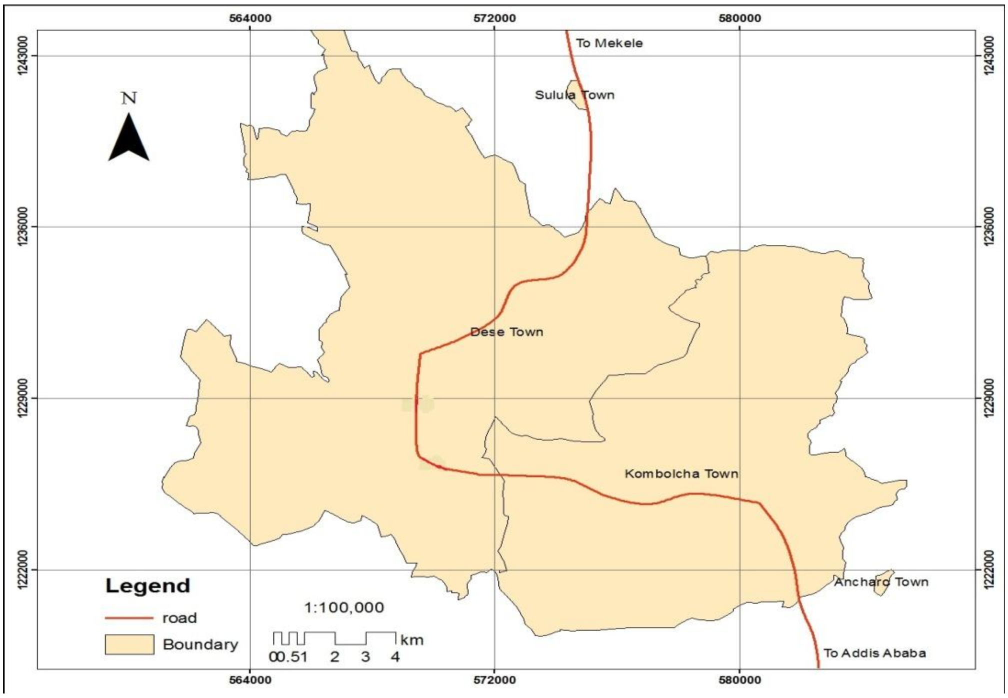


Fig.1.1 Location map of the study area

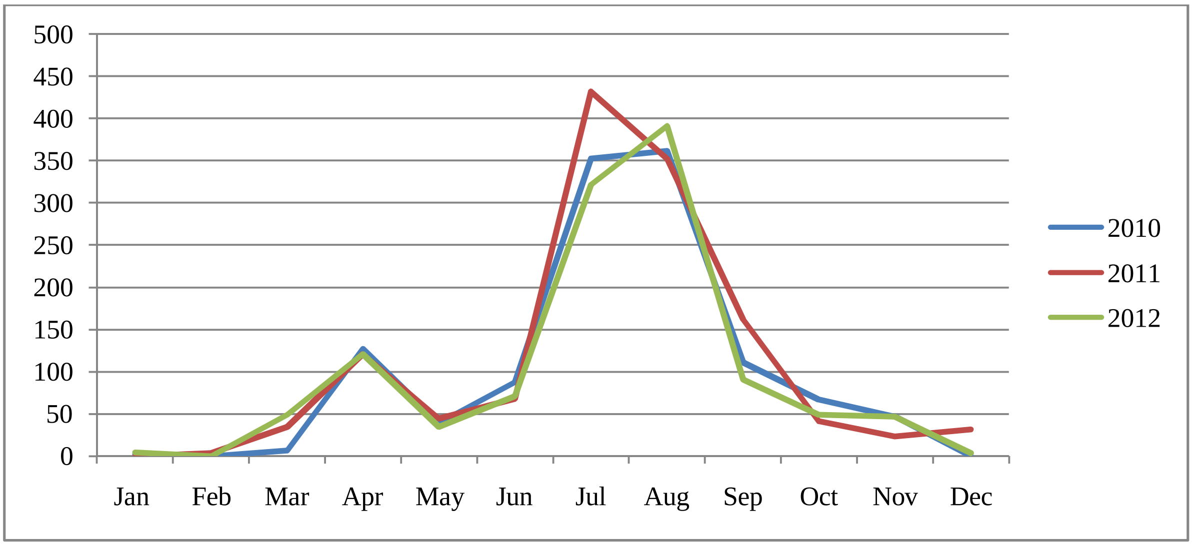
Kombolcha - Desse road passes through an extremely rugged terrain which is generally characterized by steep hill slopes and complex topographical features. It is about 400 km from Addis Ababa, the capital city of Ethiopia.

1.3.2 Climate

Examining the variability of climate that is responsible for the coming effect on an area needs the knowledge of the climatology of that area. This means knowing the long-term mean values of climatic parameters such as; temperature, rainfall, etc. and their degree of variability or deviation from the mean is very important (Guzzetti, et al, 2008).

The climate of study area is sub-humid to humid with average annual rainfall of 1385 mm, which is quite high compared to many places in the north-western highlands of Ethiopia (Gebreslassie Mebrahatu, 2011).

The long-term average annual temperature of Desse varies from 12 to 18 °C (Gebreslassie Mebrahatu, 2011). Geographically the area is in bounded between UTM 37 N coordinates of 11° 5' 28" N to 11° 10' 20" N latitude and 39° 36' 49" E to 39° 40' 40" E longitude. The graph, (Fig.1.2) shows that, from June to September rainfall is high. The road cut slopes becomes very susceptible to slope instability problems during these months.



(Source: National metrological agency)

Fig.1.2 Monthly rainfalls in mm (2010-2012)

1.3.3 Physiography and Drainage Pattern

The study area is generally characterized by highly variable topography features and complex geology, which reflect of the past geological and erosion process. The landscape includes plateaus, steep hillslopes, and deeply incised valleys and gorges. Much of the elevation of the area ranges from about 1800 to 3500m above sea level. Many of the hillslopes are steep enough to reach the limit equilibrium state, whereby external factors such as rainfall infiltration and/or excavations (artificial or natural) could trigger slope failures. The drainage shows well defined with parallel to sub-parallel patterns developed along joints of hard rocks (Tenalem Ayalew et al., 2009).

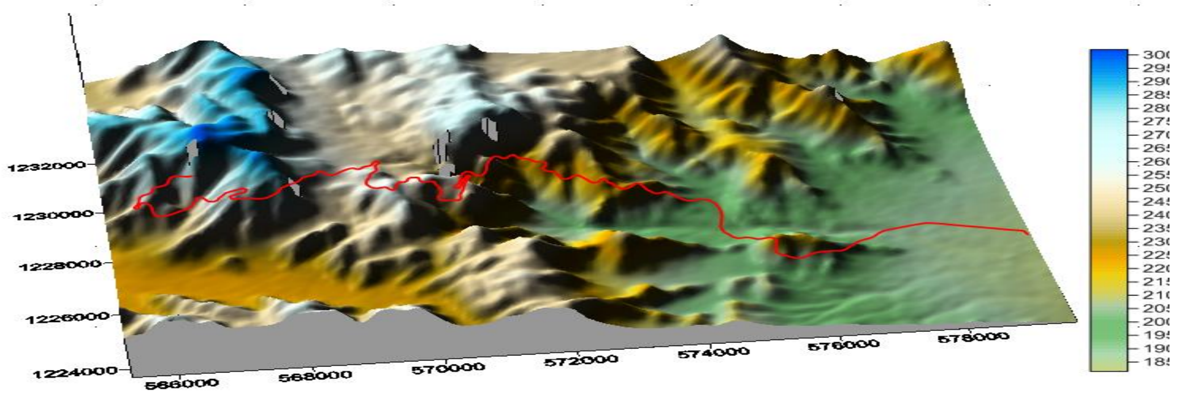


Fig.1.3 Topography and drainage pattern of the study area

1.3.4 Geology

The study area comprises different types of lithology such as: Tertiary Trap Series volcanics, Quaternary alluvial–colluvial deposits, and residual soils. Gregnanin et al. (1978) identified the following lithological units as cited by Tenalem Ayenew and Barbieri (2005). GSE, (2010) has also prepared a new geological map of the study area (Fig1.4).

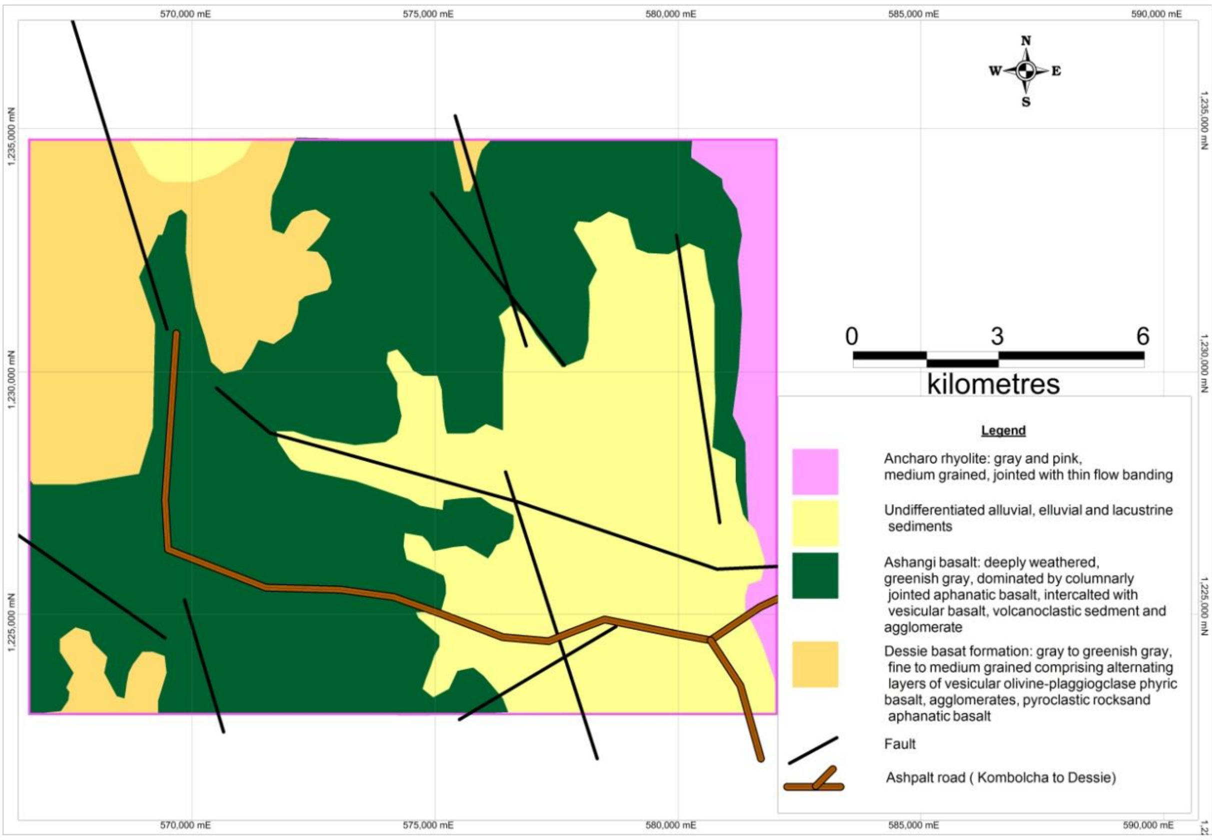
Quaternary deposits (Colluvial, alluvial and lacustrine)

These include alluvial and residual soils, colluvial or talus deposits, and patchy lacustrine deposits. Alluvial deposits are restricted to low-lying areas close to river courses and deeply incised gullies in the central part of the area. They consist of cobbles and gravels of basaltic origin with a matrix of silty clay soils. The river beds are made of big boulders and cobbles. Residual soils are mostly humus-rich and are underlain by extremely weathered basalt converted to silty clay and clay soils followed by moderately weathered basaltic rock, at places associated with volcanic pyroclastic materials and thin layers of volcanic ash.

Colluvial or talus deposits form small fans at the feet of hills and steep slope areas to the south, east, and west. The talus material is often big blocks of basalt associated with weathered friable loose basaltic material that fell from the steep cliffs. Fine-grained lacustrine deposits are confined in the central part of the city of Kombolcha and Desse.

Desse basalt formation (Stratoid and degraded basalt)

Stratoid or layered basalt covers most western part of the study area and is often inter bedded with thin paleosoil horizons; in places, vesicular basalts overlie the stratoid basalt. The distribution of geology over the study area is shown in (Fig. 1.4).



(Source: Simplified from Geological Survey of Ethiopia (GSE), 2010)

Fig.1.4 Geological map of the study area

Degraded basalts are highly weathered rocks. It is the main source of colluvial and alluvial deposits that originate in the elevated areas and are deposited in low-lying areas due to slope and fluvial processes. The basalt bedrock outcrops along the escarpment, road cuts, and hillsides are highly jointed, forming systematic and non-systematic joint sets following the major N–S and E–W tectonic trends. Most of the joints are open, with spacing ranging from a few centimeters up to 4 m. The large and dense joints favor rock fall and toppling.

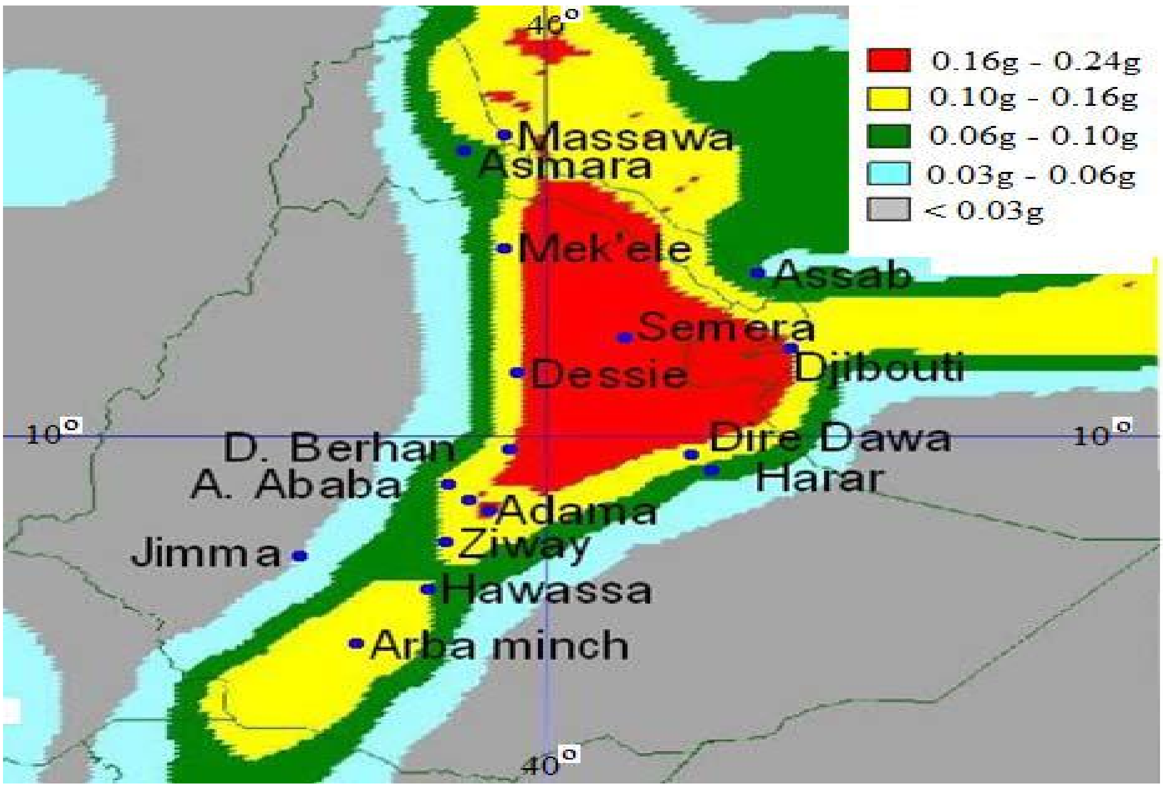
Vesicular basalt is the most dominant lithology which extends in south and south eastern region of the study area, exposed along the road cuts, hillsides, and on the banks of River. It is highly porous and locally friable.

1.3.5 Geomorphology and Geological Structure

The study area comprises different type minor and major geological structures such as; faults, systematic, non systematic joints, structurally controlled alluvial and denudational landforms. Every lithology has its own geomorphic features. Many mass movements and erosional processes modified escarpments. From the geomorphological point of view, the graben comprises different landform units, such as; fault escarpments, talus belts, fluvio-denudational slopes, graben floor, alluvial fans, terraces and river beds. The loose unconsolidated materials have been eroded and transported to low elevation areas by such processes as heavy rain, sheet wash, and gullying.

1.3.6 Seismicity of the study area

The study area is located in second degree earthquake zone according to seismic zoning map of Ethiopia (Gouin, 1976). Several earthquakes have occurred in the past in the study area (Gouin, 1979). However, the role of earthquakes in the recent landslide activity is not well understood (Tenalem Ayenew and Barbieri, 2005). According to the seismic hazard map of Ethiopia; the study area is concentrated near and around the Afar region characterized by a PGA of 0.16g to 0.24g, shown in (Asrat Worku, 1995) (Fig.1.5). The ground accelerations associated with seismic events can induce significant inertia forces that may lead to instability and permanent deformations of natural and man-made slopes (Abramson, 2001).



(Source: Asrat Worku, 2011)

Fig.1.5 The seismic hazard map of Ethiopia based on the GSHAP data for a return period of 475 years

1.4 Objective of the study

1.4.1 General Objective

- ✓ To identify critical slopes in the study area and to evolve appropriate mitigation measures for the potentially unstable cut slopes.
- ✓ To conduct a detail slope stability analysis on selected critical slope sections along the road

1.4.2 Specific Objective

- ✓ Delineation and identification of the critical road cut slopes along the road section.
- ✓ Determination of the Geotechnical and Engineering geological properties of the soil and rock mass along critical slope section
- ✓ To prepare cross sections along critical slope sections and to deduce slope geometry and geological sections.
- ✓ Conducting slope stability of critical slope sections along the road for anticipated and adverse conditions.
- ✓ To develop mitigation strategy for each section of the road and to suggest alternative solutions and designs to minimize future problems.

1.5 Importance of the study

The pervious landslide events were very devastating and it has severely influence transportation activity. In 1977, two people were killed by a seismically induced landslide (Gouin, 1979) and in 1994, landslides triggered by heavy rainfall blocked a segment of the road and destroyed a bridge and buckled the foundations of several houses.

Kombolcha to Desse road links many important towns of Northern Ethiopia to the central Ethiopia. It has long-drawn-out slope instability problems. Therefore, the present study is very important from the point of view of identifying and understanding the possible causes of the slope instabilities along the road. In addition to identifying the slope instability causes in the present study an attempt was also made to delineate the areas which are potentially susceptible for future slope problems. Furthermore, for those slopes where the results of the stability analysis indicate that the roadway slope do not meet the factor of safety requirement, both preventive stabilization and remedial measures are suggested on the potential unstable slope sections.

1.6 Methodology and materials used

1.6.1 Materials used

Based on the availability and relevance, various kinds of data/ information were collected from different sources. The material used during the present study was;

- Topographic map (at scale of 1:50,000 from Ethiopian Map Agency)
- Geological map (from literature)
- Slope/W software (Ethiopian Road Authority) and Slide software.
- Metrological data (National Metrological Agency)
- GPS
- Brunton Compass

1.6.2 Methods

In order to achieve the objectives of the present study following systematic methodology was followed;

- Review of literature from different sources both published and unpublished reports which were used to get priority information about the study area and techniques that used in related researches.
- Preparation of geological map from literature (Ethiopian geological survey)
- Establishment of a field reconnaissance program classifying the cuts and slopes along the road into stations according to geology, structure and location;
- Determination of the geographical location of each station using global positioning equipment (GPS);
- Identification of all cuts and slopes to identify those that could pose a problem to the road; and,
- Selection of critical slope sections based on field manifestation
- Application of detailed field investigations to collect rock and soil samples for laboratory testing and determining the different parameters needed to define the characteristics and the material strength parameters (friction angle and cohesion) of the geological materials
- Slope Stability analysis was carried out by using Slope/W (Geo-studio, 2007) and Silde software's in order to get the FOS of critical slope sections

1.7 Outcomes of the study

The current study mainly focuses firstly, on slope stability analysis after delineation and identification of critical slopes along the road section and, secondly, for those slope sections where the result of the stability analysis indicates that the roadway slope does not meet the factor of safety requirement, both preventive stabilization and remedial measures are suggested on the potential unstable slope sections.

Further, Ethiopian Roads Authority (ERA) is spending much of their resource for remedial measures (retaining wall, surface drainage and gabions) to mitigate the threat and damage caused by slope instability problems along the road. The recommendation made through the present study may help in deciding permanent remedial measures.

Thus, these remedial measures may help to tackle the reoccurring traffic obstacles during natural disaster such as, rainfall and earthquake. The present study may be helpful to Ethiopian Road Authority. Besides, the present study may also be utilized by future researchers.

1.8 Scope and limitation of the Study

Within the scope of this study, first of all a detailed literature survey including slope stability analysis and remediation methods were reviewed. As a second stage systematic field work was conducted. Field observations on critical slope sections were made and all pertinent data for slope stability analysis was collected. Besides, soil samples were collected which were later tested in laboratory to know the shear strength parameter of the soil and using the result as basis for sensitivity analysis. In the final stage the slope was modeled using slope stability analysis software and the most suitable stabilization technique under static and dynamic conditions were deduced. The primary objective of a stability analysis is to determine the factor of safety (FOS) of a particular slope, to predict when failure is imminent, and to assess remedial treatments when necessary. Therefore, to apply slope stability principles properly, geology, hydrology and soil and rock properties should be understood well. However, due to constraints of secondary data (ground water level, borehole data) resources, time and finance some of these properties were not actually determined such as; assessment of actual ground water regime in the critical slope sections.

However, despite all these difficulties and limitations, all efforts were being made to present the results and findings in a systematic manner, which were all supported by the actual field observation and laboratory testing.

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analysis is about to get the post construction factor of safety and to ensure that the adequacy of the slope design.
There are many form of slope failure and it will depend on the types of slope. For existing slopes the stability of a slope need to be analyzed as soon as soil or rock movement in slope is detached.

Gordon and Griffiths (2005) stated that the failure prediction of a soil and rock slope has been long standing geotechnical problems, which attracted a wide variety of solutions.

The primary objective of a stability analysis is to determine the factor of safety (FS) of a particular slope, to predict when failure is imminent, and to assess remedial treatments when necessary. Therefore, to apply slope stability principles properly, geology, hydrology and soil and rock properties should be understood well. Site conditions must be applied precisely to the model for analysis.

Engineering judgments must be based on assessing the results of analyses considering acceptable risk or safety factors (Abramson et al., 2001).

Nowadays, slope stability analyses are performed by computers and no longer by hand calculations. Several slope stability computer programs are available from public or commercial domains. In the present study, both Slope/W and Slide software were used for the analysis and determination of Factor of Safety (FOS) of slope stability for critical slope sections.

Generally, this chapter emphasize on, types of slope failure, factors that affect slope stability, slope stability analysis methods and different type of slope stabilization methods used currently.

2.1 Types of slope

Slopes can be categorized as natural or manmade (cut and fill) slope related to geology and civil engineering knowledge.

2.1.1 Natural slope

Natural slopes are the slope that formed naturally as hillside or stream banks. The slopes are being formed long time ago by the movement of earth and earthquake. These slopes normally will remain intact as long as there is no development or construction on it. Due to the influence of various kinds of forces both internal (natural process) and external (human effect) these slopes may show instabilities.

2.1.2 Cut Slope

Cut slope is the slope that formed by cutting the natural slope surfaces in order to give way for development (i.e. road construction and tunnel development).

2.2 Factors influencing slope stability

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

2.2.1 Force of gravity

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

2.2.2 Hydrologic factor

Water plays major roles in both solid rock and soil mass. Water can reduce shear strength and thereby promoting the movement of rocks and sediment down slope under the pull of gravity. Water reduces shear strength by creating positive pressure in the pore spaces of earth materials. Water infiltrating into slope materials can saturates the soil particles at depth by filling the pore spaces. The weight of water lying above creates water pressure that drives soil particles apart (Sidle, 1982).

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

2.2.3 Seismicity

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

2.2.4 Land cover change

Land cover change has been recognized throughout the world as one of the most important factors influencing the occurrence of rainfall-triggered slope failures (Cruden and Varnes, 1996). Deforestation increases the probability of slope instability. In developing countries people have cut down trees and removed vegetation to build their houses. The roots of this vegetation bind the soil together and protect it from heavy rainfall keeping the slope stable but, if vegetation is removed the slope is exposed to risk of slope failure. Many slope failures occur on areas that have undergone significant deforestation (Tenalem Ayenew, 2005).

2.2.5 Anthropogenic (human) factors

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

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

- ✓ Overloading of slopes during mining and quarrying operations. This extra weight may increase the chance of slope failure; altering the hydrology may have dramatic effects on slope stability (Long, 2008).
- ✓ Deforestation of trees due to human intervention promotes soil slope failure, when the roots of tree are no longer binding the soil together and unable to protect it from heavy rainfall. It also reduces evapo-transpiration and raises the water tables (Varnes, 1996).

- ✓ Vibrations resulted from artificial causes such as; machine activities and underground explosions cause serious flooding and sediment failure in big reservoirs.

2.2.6 Geologic factors

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

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

Geological structures: geological structures such as; bedding, joints, foliation, cleavage, schistosity, and faults are potentially weak planes in a slope.

Their strength is generally less as compared to the strength of surrounding intact rock. Tectonic setting of the area also contributes slope instability by fracturing, faulting, jointing and foliation structures (Hoek and Bray, 1998). Fault and fracture zones indicate weak zones; therefore they may be favorable planes of weakness where failures occur.

Therefore, it is important to know their orientation in relation to slope angle, direction, and strength along such potential weak planes (Sidle, 2006).

Furthermore, Varnes (1984) indicate that the degree of fracturing and shearing plays a significant role in determining slope stability.

2.2.7 Geomorphic factors

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

Slope gradient; with increasing slope gradient, the shear stress increases due to the effect of gravity thus, down slope movement of material is enhanced. According to Carson and Petley (1970) slope gradient is taken as the main driving forces of mass movement, especially for shallow landslides. In most cases of landslide assessment, slope gradient is taken as main causative factor (Swanston and Dryness, 1973).

2.2.8 Slope aspect and shape

Slope aspect refers the direction to which the slope is facing. It strongly affects hydrological processes via differential evaporation, hence affects weathering process and root development, especially in drier environments (Sidle and Ochiai, 2006). Aspect is closely related to the bedrock structure particularly in metamorphic rocks. Failures of rocks are common on slope aspect oriented parallel to the direction of foliation and lineation planes (Vieira and Fernandez, 2004).

Shape of the slope has great influence on the slop stability of steep terrain by concentrating or dispersing surface and primarily subsurface water in the landscape (Sidle and Ochiai, 2006). Three hydrogeomorphic slope units important in assessing terrain stability are; (a) divergent, (b) planar and (c) convergent slope shapes have different degree of infiltrating water. In divergent slope, subsurface (and surface) water is dispersed rapidly; thus pore pressure is typically lower than other slope forms.

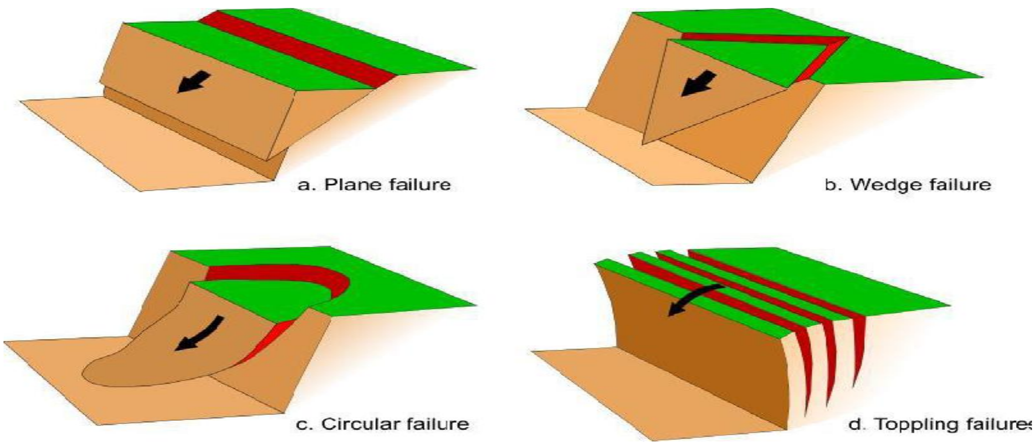
Convergent slope tends to concentrate subsurface water into small area of the slope, thereby high pore water pressure develops which reduces shear strength of the material, thus, finally promotes down slope failure. Planar slopes are intermediate in susceptibility to landslide between divergent and convergent slopes.

2.2.9 Engineering properties of soils

Slope stability analyses by methods of limiting equilibrium require a quantitative determination of soil shear strength. This provides insights into important engineering properties that contribute to slope stability and the relationship of these properties to other characteristics of soil materials (Wu and Sangrey, 1978). Soil shear strength is a fundamental property that governs the stability of natural and manmade slopes. Shear strength of soil material is basically defined as a function of different shear strength parameters namely, normal stress on the slip surface, cohesion, and internal angle of friction. Increasing soil moisture reduces soil cohesion (Baver et al., 1972); thus, soil strength declines. As the amount of clay content in soil increases, cohesion of the material typically increases proportionately with clay content because of the greater surface contact area associated with clay particles. Internal angle of friction is most important in low cohesion soils.

2.3 Type of slope failure

According to Hoek E., (2009) failures in rock slopes can be classified into four types; (i) Plane mode of failure, (ii) Wedge mode of failure, (iii) Circular or Rotational mode of failure, and (iv) Toppling mode of failure. Kliche (1999) also states that there are four general modes of slope failure: planar failure, rotational failure, wedge failure and toppling failure.



(Source: Hoek E., 2009)

Fig.2.1 Simplified illustrations of most common slope failure modes

2.3.1 Circular (Rotational) Failure

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

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

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

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

This study is mainly concentrated on circular mode of failure. All the failures exist along the road are circular types.

2.3.2 Plane failure

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

Conditions for appearance of planar failure:

- ✓ The strike of the slope doesn´t differs more than ± 20° from the strike of the weakness plane.
- ✓ The toe of the failure plane has to cross the slope between toe and crest.

- ✓ The dip of the failure plane must be less than the dip of the slope face, and the internal angle of friction for the discontinuity must be less than the dip of the discontinuity (Hoek and Bray, 1981).

2.3.4 Wedge failure

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

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

2.3.4 Toppling

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

Toppling will occur if the vector representing the weight of the block falls outside the base and this will occur if the ratio of base to height (b/h) < tan Ψp. Where; 'Ψp' is the dip of the plane, 'h' is the height of the rock block and 'b' is the width. Toppling failures in rock are structurally controlled, and occur under very strict geometric conditions (b/h relations, dip angle and spacing of the joint sets); toppling phenomena are almost independent of the shear strength of the rock joints (Hoek and Bray, 1981).

2.3.5 Rock fall

When rock blocks detach from the rock mass and fall freely under gravity it is known as rock fall. The rock blocks may role down, bounce or slide along the surface. These are characterized by movement away from existing discontinuities, such as; joints, fissures, steeply-inclined bedding planes, fault planes, etc. and within which the slope failure assisted or precipitated by the effects of water or ice pressure (Roy, 2001). Rock fall is trigger lot of problems in safety and maintenance of highways in mountainous terrains.

2.4 Slope stability analysis methods

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

2.4.1 Limit equilibrium method

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

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

2.4.2 Finite element method

The finite element method is a powerful calculating method in engineering sciences. This method is by far method used for analyzing geotechnical problems. Unlike the limit equilibrium method, the finite element method considers linear and non-linear stress – strain behavior of the soil in calculating the shear stress for the analysis. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffiths and Lane 1999).

Today, new analysis method in engineering can be studied with ‘FEM’ as reference of exact solution. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffiths and Lane 1999).

In the last two decades, several methods were presented for slope stability analysis by ‘FEM’. Through this method, gravity increase method and ‘strength reduction method’ have widely used. In the gravity increase method, the gravity forces, such as; weight, increase gradually until the slope becomes unstable. Then factor of safety define ratio between gravitational acceleration in failure time and actual gravitational acceleration. In the ‘strength reduction method’, strength parameters of the slope are decreased until the slope become unstable. Hence, the safety of factor determine ratio between actual strength parameters and critical strength parameters. The gravity increase method is well suited for analyses of stability of embankment (Swan, 1999).

2.4.3 Numerical Analysis Methods

Numerical analysis methods give reasonable approximations to the correct mathematical solution of the governing equations of the mechanics of slope stability.

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

2.5 Slice Methods

The slice methods can be divided into two groups: non rigorous and rigorous. Non-rigorous methods satisfy either force or moment equilibrium, whereas rigorous methods satisfy both force and moment equilibrium. The factor of safety estimated from rigorous methods is relatively insensitive to the assumptions made to obtain determinacy (Duncan, 1992).

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

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

Common to all slice methods is the assumption that the assumed soil mass and failure surface can be divided into a finite number of slices. Equilibrium conditions are considered for all slices. The problem is strongly indeterminate, requiring several basic assumptions regarding the location of application or resultant directions of applied forces (Abramson et al., 2002).

Table 2.1 Equilibrium Conditions Satisfied by Limit Equilibrium Methods

Methods	**Force Equilibrium**		**Moment Equilibrium**
	Vertical	**Horizontal**	
Ordinary Method of Slice	No	No	Yes
Bishops Simplified	Yes	No	Yes
Janbus Simplified	Yes	Yes	No
Bishops Rigorous	Yes	Yes	Yes
Janbus Generalized	Yes	Yes	Yes
Spencer	Yes	Yes	Yes
Morgenstern-Price(M-P)	Yes	Yes	Yes

(Source: Abramson et al., 2002)

However, non rigorous solutions can produce significantly different estimates of safety depending on the assumptions made. In general, a non rigorous solution satisfying only moment equilibrium is superior to one satisfying only force equilibrium and will provide solutions close to a rigorous method.

There are different types of slice analysis methods based on limit equilibrium principles (Abramson et al., 2002) such as;

2.5.1 Ordinary Method of Slices (OMS)

This is one of the earliest analytical techniques based on the method of slices and limit equilibrium principles. It satisfies the moment equilibrium for a circular slip surface, but neglects both the inter slice normal and shear forces. The advantage of this method is its simplicity in solving the Factor of Safety (FOS), since the equation does not require an iteration process. The 'FOS' is based on moment equilibrium (Abramson et al., 2002). Horizontal and vertical equilibrium conditions are not satisfied. In addition, it fails to satisfy inter slice equilibrium where adjacent slices have different base angles. For such cases, the assumption is equivalent to specifying different inclination angles for the same inter slice force between adjacent slices. This will generally lead to the calculation of inconsistent effective stresses at the base of the slices (Abramson et al., 2002).

2.5.2 Bishop's Simplified Method

Bishop's simplified method (BSM) is very common in practice for circular shear surface (SS). This method considers the inter slice normal forces but neglects the inter slice shear forces (Abramson et al., 2002).

2.5.3 Janbu's methods

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

2.5.3.1 Simplified Janbu Method

Janbu et al. (1954) have proposed this simplified method for routine calculations, which could be implemented without the need for computers. The expression for the factor of safety is derived based on force equilibrium conditions. In this simplified method, the inter slice shear forces are assumed to be zero. This method is a simplified variation of Janbu's rigorous method and is applicable to general shape of slip surfaces

2.5.3.2 Janbu's generalized method

Janbu's generalized method (JGM) is also called as Janbu's generalized procedure of slices (GPS) (Janbu, 1973). In this method, the both slice forces are considered and it is assumed as a line of thrust to determine a relationship between both slice forces. Hence, the 'FOS' becomes a complex function with both slice forces (Janbu, 1954).

2.5.3.3 Janbu's direct method

Another Janbu's method is called Janbu's direct method (JDM). It is based on dimensionless parameters and the series of stability charts (Janbu, 1954). These charts provide a powerful tool to carry out slope stability analysis. It also includes various load conditions such as; groundwater, surcharge and tension cracks. In addition to this, the method can be used for both total and effective stress analyses. The FOS for cohesive and frictional soils can be computed by Janbu direct methods (Janbu, 1996).

2.5.4 Morgenstern and Price Method

This method was developed by Morgenstern and Price (1965) which considers not only the normal and tangential equilibrium but also the moment equilibrium for each slice. In this method, a simplifying assumption is made regarding the relationship between the inter slice shear force and the normal force. This method needs a lot of individual judgment. For example, when the assumption has been made and the output has been obtained from the computer, all the quantities must be examined to determine whether they seem reasonable. Morgenstern presented a comprehensive and rigorous method of solution that satisfies all static equilibrium conditions and that is applicable to slip surfaces of general shape. The Morgenstern method is similar to the other method which also satisfies both force and moment equilibriums and also assumes the inter slice force function. To attain statical determinacy of the problem, they recommended making assumptions on inter slice force relations, with the intention of avoiding numerical difficulties that may evolve in the solution procedure as well as obtaining reasonable results.

2.5.5 Spencer's Method

The Spencer's method (SM) concept is considered same as M-PM' except the assumption made for inter slice forces. In this method, a constant inclination is assumed for inter slice forces and the FOS is computed for both equilibriums (Spencer, 1967). Spencer (1967) presented slope stability analysis method that satisfies all conditions of equilibrium for circular slip surfaces. Later, he generalized and modified his method to adopt it to general slip surfaces.

2.5.6 Sarma's Method

Sarma (1973) presented a different approach of attaining solution within the framework of limit equilibrium principles and method of slices. In his method, rather than iterating for the factor of safety, the critical horizontal acceleration coefficient, Kc, that is required to bring the mass of soil bounded by the slip surface and the free surface of the slope to limiting equilibrium, is computed. This critical acceleration coefficient (Kc) is supposed to be a measure of the factor of safety of the slope. Sarma's method satisfies all conditions of equilibrium and is applicable to general-shape slip surfaces.

In Sarma's method, it is assumed that under the action of horizontal thrust (Kc multiplied by the weight of the soil above the slip surface, W), the complete shear strength of the soil along the slip surface is mobilized (i.e. F=1 along the slip surface).

2.6 Slope/W software

SLOPE/W, developed by GEO-SLOPE International Canada, is used for slope stability analysis. This software is based on the theories and principles of the LE methods discussed in the previous sections. It is the leading software product that used for computing the factor of safety of earth and rock slopes. In this study, SLOPE/W has been applied separately and together with Quake/W, which computes the dynamic condition of slope. The software SLOPE/W computes FOS for various shear surfaces, for example circular, non-circular and user-defined surfaces (SLOPE/W 2002, Krahn, 2004). This software works on a limit equilibrium framework and includes methods such as; the Morgenstern-Price, Spencer's method, Bishop's simplified method, Janbu's generalized method and Ordinary method of slices etc.

Illustrative example are provided in the SLOPE/W manual (2007) for verification of the analyses using SLOPE/W program. These examples show a detailed comparison of the analysis results from SLOPE/W with solutions obtained from the Stability charts developed by Bishop and Morgenstern (1960), a comparison with published results and a comparison with theoretical calculations of earth pressures. The analyses results from SLOPE/W prove to be the same as the values obtained from the other methods, indicating that the results obtained using SLOPE/W program are reliable.

2.7 Slide software

Slide software, developed by Rocscience Inc Toronto Canada, is also used for slope stability analysis for soil and rock slopes. The software is also 2D-LE based computer program, which can be applied to evaluate the stability for circular or non-circular failure surfaces (Slide, 2003).

In fact, Slide is found similar to the Slope/W though there are few additional features, for example groundwater analysis and back analysis for support forces. Modelling in Slide for the study was possible for external loading, groundwater and forces, like surcharge and from pseudo-static earthquakes. The circular CSS (Critical Slope Surface) was located automatically and the corresponding FOS was computed by the software in the similar manner as in Slope/W.

In the present study in addition to Slope/W Slide software is used for the limit equilibrium slope stability analysis for critical slopes for the reasons that: (a) software is freely available using on line hardlock, (b) it is friendly software for user, and has similar features and functions as that of Slope/W software.

Modeling in Slide software for the slope analysis was possible for external loading, groundwater and forces and from pseudo-static earthquakes. The methods such as Bishop Simplified (BS), Janbu Simplified (JS), Spencer (SP), and Global Limit Equilibrium/Morgenstern-Price (GLE/M-P) methods are selected for the slope stability analysis in the Slide software.

2.8 Shear strength characterization

Shear strength is the main concern in slope stability analyses. Determination of the shear strength is a sensitive work and understanding the theory is essential in order to conduct analysis successfully.

The limit equilibrium and numerical methods used for evaluating the stability of slopes require an accurate and reliable estimate of the in situ shear strength of the slope materials.

However, the shear strength parameters are strongly influenced by many complex conditions, including the in situ state of stress, drainage, loading rates and soil or rock composition (Abramson, 2001).

Shear strength parameters used as input data for slope stability analysis in the current study are discussed below;

2.8.1 Cohesion and Friction angle

Cohesion '*C*' and angle of friction 'φ' are usually determined in the laboratory from the Direct Shear Test. For the present study the test was conducted on representative samples in Geotechnical Laboratory of Transport Construction Design Share Company (TCDSCo).

2.8.2 Unit weight

Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight, γ, is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. The test was conducted on representative samples in Geotechnical Laboratory of Transport Construction Design Share Company (TCDSCo).

2.9 Slope Stabilization methods

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

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

2.9.1 Soil slope stabilization methods

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

2.9.1.1 Retaining structures

Retaining structures are structures usually provided at the toe of a slope to stabilize it from slope failure, overturn or collapse. Retaining walls, sheet pile wall, sheeting in excavations, basement walls etc. are example of retaining structures. A retaining structure helps in maintaining the ground surface at different elevations on either side of it. Without such a structure, the soil at higher elevation would tend to move down till it acquires its natural stable configuration (Arora, 1997).

2.9.1.2 Gabions

Gabions are wire mesh, boxlike containers filled with cobble-sized rock. A gabion can also be constructed from stacked gabions. Gabion walls usually are inexpensive and are simple and quick to construct. Due to their flexibility, they can withstand foundation movement, and they do not require elaborate foundation preparation. Because of their coarse fill, they are very permeable and thus provide excellent drainage.

Gabion walls work because the friction between the individual gabion rows is very high, as is the friction between the basal row and the soil underneath. When failure occurs, it is almost always in the foundation soil itself. Gabion walls built on clay soils require counterforts, which can be constructed as gabion headers extending from the front of the wall to beyond the slip circle. The counterforts serve as both structural components and as drains (Hutchinson, 1977).

2.9.1.3 Drainage Techniques

Ground water probably is the most important single contributor to slope instability initiation. Not surprisingly, therefore, adequate drainage of water is the most important element of a slope stabilization scheme, for both existing and potential landslides. Drainage is effective because it increases the stability of the soil and reduces the weight of the sliding mass. Drainage can be either surface or subsurface. Surface drainage measures require minimal design and costs and have substantial stability benefits. This is recommended on any potential or existing slide. The two objectives of surface drainage are to prevent erosion of the face, reducing the potential for surface slumping, and to prevent infiltration of water into the soil, thereby reducing ground-water pressures. Subsurface drainage also effective but can be relatively expensive. It is therefore essential that ground water be identified as a cause of the slide before sub-surface methods are used (Hutchinson, 1977).

2.9.1.4 Stabilization Using Vegetation

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

2.9.2 Rock slope stabilization methods

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

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

2.9.2.1 Slope geometry alteration (Scaling)

Scaling is the process of removing loose or potentially unstable material that might dislodge or affect the trajectory of falling rock by creating a launching point for materials falling from above. It is accomplished by hand or mechanical scaling, or by small blasting operations called trim blasting. Scaling is effective on natural and newly excavated slopes, and is done as periodic maintenance for any slopes that pose a potential rock fall hazard to roadways.

As a stabilization or mitigation measure, scaling is typically effective for a period of two to ten years, depending on site conditions, so it is not considered a permanent mitigation measure. However, it is relatively inexpensive and serves as an effective short-term strategy. Because it enhances site safety, it is routinely included with other mitigation efforts such as; new rock excavation, rock reinforcement, or draped mesh.

Because of the obvious danger from falling debris, complete road closures are generally employed during scaling operations. In some cases, temporary measures such as; draped netting suspended from a crane can be employed while traffic is flowing, but such cases are rare. Temporary barriers (earthen berms) are often used to protect the roadway surface, bodies of water, buildings, or other critical features from rock fall (Selby, 1993).

2.9.2.2 Reinforcement systems

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

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

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

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

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

2.8.2.3 Shotcrete

Shotcrete is a wet-or dry-mix mortar with a fine aggregate (up to 23 mm, or 7/8 in) that is sprayed directly onto a slope using compressed air. Several applications may be needed to build the shotcrete up to the required thickness. Unreinforced shotcrete gives little structural support or protection against weathering, but can be used to prevent differential erosion between units, slope raveling, and loosening of blocks.
Shotcrete can also be applied around the exposed ends of rock bolts to help prevent weathering around the bearing plates and limit slope degradation. In most instances, structural shotcrete is applied to rock slopes to protect a surface which, left untreated, would erode (such as; a fault zone or clay seam), or to provide structural support for otherwise sound rock that is either undermined by erosion or is unstable due to unfavorable orientations or degree of fracturing.

This type of shotcrete application can be part of the original construction or part of the remediation of an existing unstable rock slope. Structural shotcrete can also be used to form part of a retaining system supporting the rock slope (Abramson et al., 2002).

2.9.2.4 Drainage systems

Slope stability can also be improved through the installation of drainage systems, which most often consist of horizontal weep drains. Water in a rock slope often contributes to slope instability, as excessive pore pressure acts on the rock mass and lowers the shear strength along any discontinuities. Water also contributes to rock degradation and fracture expansion and during the process of freeze-thaw weathering.

Normally, drainage systems are used in weak, highly fractured, or layered rock where instabilities could occur along a potential sliding surface. Drainage is generally used to mitigate larger rockslides and failures. In most cases, the drains are installed as uncased holes in massive rock units, drilled with a track rig or portable drill. In weak or highly fractured rock, the drain may be cased with a slotted polyvinylchloride (PVC) pipe to maintain the drain opening. Drains are installed at the base of the slope, and require periodic maintenance to prevent clogging (Hutchinson, 1977).

Usually, they are used in conjunction with other stabilization measures. Horizontal drains can be installed in a rock slope to reduce pore pressure and improve stability, and are cost-effective, aesthetically pleasing, and relatively low-maintenance option for most slopes with excessive flowing water. They are most effective for large-scale slope instability, where the potential sliding planes are deeply seated within the rock mass (Hutchinson, 1977).

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CHAPTER THREE METHODOLOGY

3.1 Introduction

The rapid development of computer software and hardware technology has helped to solve the complexity of engineering geological problems and analysis. There are various software packages commercially available for slope stability problem analysis such as; Geo-Studio (Slope/W), Slide, Plaxis and others. However, software analysis needs understanding, a sound knowledge of soil mechanics, rock mechanic and geology (Geo-Studio, 2007). Further, the correct engineering geological input data is also required to ensure the accuracy of the actual analyzed results by the software.

The limit equilibrium analyses have been commonly involved in geotechnical engineering for slope stability analysis. Limit equilibrium is widely used and accepted methods for slope stability analysis by using Slope/W and Slide software. However, the analysis and interpretation is a difficult task. Therefore, efforts should be made to collect the field data and the failure observations in order to understand the failure mechanism, which determines the slope stability method applicable for analysis.

Natural or cut soil and rock slopes are non-isotropic and have heterogeneous properties. The boundary conditions for the soil and rock model are very important in analysis, which also determines the accuracy of the actual model analysis. The correct material properties and boundary conditions of the particular soil and rock model are part of the solution results (Geo-Studio, 2007). The Slope/W and Slide software provides graphical output of results. These graphical outputs generally depicts failure plane/ slip surfaces and slice forces which ultimately gives the clear idea on stability condition.

The present study deals with slope stability analysis by limit equilibrium (LE) method where the results obtained from Slope/W and Slide software were compared for better interpretation. The LE based methods were compared based on the factor of safety (FOS) results obtained for various anticipated conditions (static and dynamic) through simplified slope geometry and input parameters. In general, the factor of safety (FOS) results obtained by Bishop simplified Method (BSM), and Morgenstern-Price methods (M-PM) were compared for various anticipated conditions. In these computations and predictions it was observed that critical slip surface is greatly influenced by the search technique.

3.2 Methodology adopted for the present study

Through a systematic literature review during the present study a conceptual framework on slope stability studies was developed. With this background knowledge a general methodology for the present study was evolved. Before field work different works have been done to have detailed information about the area and to be well prepared for the field work. Since there are several studies carried out in past by many researchers in the area, therefore as a part of literature review exhaustive review of previous studies was also undertaken. Thus, to meet out the objectives of the present study following systematic methodology was evolved and followed;

3.2.1 Literature review

In order to assess what previous research works on landslide were carried out in Ethiopia, in general and works in study area, in particular, a thorough systematic literature review was carried out. This literature review provided a detailed knowledge on the subject matter and also helped to understand various aspects related to landslides in the present study area.

In Literature Review (Chapter 3) a detailed description about the geology, geomorphology, structures and the engineering properties of geotechnical materials present in the study area has already been presented. Further, during the literature review, it was also attempted to understand the subject matter and to have a detailed knowledge about the landslides. A clear understanding on types of slope failure, types of slopes, mechanism of failure and triggering factors for slope instability was acquired. Besides, methods of slope stability analysis and mitigation measures were also thoroughly reviewed and grasped.

3.2.2 Collection of secondary data

Collection of secondary data is very important particularly to know about general conditions of the study area. Even though there was scarcity of secondary data such as; borehole data, ground water level and laboratory tests data, still attempts were made to collect existing data/information which were relevant for the present study. Such secondary data was collected from published and unpublished sources.

The secondary data, thus collected for the present study includes; (i) Geological map, (ii) Topographic map and (iii) Metrological data.

3.2.3 Reconnaissance survey

In order to gather general information about the study area a reconnaissance field survey was conducted during February15-19/2013. During reconnaissance survey information on landslides and related slope instability problems in the study area was mainly obtained through visual observations and by interviewing the local respondents.

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3.2.4 Detailed field work (Field observations and data collection)

The detailed field work was conducted during March 18-23/2013. During the detailed study, emphasis was made to identify and locate the most critical slope sections in the study area. For this thorough observations were made to know the actual or potential manifestations of slope instability in the study area. After the identification of critical slope sections further field observations relevant to slope stability studies were performed. It was believed that this stage was the best opportunity to collect the exact data needed for further analysis of the present research work. The field observation in general helped in understanding the site condition and provided a clear knowledge about the geology and geomorphology of the study area in general and specific to critical slope sections, in particular. Further, all necessary field observations/ sampling were made relevant to the study. Besides, all necessary primary data pertaining to various parameters were collected, this eventually formed the basis for analysis, as per the scope of the present study.

3.2.4.1 Field observations

During the field work, efforts were made to identify the different instability manifestation features present on the slope and to identify and collect data for the probable causative factors for the slope instability. Besides, information on past slope instability activities was gathered by interviewing the local respondents. In addition to these, it was attempted to assess the damages caused by the slope instability on infrastructure.

In general, the damages caused during and after the slope instability activities in the study area includes:

- ✓ Cracks on some houses and school building
- ✓ Displacement of the retaining wall and deformation of gabion structures
- ✓ Displacement of trees from high slopes up to the vicinity of road.

✓ Collapsed colluvial material along the road.

Plate 3.1 Cracking on roof and floor of school Building

Plate 3.2 Movement of colluvial material from steep slope

Plate 3.3 Displaced retaining wall near to school Building

Plate 3.4 Sliding of Silty clay soil within constructed structure

3.2.4.2 Data collection

During the field work all relevant information and data pertaining to the stability condition of the slopes were collected. Information was collected through visual observations pertaining to aspects related to slope stability and also by informal interviews of the local respondent.

Plate 3.5 Removed toe support for Road construction

During the field work following data was collected:

- ✓ Identification of the type and the distribution of various geotechnical materials forming the slope and the field manifestations of slope scarps and movement of material.
- ✓ Collection of soil sample to conduct laboratory test and to know shear strength parameters of various soil types forming the critical slope sections.
- ✓ Scarp faces on slope were observed and recorded.
- ✓ Damages that occurred on the houses because of the landslide process were also recorded.
- ✓ Indications for potential future landslide activity was also observed and recorded.
- ✓ Data about the slope instability process by informal interviewing of the local respondent were also carried out.
- ✓ GPS was used to locate/ mark the various instability features on the map.

3.2.5 Selection of critical slope sections

In general, the present study area is highly affected by slope instability problems with a circular/rotational mode of failure involving different slope forming materials. Previous studies have shown that the slopes along roads were affected several times by slope instability (landslide) problems (Lulseged Ayalew, 1999; Tenalem Ayenew and Barebieri, 2005).

Lulseged Ayalew (1999) has recorded and reported previous landslides and indications for potential future slides such as; cracks on houses and school building, scarps on steep slopes, displacement of retaining wall constructed near to Mehnber Tsaye School and huge colluvial material movement along the road side. Specifically, the study area has a high potential for slope instability; the slope mass is mainly composed of silty clay and colluvial material, which is the result of weathered basalt. The slopes formed of such material in the present study area are generally, characterized by circular/ rotational mode of failure.

Further, based on field manifestations of slope movement two critical slope sections were selected for detailed stability analysis. Both the selected critical slope sections fall on soil slopes and generally characterized by circular/ rotational mode of failures. Moreover, detailed selection criteria of critical slope section and observed field manifestations is presented in Chapter four.

3.2.6 Data Processing

After the field work, different tasks were performed following various approaches. Data were organized in such a way, so that it can be accessed and used easily when needed. The tasks performed include: primary and secondary data, as mentioned above, was analyzed and processed to get the required information and input for further detailed stability analysis.

One of the important input parameter for any slope stability analysis is the shear strength parameters; cohesion and angle of shearing resistance. Since the present slope is mainly composed of silty clay associated with colluvial material. Thus, attempts were made to know the shear strength parameters by conducting sensitivity analysis and Laboratory test. Detailed slope stability analysis was performed using a deterministic approach of slope stability analysis. For the present study '*Slope/W*' and '*Slide*', commercially available software, were used for the slope stability analysis.

3.3 Computer program (software's) used for slope stability analysis

Nowadays there are several computer based geotechnical software's used in the slope stability analysis. Some of the commonly used software's for the Limit Equilibrium analysis are; *Slope/W*, which is developed by Geo-Slope International Canada, and *'Slide'* software, developed by Rocscience Inc Toronto Canada. Both the software's provides almost similar type of results with minor variations. Prasad, (2006) used these software's for limit equilibrium slope stability analysis and he finally concluded that the two software's computed identical safety factor within ± 1% variations only.

3.3.1 General description about Slope/W software

The application is crated based on limit equilibrium method and it include different types of methods like, Bishop Simplified (BS), Janbu Simplified (JS), Spencer (SP), and Global Limit Equilibrium/Morgenstern-Price (GLE/M-P) methods. For present FOS computations only Bishop Simplified (BS) and Morgenstern-Price (M-P) were used.

The results of stability analysis from the Slope/W can be obtained as both visuals and numbers. The visually interpreted results make it possible to easily understand the results in numbers. The very important advantage of the Slope/W analysis is that it allows handling of all possible slides in a same model with the corresponding factor of safety.

3.3.1.1 Procedure of slope stability Analysis by Slope/W

The general procedure followed for slope stability analysis by using GeoStudio (Slope/W) software is given as under;

- ✓ Geometrical modeling (two-dimensional representation) of selected slope sections is delineated through site observations/ estimations and topography review. For the present study elaboration on how the geometry of slope was deduced and how the soil layer thickness was estimated is discussed later in detail in chapter four.
- ✓ Factor of safety was computed by Slope/W using limit equilibrium analysis by different methods, such as: Bishop simplified Method (BSM), Janbu simplified Method (JSM) and Morgenstern-Price method (M-PM).
- ✓ Minimum factor of safety was determined by considering several slip surfaces through techniques provided in the software. Such techniques are defined in terms of; grid and radius and exit and entry methods.

- ✓ Auto search provided in the software was used to find the minimum factor of safety. This required analyzing the problem with other modern methods such as; Morgenstern-Price method. This is to verify the minimum factor of safety, as mentioned above.
- ✓ Later, for relative interpretation of FOS under various anticipated conditions, for each critical slope section, was prepared in tabular format.
- ✓ Further, critical cases were identified based on the minimum factor of safety, specified as less than the acceptable limit.
- ✓ Later, critical cases were re-analyzed after adopting suggestive correction measures in order to increase the factor of safety to the acceptable limit.

3.3.1.2 Slip Surface Shapes

The process of determining the position of the critical slip surface with the lowest factor of safety is one of the important aspects in a stability analysis. It is known that, the critical slip surface finding process will involve a trial and error procedure. In this process a possible slip surface is created and the associated factor of safety is computed. These processes are repeated for many possible slip surfaces until the trial slip surface with the lowest factor of safety is deemed the governing or critical slip surface (Geo-Studio, 2007).

There are many different ways for defining the shape and positions of trial slip surfaces but in the present study the focus was on all procedures that are generally used in grid and radius for circular slip**s** in SLOPE/W, and the applicability of the methods to various situations.

The soil strata are one of the aspects that may influence the critical mode of potential failure. The strata therefore must be considered in the selected shape of the trial slip surfaces. In general, not all potential modes of failure can be investigated in one analysis. In such cases, the positions of the trial slip surfaces needs to be specified and controlled to address specific issues (Geostudio,2007).

A general procedure for defining trial slips may result in some unrealistic trial slip surfaces which is the trial slip surface shape and cannot exist in reality. Hence, it is not possible to compute a safety factor value for such unrealistic situations, due to lack of convergence. The software cannot necessarily make this judgment.

Another important aspect that comes into consideration in determination of the critical slip surface position is the selection of soil strength parameters. Different soil strength parameters can give different computed positions of the critical slip surface.

3.3.1.3 Grid and radius for circular slips

A grid and radius for circular trial slip surfaces at first are inherent in limit equilibrium formulations, however the techniques of specifying circular slip surfaces has become entrenched in these types of analyses lately. The trial slip surface is an arc of circle that cuts through the slope. These circles can be defined by specifying the x-y coordinates of the circle centre and the radius. A wide variation of trial circular slip surfaces can be specified by defining a grid of circle centers and a range of defined radius. In SLOPE/W, this procedure is known as Grid and Radius method.

Each grid point is the circle centre for the trial slips. In SLOPE/W, the grid is defined by three points; they are upper left (18), lower left (16) and lower right (12) (Geo-Studio, 2007). The circle radius of the slope is defined by the radius or tangent lines. The lines are specified by the four corners that are 19 (upper left), 21 (lower left), 22 (lower right) and 20 (upper right).

During positioning the four points, it is needed to start the point at the upper left and proceed in a counter-clockwise direction around the box. The number of increments between the upper and lower corners can be specified after the four points are placed.

SLOPE/W will start forming the trial slip surfaces by forms equation for the first radius line. Next, it will find the perpendicular distance between the radius line and a grid centre. Hence, the perpendicular distance becomes the radius of the trial slip surface. These specified radius lines are actually more correctly tangent lines; that is, they are lines tangent to the trial circles

3.3.1.4 Geometry

Slope/w uses the concept of region to define geometry. Basically this simply means drawing a line around a soil or stratigraphic layer to form a closed polygon, similar to what we would do sketching a problem to schematically explain a particular situation to someone else. In this sense it is a natural and intuitive approach. The concept of regions that are used in SLOPE/W is to define the geometry. Basically to create these regions it can be simply done by drawing a line around a soil unit or stratagraphic layer to form a closed polygon (Geo-Studio, 2007).

In SLOPE/W, the regions are in essence n-sided polygons. The Polygons with three sides are known as triangles while the polygons with four sides are known as quadrilaterals (Fig. 3.1).

In SLOPE/W all the regions must be connected to form a continuum. This can be done with the use of Points. The points are represented by the small black squares at the region corners as shown in Fig. 3.2. The regions are connected by sharing the points. As shown in Fig. 3.2, Points 6 and 7 are common to the two regions and the two regions consequently behave as a continuum.

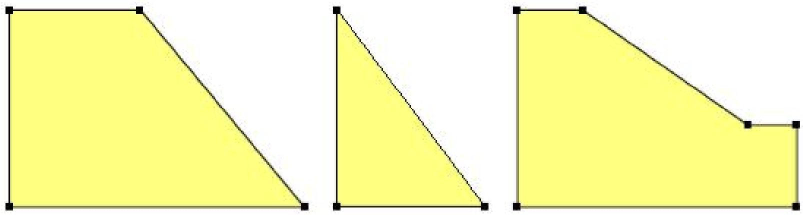

Fig. 3.1 Representative basic region (Geo-Studio, 2007)

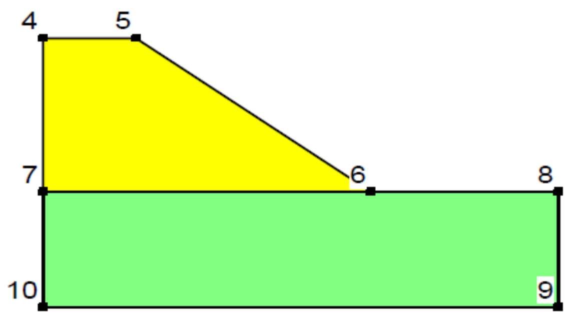

Fig. 3.2 Region point (Geo-Studio, 2007)

One of the advantages of the regions is that the geometry can be easily modified by moving the points. In this case, the regions do not necessarily need to be redrawn, it is only by moving the points in order to make modification to the geometry. By the way, regions do have a couple of restrictions. They are:

- ✓ A region can have only one material (soil) type. Even though the same soil type can be assigned to as many as different regions, but each region can only be assigned by soil type.
- ✓ The regions drawn cannot be overlapped.

3.3.1.5 Slice discretization

The SLOPE/W software uses a variable slice width approach in the sliding mass discretization. In this means, SLOPE/W will discretize the soil mass with slices of varying width to ensure that only one soil type exists at the bottom of each slice. It is also used to prevent a ground surface breaking occurred along the top of the slice and to prevent the phreatic line from cutting through the base of a slice. The objective is to have all the geometric and property changes which have occurred at the slice edges (Geo-Studio, 2007).

Basically SLOPE/W divides the potential sliding mass into a section. In this situation, the first section will start on the left where the slip surface enters the ground surface while other sections occur where;

- ✓ the slip surface crosses the piezometric line,
- ✓ the slip surface crosses a stratigraphic boundary,
- ✓ wherever there is a region point, and
- ✓ where the piezometric line crosses a soil boundary

SLOPE/W then will finds the horizontal distance from slip surface entrance to exit and divides this distance by 30 number of slices or the number of slices specified by the user, with that all the slices has an average width. The result of slice discretization process is shown in Fig. 3.3 with the specified number of slices (15). Even though the number of slices has been specified by the user but the actual number of slices in the final discretization may be slightly higher or lower than the specified value.

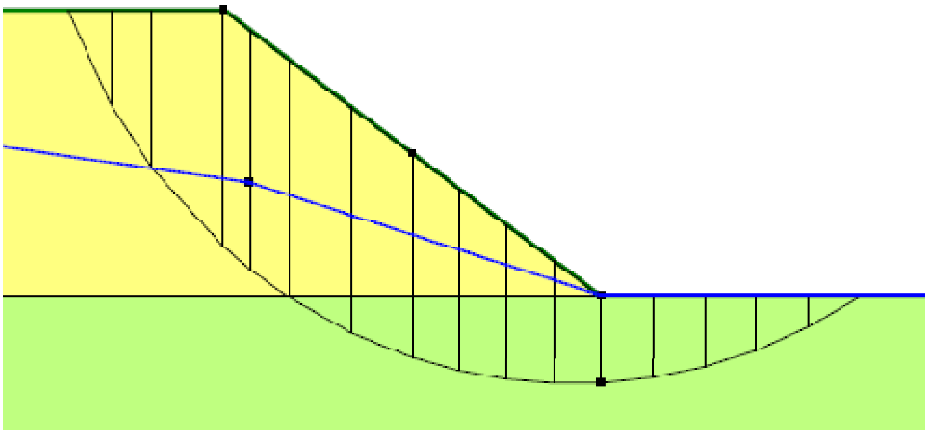

Fig. 3.3 Descretization when the specified number is 15 slices (Geo-Studio, 2007)

The SLOPE/W approach with variable slice width will be resulting into the factor of safety relatively insensitive with the number of slices. The specified number of slices which is greater than the default number of 30 will have minor impact on the factor of safety. While specified number of slices lower than the default number is not recommended unless it is used to investigate a specific issue like comparing the results obtained with manual calculations. The discretization with the number of specified slices equal to 30 is shown in Fig. 3.4.

3.3.1.6 Material Strength

Mohr-Coulomb criteria

In SLOPE/W, there are different methods by which the material strength to be used in a stability analysis can be obtained. The one that will be focused on is Mohr-Coulomb method.

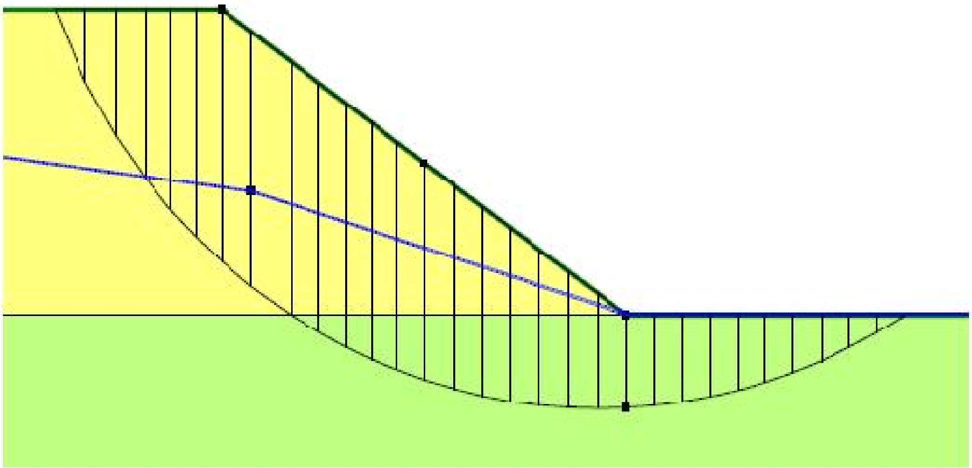

Fig.3.4 Discretization when specified number is 30 (Geo-Studio, 2007)

Mohr-Coulomb is one of the most common methods for obtaining the shear strength of geotechnical materials.

This can be done by Coulomb's equation which is:

$\tau = c + \sigma \tan\Phi$ …………..*eq. 3.1*

An above equation represents a straight line on a shear strength-normal stress plot as shown in Fig. 3.5. The intercept on the shear strength axis is the cohesion (c) and the slope of the line is the angle of internal friction (Φ) (Geo-Studio, 2007).

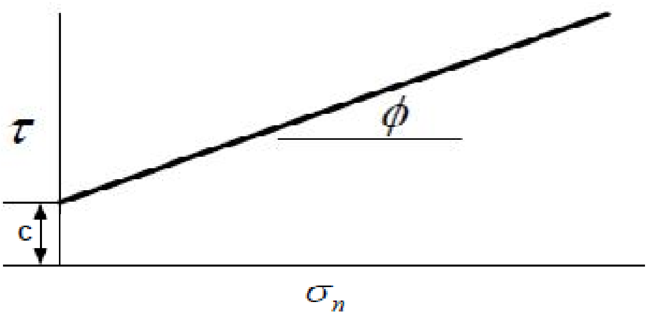

Fig. 3.5 Graphical representation of Coulomb shear

The material that has been tested under undrained conditions will give a value for internal friction angle (*Φ*) equal to zero. Basically, triaxial test will give the strength parameters 'c' and '*Φ'* in the forms of total strength parameters or effective strength parameters which came from undrained or drained test, respectively.

Even that, in SLOPE/W there is no distinction between these two sets of parameters. The appropriate sets of parameters for a particular analysis are project-specific, which is needed to be decided by user. Based on slope stability analysis point of view, effective strength parameters from drained test will give the most realistic results, especially with respect to the position of the critical slip surface. This is different when undrained strengths parameters are used in a slope stability analysis. Even though these strength parameters will give the lowest factor of safety but the position of the slip surface given is not necessarily close to the position of the actual slip surface if the slope has to fail.

3.3.2 Description about Slide Software

Slide is a 2D slope stability program which can be used for the evaluation of the stability of circular or non-circular failure surfaces in soil or rock slopes. Slide is very simple to use, and yet complex models can be created and analyzed quickly and easily. External loading, groundwater and support can all be modeled in a variety of ways. In the present study Slide software was used for the reasons that: (i) it is freely available using on line hard lock, (ii) it is friendly software for user, and it has similar features and functions with that of Slope/W software. Modeling in Slide software for the slope analysis is possible for external loading, groundwater and forces from pseudo-static earthquakes. This application is created based on limit equilibrium method and it also includes different types of methods like Ordinary, Bishop, Janbu and Morgenstern – Price methods.

3.3.2.1 General Procedure of slope stability Analysis by Slide

The general procedure followed for slope stability analysis using Slide software is given as under;

- ✓ Circular surface grid search
- ✓ Single circular surface defined by a centre and radius or by three points on the surface
- ✓ Slope Search method (allows users to define slope parts through which circular slip surfaces must pass)
- ✓ Auto-refine search (an iterative technique for locating the minimum slip circle that uses the results of a previous iteration to narrow the search area in the next step)

- ✓ Analysis of the slope for dynamic condition using Slide software by introducing horizontal Peak Ground Acceleration (Z factor or PGA factor) to introduce seismic forces.

3.3.2.2 Location of Critical Surfaces

One of the most important facets of slope stability analysis by Slide software is finding the slip surface, which has the lowest factor of safety. The developers of Slide implemented proven search techniques for locating both circular and noncircular slip surfaces. They help engineers to ensure that they have indeed determined critical surfaces.

3.3.2.3 Strength Models

The shear strengths of the materials that form a slope have significant impact on stability and are required for all limit equilibrium methods. The following strength models are commonly used in Slide: Mohr-Coulomb, Hoek-Brown, and Generalized Hoek-Brown. For current study the Mohr-Coulomb strength model was entirely used.

In the majority of practical slope problems, the greatest uncertainty is associated with the evaluation of shear strength parameters. As a result engineering geologist often has to assess the influence of various assumed strength models and parameters on stability (Rocscience Inc, 1989 - 2003). To facilitate this process, significant effort was made in creating simple, quick-to-use means for entering strength parameters into Slide.

3.3.2.4 Model Creation Tools in Slide

Unlike most slope stability programs, Slide does not impose severe restrictions on the types of geometries that can be modelled, nor on the manner in which boundaries have to be defined. It can readily accommodate complex slope and material boundary geometries. The geometry of models is entered into Slide using CAD-based graphical data technology. Such an approach allows for the interactive building and editing of models. Slide can also import model geometries from AutoCAD™ DXF files (Rocscience Inc, 1989 - 2003).

3.4 Preparation of input parameter for slope stability analysis

One of the important inputs for any slope stability analysis is the shear strength parameters; Cohesion and angle of shearing resistance. The critical slope sections of the study area is mainly composed of colluvial material in critical slope section one and silty clay material in critical slope section two. From each soil section samples were collected for laboratory test. From coluvium material finer fraction of disturbed soil sample was collected for laboratory testing and soil test results were utilized for sensitivity analysis. In case of critical slope section two disturbed soil samples were collected for laboratory test. Later, attempts were made to make estimations on shear strength parameters by conducting sensitivity analysis. Sensitivity analysis is an interactive process adopted for the purpose of realistically simulate the slope instability conditions and to determine the relative influence of different parameters on the value of Factor of Safety. Sensitivity analysis indicates that which input parameters may be critical to the slope stability condition and which input parameters are relatively less important. Computations were performed using the Slide program and were based on the shear strength parameters on the determination of the Factor of Safety. These computations were made to identify the critical situations for the slope stability. Soil samples collection, laboratory test results and interpretations of results is discussed in the following paragraphs;

3.4.1 Preparation of input parameter for slope section one (SS1)

Determination of Shear Strength parameter of slope material (Colluviall material) (SS1)

Colluviall material is characterized by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders located at the base of cliffs and steep slopes, with increasing thickness towards the base of the slopes. Engineering properties of colluvial material are extremely difficult to determine in the lab or in in-situ conditions. A useful method for determining shears strength properties in colluviall materials is to analyze an existing slope failure (WSDOT Geotechnical Design Manual, 2010).

The shear strength parameter values for the present slope stability analysis (SS1) were adopted by performing a sensitivity analysis. The sensitivity analysis provided a representative value for the shear strength parameters which were later utilized for slope stability analysis. The sensitivity analysis was carried out for cohesion and angle of internal friction by keeping one of these values constant at a time and by varying the other value within its estimated permissible limits. The results of the sensitivity analysis indicated that material properties play a significant role in controlling slope processes and that the slope geometry and climatic conditions should not be considered as the only parameter to define instability. By evaluating a sensitivity analysis of the factor of safety to variables in slope stability model, one can understand easily the importance of accurate parameter estimation as well as their possible significance. For the present stability analysis process the results from the sensitivity analysis and the test results from the laboratory on finer soil fraction of coluvium material were utilized.

The sensitivity analysis results for shear strength parameters are presented in Table 3.1, 3.2 and Fig. 3.6, 3.7.

Table 3.1 Sensitivity analysis for Cohesion by keeping friction angle and unit weight constant (SS1)

Unit weight(KN/m^3)	Friction angle (^{0})	Cohesion (KN/m^3)	FOS
16.464	31	12	0.95
		13.2	0.97
		14.4	0.99
		15.6	1.002
		16.7	1.02

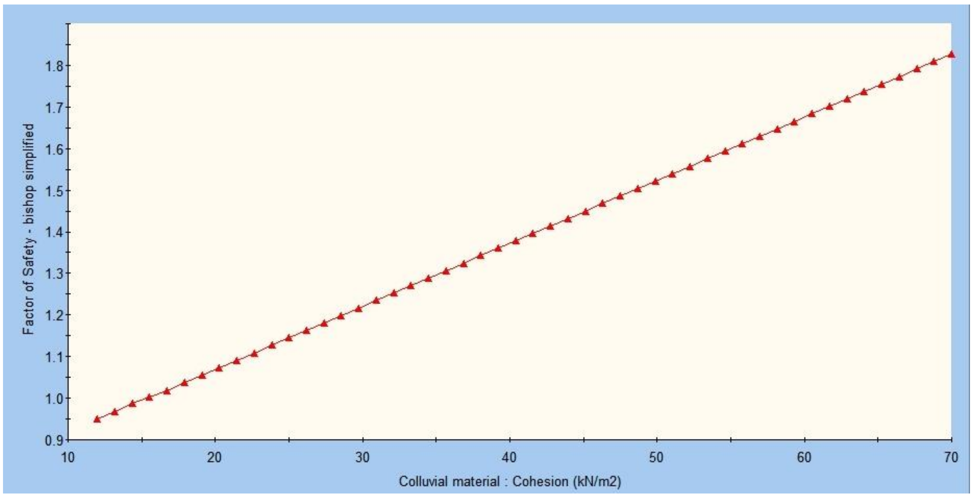

Fig 3.6 Sensitivity of FOS for Cohesion value (SS1)

Table 3.2 Sensitivity analysis for friction angle by keeping Cohesion and unit weight constant (SS1)

Unit weight(KN/m^3)	Cohesion (KN/m^3)	Friction angle (^{0})	FOS
16.464	15.6	29.4	0.96
		30.1	0.978
		30.7	0.995
		31.3	1.001
		31.9	1.03

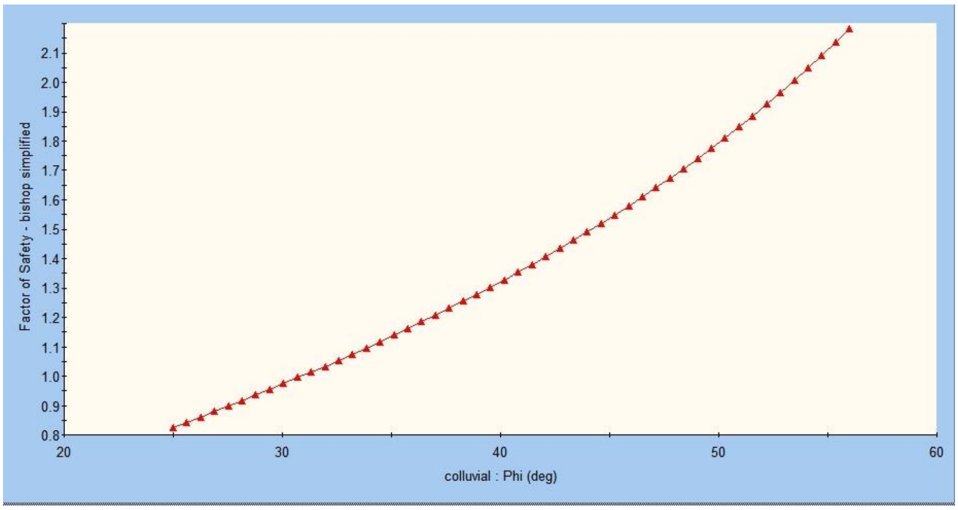

Fig 3.7 Sensitivity of FOS for friction angle (SS1)

In general, a sensitivity analysis indicates that which input parameter may be critical for the assessment of slope stability, and which input parameters will be less important. A perusal of graphs in Fig. 3.6 and Fig. 3.7 clearly shows a linear relationship between FOS and the shear strength parameters. In general, as the shear strength parameters values increases FOS also increases. However, within the permissible limits of each parameter the effect of FOS is same. In terms of relative importance both parameters are contributing equally to FOS.

Shear strength parameters of colluvial material are extremely difficult to determine in the laboratory or in in-situ condition due to its variable composition in terms of particle size. However, for present study attempts were made to estimate the shear strength parameters through direct shear test which was conducted on the finer fraction of the soil in Geotechnical Laboratory of Transport Construction Design Share Company (TCDSCo).

The laboratory disturbed soil sample results (finer soil portion) are presented in Table 3.3;

Table 3.3 Material Properties and Laboratory test result of finer soil (SS1)

Section	Material	Internal friction angle(^{0})	Cohesion (KPa)	Bulk Unit weight(gm/cm^3	Dry unit weight (gm/cm^3)
SS1	Colluvial material(silty clay dominated)	31	58	1.98	1.68

Determination of the unit weight

The unit weight used in the present analysis was obtained by bulk and dry density values obtained from laboratory tests conducted on finer samples. From the bulk and dry density result, the unit weight was determined by using following formula:

*Unit weight (Bulk) = Bulk density*g=1.98* gm/cm^3*9.8m/s^2=19.404KN/m^3

*Unit weight (dry) = Dry density*g=1.68* gm/cm^3*9.8m/s^2=16.464KN/m^3

Where; g acceleration due to gravity is equal to 9.8m/s^2.

The values as mentioned above were utilized for the slope stability analysis.

3.4.2 Slope Geometry of critical slope section (SS1)

The general slope geometry of SSI slope section is presented in Fig. 3.8. The slope section has a length of about 125m and a height of about 60m. The general failure direction is along the road at angle of 43^0. The slope geometry of SS1 critical section was deduced through field manifestation, measuring lateral and vertical distance and measuring slope by Burton compass.

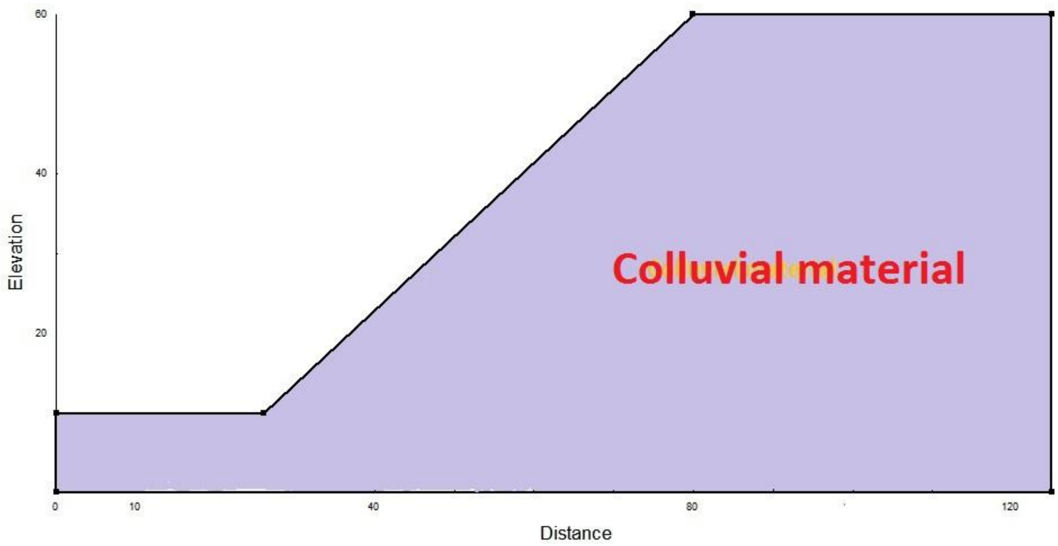

Fig 3.8 Slope geometry of critical slope section SS1

For the slope stability analysis, the angle of internal friction was adopted as 31.3^0 and that of cohesion (C) as 15.6 KPa for the dry condition. For the worst anticipated and present condition, adjustment was made on the cohesion, since cohesion of the colluvial soil materials could be reduced by the impact of the water, i.e. when it become partial and fully saturated. Thus, the value of cohesion adopted for present condition was 14.4 KPa and for worst anticipated condition was taken as 13.2 KPa.

Table 3.4 Shear strength parameters adopted for the stability analysis SS1

Conditions	Unit weight (KN/m^3)	Cohesion (KPa)	Internal friction angle(^{0})
Static Dry	16.464	15.6	31.3
Static Present	19.404	14.4	31.3
Static worst	19.404	13.2	31.3
Dynamic dry	16.464	15.6	31.3
Dynamic present	19.464	13.2	31.3
Dynamic worst	19.464	13.2	31.3

3.4.3 Preparation of input parameter for slope section two (SS2)

The shear strength parameters (Internal friction angle and Cohesion) and unit weight of the soil (disturbed) for this section were determined through direct shear test conducted in Geotechnical Laboratory of Transport Construction Design Share Company (TCDSCo).
The laboratory soil sample (disturbed) results that were used as basis for sensitivity analysis and as input parameter for slope stability analysis of SS2 section is presented in Table 3.5;

Table 3.5 Material Properties and Laboratory test result for critical slope section (SS2)

Section	Material	Internal friction angle(^{0})	Cohesion (KPa)	Bulk Unit weight(gm/cm^3)	Dry unit weight(gm/cm^3)
SS2	Silty clay	13	78	1.73	1.36

Determination of the unit weight

The unit weight used in the present analysis was obtained by bulk and dry density values obtained from laboratory tests conducted on representative samples. From the bulk density result, the unit weight was determined by using the following formula:

*Unit weight = Bulk density*g=1.73* gm/cm^3*9.8m/s^2=16.954KN/m^3
*Unit weight = Dry density*g=1.36* gm/cm^3*9.8m/s^2=13.328KN/m^3

Where; g acceleration due to gravity is equal to 9.8m/s^2.

The values as mentioned above were utilized for the slope stability analysis.

Further for SS2 critical slope section also sensitivity analysis for shear strength parameters was made following similar procedures as followed for critical slope section SS1. The sensitivity analysis results for shear strength parameters for SS2 slope section are presented in Table 3.6, 3.7 and Fig. 3.9, 3.10.

Table 3.6 Sensitivity analysis for Cohesion by keeping friction angle and unit weight constant (SS2)

Unit weight(KN/m^3)	Friction angle (^{0})	Cohesion (KN/m^3)	FOS
16.954	5.8	75	0.984
		76.6	0.999
		78.2	1.013
		79.8	1.03
		81.4	1.058

Table 3.7 Sensitivity analysis for friction angle by keeping Cohesion and unit weight constant (SS2)

Unit weight(KN/m^3)	Cohesion (KN/m^3)	Friction angle (^{0})	FOS
16.464	78	5	0.974
		5.3	0.986
		5.5	0.999
		5.8	1.013
		6.1	1.03

A perusal of graphs in Fig. 3.9 and Fig. 3.10 clearly shows a linear relationship between FOS and the shear strength parameters. In general, as the shear strength parameters values increases FOS also increases. However, within the permissible limits of each parameter the effect of FOS is same. In terms of relative importance both parameters are contributing equally to FOS. The trend in sensitivity results are almost similar to what were observed in the case of critical slope section SS1.

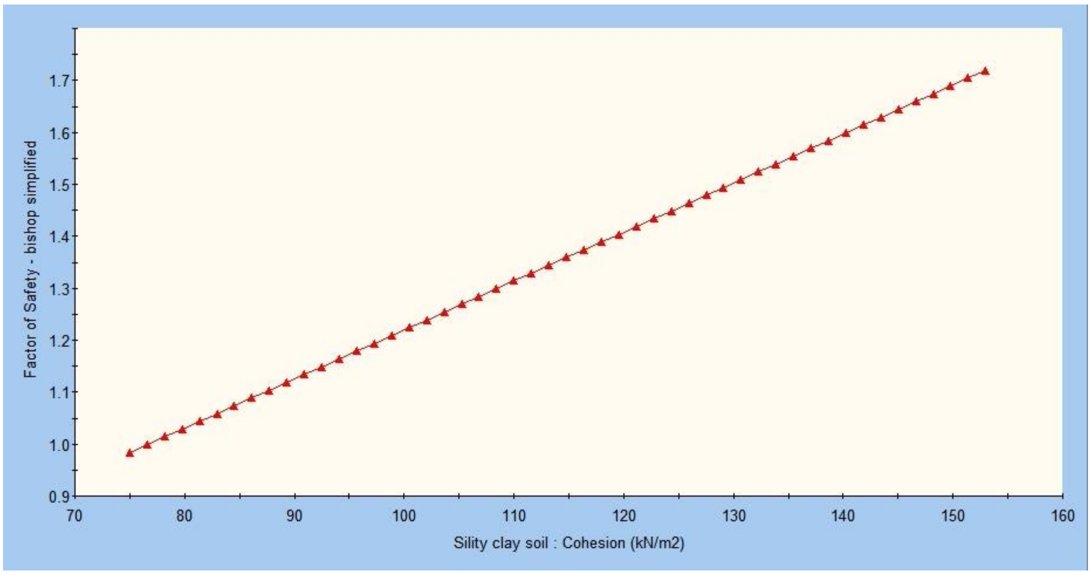

Fig 3.9 Sensitivity of FOS for Cohesion value (SS2)

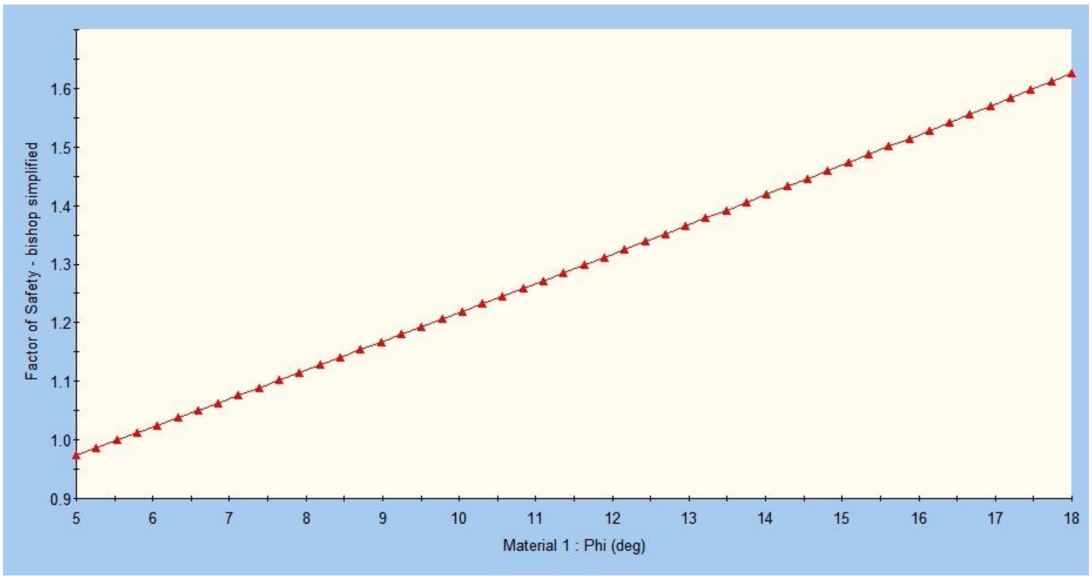

Fig 3.10 Sensitivity of FOS for friction angle (SS2)

3.4.4 Slope Geometry of critical slope section (SS2)

The general slope geometry of critical slope section is shown in the Fig. 3.11. It has a length of about 150m and a height of about 50m. The general failure direction is towards the road. The angle of the slope profile is about 29^0 in the South direction from Menber Tseaye School. The slope geometry of this critical section was deduced from field manifestations, measuring lateral and vertical distance and measuring slope by Burton compass.

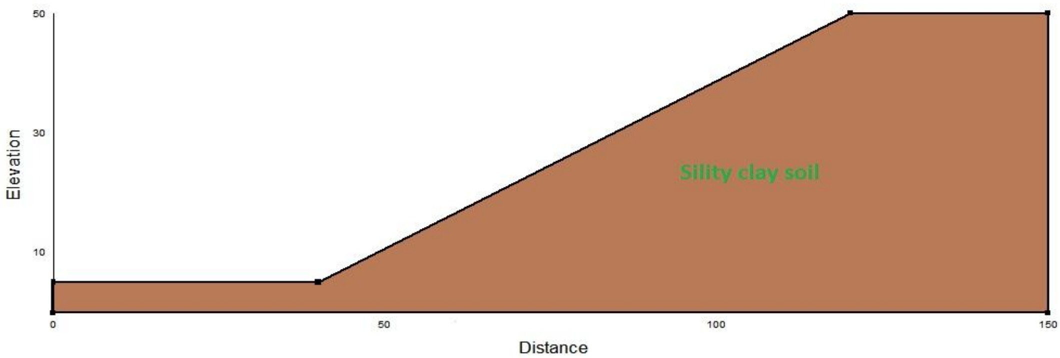

Fig 3.11 Slope geometry of critical slope section two (SS2)

Based on sensitivity analysis and Laboratory results for the slope stability analysis, the angle of internal friction was adopted as 5.8^0 and that of cohesion (C) was taken as 78.2 KPa for the dry condition. For the worst anticipated and present condition, adjustment was made on the cohesion, since cohesion of the silty clay soil materials may be reduced by the impact of the water, i.e. when it become partial and fully saturated.

Further, the value of cohesion adopted for present condition was taken as 76.6 KPa and for worst anticipated condition it was taken as 75 KPa.

Table 3.8 Shear strength parameters adopted for the stability analysis (SS2)

Conditions	Unit weight (KN/m^3)	Cohesion (KPa)	Internal friction angle(^{0})
Static Dry	13.328	78.2	5.8
Static Present	16.954	76.6	5.8
Static worst	16.954	75	5.8
Dynamic dry	13.328	78.2	5.8
Dynamic present	16.954	76.6	5.8
Dynamic worst	16.954	75	5.8

3.5 Conditions for which stability analysis was made

During the present study factor of safety was determined for existing and possible worst conditions defined as follows;

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

ii) Moderately Saturated/Present Condition: Moderately saturated condition is the condition which represents the current situation of the groundwater condition. Under this situation the water forces within the slope will be moderate and the slope material will be partially saturated. Such situation may occur during moderate rainfall. For this condition stability analysis was performed under both static and dynamic loading situation.

iii) Worst anticipated condition: The worst anticipated condition is the condition which represents the worst condition when the slope is fully saturated with water, i.e. when the groundwater level is almost at the surface. This situation may occur during very heavy sustained raining in the area. The groundwater will be fully recharged and the slope material will be fully saturated. Under such situation the water forces developed within the slope material will be destructive and will induce total instability condition in the slope. For this condition stability analysis is performed under both static and dynamic loading situation.

Finally, by comparing the computed factor of safety FOS from software’s an appropriate mitigate measure was evolved for the potentially unstable critical cut slopes.

A detailed description on stability analysis and possible remedial measures is discussed in following chapters.

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CHAPTER FOUR

4. SLOPE STABILITY ANALYSIS AND DISSCUSION

4.1 Introduction

Slope stability analyses is mainly performed to assess the safety factor of a particular slope in a given geologic and physical conditions. For a slope to be stable the resisting forces in the slope must be sufficiently greater than the forces causing the failure (Duncan and Wright, 2005).

Stability analysis can be used for the following;

- ✓ To assess the safety of a structure in terms of its stability.
- ✓ To locate the critical failure surface and to know its shape of failure.
- ✓ To understand and numerically evaluate the sensitivity of stability to its geologic parameters and climatic conditions.
- ✓ To assess the movement of the slope.
- ✓ To assess remedial measures and aid in their design.

To perform a slope stability analysis the geometry of the slope, external and internal loading, soil stratigraphy and strength parameters and variation of the ground water table all along the slope must be defined. The accuracy of the analysis of a particular slope depends on precise calculations of the slope geometry, the groundwater conditions and soil properties. It is also important that the analysis models the slope conditions precisely and that the method of analysis is reliable (Nash, 1987).

During the present study slope stability analysis for selected slope sections in the study area was conducted based on geometry of the slope, anticipated earthquake loading, possible ground water effects, soil stratigraphy and the shear strength of the slope forming materials. The required data for analysis was collected from field work and laboratory soil sample tests.

Further, stability analysis was performed by considering, slope geometry, geotechnical input parameter and anticipated existing and adverse site conditions likely to prevail on the identified critical slope sections. The stability analysis was made by following limit equilibrium (LE) method. In general, LE methods are important because of two reasons; firstly, the methods have proved to be reasonably reliable in assessing the stability of slopes and secondly, the methods require a limited amount of input, but can quickly perform an extensive trial-and-error search for the critical failure surface (Fredlund and Rahardjo, 1993).

4.2 Uncertainties in slope stability analysis

In slope stability analysis, there are uncertainties that come from different sources and conditions. According to Alonso (1976 as cited in Samuel Molla, 2011), uncertainties mainly accounts for; soil properties, environmental conditions, and theoretical models. These uncertainties are the most important source for a lack of confidence in deterministic analysis.

Morgenstern (1995) divided geotechnical uncertainties into three distinctive categories; (i) Parameter uncertainty, (ii) Model uncertainty and (iii) Human uncertainty.

Parameter uncertainty is the uncertainty in the inputs of analysis; model uncertainty is due to the limitation of the theories and models used in performance prediction; it is the gap between the theory adopted in prediction models and reality while human uncertainty is due to human errors and mistakes, this results when incorrect mechanism is modeled or failure occurs due to mistake in correctly determining the chosen model or it may be due to carelessness and ignorance, misleading information, poor construction, inappropriate contractual relationships and lack of communication between parties involved in the project. Model uncertainty is probably the major source of uncertainty in geotechnical engineering (Morgenstern, 1995).

4.3 Factors causing slope instability in the present study area

There are many factors that contribute to slope failure along Kombolcha to Desse road. These include;

- ✓ Extremely high relief and very steep slopes.
- ✓ Nature of geologic formations exposed along the road section (abundance of unconsolidated material, highly weathered and fractured rock masses).
- ✓ Presence of loose rock blocks of variable sizes on natural slopes as a result of weathering.
- ✓ Severe climatic conditions leading to rapid weathering.
- ✓ Poor drainage conditions
- ✓ Slope disturbance during road construction

4.4 Identification of critical slope sections in the study area

During the present study, through a reconnaissance survey most critical slope sections were identified in the study area. For this purpose the following field manifestations were observed to be an indicator of a critical slope sections;

(i) Presence of scarp face on steep slopes,
(ii) Removal of toe support for road construction or maintenance,
(iii) Presence of evidences of slope distress, such as; development of tension cracks, bulging of slope face and other such features,
(iv) Damages on the community houses and other nearby buildings along the main road
(v) Collapsing of tress from steep slope associated with colluvial material
iv) Displacement of retaining wall constructed near to the School building of Menber Tseaye.

Thus, based on the above field manifestations, two circular/rotational critical slope sections were identified along the road and its corridors. These critical slope sections contain mainly colluvial and silty clay type soil material.

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

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

A purely circular failure surface on a rotational failure is quite rare because frequently the shape of the failure surface is controlled by the presence of pre-existing discontinuities, such as; faults, joints, bedding, shear zones, etc. The influence of such discontinuities must be considered when a slope stability analysis of rotational failure is being conducted. The location of the critical failure surface is found by determining the lowest value of safety factor obtained from a large number of assumed failure surface positions (Kliche, 1999).

The locations of two critical slope sections, identified for further analysis during present study, are shown in Table 4.1 and Fig.4.1. The detailed slope stability analysis for these critical sections is presented later in this chapter.

Table 4.1 UTM locations of the critical slope sections

Location	Critical Slope Sections	
	SS1	**SS2**
Easting (mN)	0570029	0569952
Northing (mE)	1227270	1227939

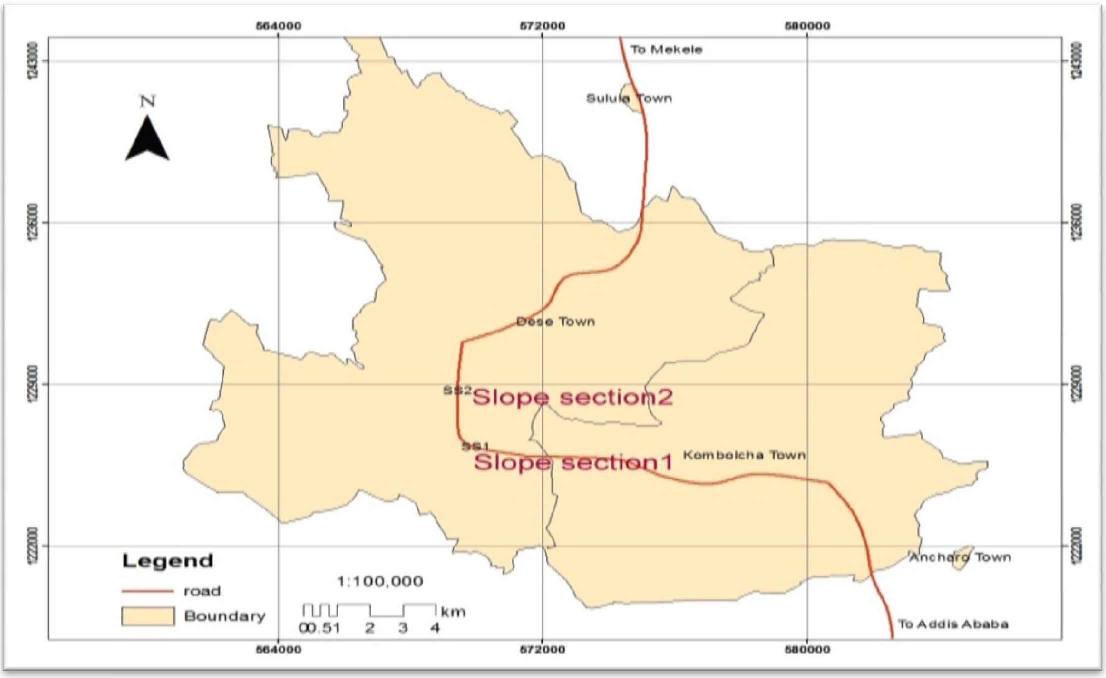


Fig.4.1 Critical Slope sections in the study area

4.4.1 Critical Slope Section one (SS1)

Critical slop section SS1, having circular mode of failure, is located at UTM coordinates 0570029 m E/ 1227270 m N along the road cut. This section mainly comprises of colluvial material which mainly contains basaltic fragments of varied shape and dimensions in a matrix of clayey silty soils. Along the road side the slope section is steep and the colluvial material generally collapse easily along the road side. The colluvial material and silty clay soil that has moved from the steep cliffs in the past has deposited along the road side and is susceptible to huge mass movement (Plate 4.1).

The colluvial materials are expected to be prone to failure as they are characterized by low friction resistance. Also, the topography of the area fulfills the conditions for circular modes of failure. Normally, what was observed from the present field work is that the failure surface shows scarp on the steep slope and these develop into an arc segment that serves as a failure plane. Any available water body passes through this scrap may further reduce the shear strength of the soil/colluvial material. Further, this water body serves as a lubricant for the sliding mass.

The movement of huge mass of colluvial material from steep slope to the side of the road

Plate 4.1 Critical slope section SS1

According to Duncan and Wright (20050, the reduction of the shear strength with intensive rainfall together with favorable slope geometry generally results into rotational/ circular mode of failure.

The fundamental triggering factor for the process of instability in the present study area is rainfall as this tends to raise the groundwater level which in turn induces increase in void pressure around failure surface and causes slope instability. Also due to the continuous supply of colluvial material from the upper section, there will be an increase in weight on the lower sides that will generally increase the amount of the horizontal stress which in turn favors sliding. This can be evidenced by the remarkable collapsing of the tress as observed along the slope on the main road.

4.4.1.1 Input Parameters for slope stability analysis (SS1)

As already discussed in Chapter 3, engineering properties of colluvial material are extremely difficult to determine in the lab or in in-situ state. A useful method for determining shears strength properties in colluviall materials is to analyze an existing slope failure (WSDOT Geotechnical Design Manual, 2010).

The representative shear strength parameter values for the present slope stability analysis (SS1) were adopted by conducting a sensitivity analysis. The sensitivity analysis was carried out for the different values of cohesion and angle of internal friction by keeping one of these values constant at a time. The results of the sensitivity analysis indicated that material properties play a significant role in controlling slope processes and that the slope geometry and climate condition should not be considered as the only parameter to define instability. By evaluating the results of sensitivity analysis in slope stability model, one can understand easily the relative importance of individual parameter on stability condition.

The sensitivity analysis for SS1 slope section (Table 3.1, 3.2, Chapter 3) reveals in general that as the shear strength parameters values increases FOS also increases. However, within the permissible limits of each parameter the effect of FOS is same. In terms of relative importance both parameters are contributing equally to FOS.

Determination of the unit weight

The unit weight used in analysis was obtained by bulk density and dry density values obtained from laboratory tests conducted on finer samples. From the bulk and dry density result, the unit weight was determined. For the stability analysis of SS1 slope section the unit weight used for dry and saturated conditions was considered as 16.464KN/m^3 and 19.404KN/m^3, respectively.

For the dynamic analysis, both Slope/W and Slide software incorporates the earthquake loading using a static force equal to the weight of the potential failure mass multiplied by a seismic coefficient, k (acceleration produced by an earthquake). Regarding the earth quake coefficients of Peak Ground Acceleration (PGA) values for the study area, there are several publications and building standards with variable values.

According to the seismic hazard map of Ethiopia (GSHAP data for a return period of 475 years); the study area is located near and around the Afar region characterized by a PGA of

0.16g to 0.24g, (Asrat Worku, 1995). In the present study PGA equal to 0.24g was adopted as an average value for the study area for the dynamic slope stability analysis.

Based on sensitivity analysis for the slope stability analysis, the angle of internal friction was adopted to be 31.3^0 and that of cohesion (C) was 15.6 KPa for the dry condition. For the worst anticipated and present condition, adjustment was made for the cohesion, since cohesion of the colluvial soil materials may be reduced by the impact of the water, i.e. when it become partial and fully saturated. The value of cohesion adopted for present condition was taken as 14.4 KPa and for worst anticipated condition it was taken as 13.2 KPa. The input values for cohesion, internal friction and unit weight used for various conditions during stability analysis of SS1 section were presented in Table 3.4 Chapter 3.

4.4.2 Critical Slope Section SS2

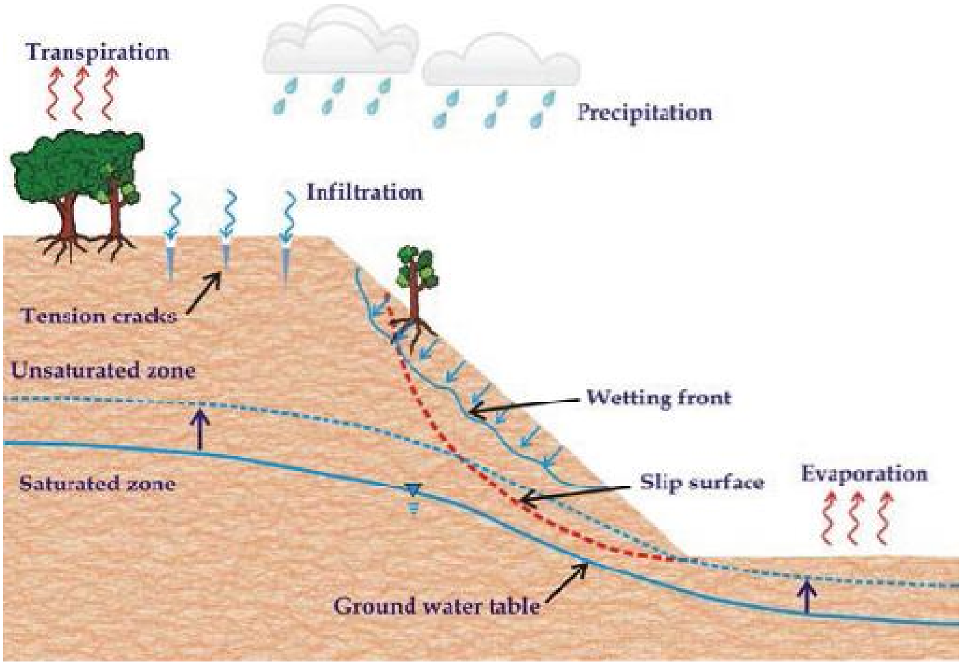
Critical Slope Section SS2 is defined by UTM coordinates 0569952mE/1227939mN and is located on the side of the road near to the Mehmber Tseay School. The slope section mainly comprises of residual soils (clayey silty) and possess rotational mode of failure.

The residual soils in the present study area were developed mainly from extensive physical and chemical in-situ weathering and decomposition of basaltic rock. Hence, these residual soils possess characteristics which are different from those of transported soils. Since the formation process of the residual soils is usually complex hence it is usually described in terms of different weathering grades. The weathering profile normally has significant effect on the slope stability analysis in residual soils because it may determine potential failure surface and mode of failure, groundwater hydrology and erosion characteristics of the soil materials. Tension cracks can form at the top of the slip circle in soils that have some cohesion. The appearance of such cracks (which can run parallel to the top of the bank or form the arc of a circle) is often the first indication of slope instability (Reference).

From field manifestations cracking of Mehmber Tseay School building, located on the top of this slope section (SS2), displacement of retaining wall constructed on the toe of the school building and sliding of soil within constructed structure indicate that the slope section SS2 is actually and potentially unstable.

In general, what was observed in this critical slope section is that the failure surface shows scarp on the steep slope and probably these were developed into an arc segment that serves as a potential failure plane (Plate 4.2). Any available water body may likely pass through this scrap and probably reduces the shear strength of the soil and also it may lubricant the sliding surface, further facilitating the process of sliding.

Plate 4.2 Sliding of silty clay soil along Critical slope section SS2



(Source: after Rahardjo et al., 2007)

Fig 4.2 Mechanism of rainfall-induced slope failure

4.4.2.1 Input Parameters for slope stability analysis (SS2)

The shear strength parameters (Internal friction angle and Cohesion) and unit weight of the soil (disturbed) for SS2 slope section were determined by using direct shear test in Geotechnical Laboratory of Transport Construction Design Share Company (TCDSCo). The test results used as input parameters for slope stability analysis of SS2 slope section are shown in Table 4.3.

Table 4.3 Material Properties and Laboratory test result of critical (SS2)

Section	Material	Internal friction angle(^{0})	Cohesion (KPa)	Bulk Unit weight(gm/cm^3)	Dry unit weight(gm/cm^3)
SS2	Silty clay	13	78	1.73	1.36

Determination of the unit weight

The unit weight used in analysis was obtained by bulk and dry density values obtained from laboratory tests conducted on representative samples. For the stability analysis of SS1 slope section the unit weight used for dry and saturated conditions was considered as 13.328KN/m^3 and 16.954KN/m^3, respectively.

Further, for stability analysis of SS2 slope section based on the sensitivity analysis and Laboratory result, the angle of internal friction was adopted to be as 5.8^0 and that of cohesion (C) as 78.2 KPa for the dry conditions. For the worst anticipated and present condition, adjustment was made on the cohesion, since cohesion of the silty clay soil materials may be reduced by the impact of the water, i.e. when it become partial and fully saturated. The value of cohesion adopted for present condition was taken as 76.6 KPa and for worst anticipated condition it was 75 KPa. The input values for cohesion, internal friction and unit weight used for various conditions during stability analysis of SS2 section were already presented in Table 3.5 Chapter 3.

4.5 Groundwater Conditions consideration for the present study

One of the factors that control the slope instability is groundwater fluctuation and its movement through the underground. Most slope failures are preceded by saturation of the slope. Subsurface water buildup can cause landslides in several possible ways. The water may increase the weight of materials on a slope above their point of gravitational equilibrium. It may increase pore pressures within a zone of weakness in the materials underlying a slope. Further, it may also decrease the coefficient of friction on a potential sliding surface (Duncan and Wright, 2005).

Slope failures within colluviall often occur during high rainfall. Among the possible causes are the reduction in the shear strength at the interface between the colluviall and the residual soil by the increase in the pore pressures.

The main source of the groundwater recharge in the study area is rainfall. The water that comes from rainfall will infiltrate into the ground through the colluvial material in the area. During the present study ground water level data was not available. However, based on rainfall precipitation in the study area and by field manifestation slope stability analysis was carried out for anticipated condition represented as; (i) moderately saturation condition by considering the ground water level up to mid slope height such condition may prevail during moderate rainfall and (ii) fully saturation condition by considering ground water level on 3/4 three fourth of slope height, this may be a situation when there are prolonged rains and much of the slope section is saturated.

4.6 Effect of Rainfall on the overall slope stability

The rainfall trend in the present study area, as already discussed in Chapter 1, is high from June to September. The area is characterized by sub-humid to humid climate and with average annual rainfall of 1385 mm, which is quite high compared to many places in the north-western highlands of Ethiopia (Gebreslassie Mebrahatu, 2011). It usually peaks either in July or August at about 350 to 420 mm per month. Due to high rainfalls in these months ground water level increase. This increase in the level of the ground water tends to develop a high pore water pressure within the soil/colluvial material as well as it serves as a lubricant that later induces instability. Thus, it may be concluded that in the present study area rainwater is the main triggering factor for the movement of the colluvial material down the slope.

4.7 Conditions for which stability analysis was made

During the present study factor of safety was determined for existing and possible worst conditions, represented as; (i) static dry condition, (ii) static moderately saturated/ Present Condition and (iii) static worst anticipated condition. (iv) dynamic dry condition, (v) dynamic moderately saturated and (vi) dynamic worst anticipated condition. A detailed description on all these conditions has already been made under Para 3.5, Chapter 3.

The primary aim of considering these six conditions for stability analysis was not only to evaluate the minimum Factor of Safety (FOS) but also to see how these combinations of conditions in the natural environment may affect the slope stability condition in the area in the future.

4.8 Slope stability analysis

During the present study stability analysis was conducted by using, Slope/W and Slide, commercially available software. Further, the Factor of safety was determined for existing and possible worst conditions, represented by six conditions as mentioned in the previous paragraph.

Assumptions made for Present Slope Stability Analysis

As described above, the sensitivity analysis was conducted using Slide software for different shear strength parameter values for cohesion and internal friction angle. The results of the sensitivity analysis for SS1 and SS2 slope sections are already presented in Tables 3.4 and 3.8 Chapter 3.

Based on the sensitivity analysis of the shear strength parameters for both worst anticipated conditions and present condition, adjustment was made on the cohesion, since cohesion of the colluvial soil materials and silty clay soil may be reduced by the impact of the water, i.e. when it become partial and fully saturated. For critical slope section SS1 the value of cohesion for present condition was taken as 14.4 KPa and for worst anticipated condition it was taken as 13.2 KPa. For critical slope section SS2 the value of cohesion for present condition was taken as 76.6 KPa and for worst anticipated condition it was taken as 75 KPa. Further, the input unit weight was calculated from both bulk and dry density for both the slope sections. The input values adopted for unit weight used in stability analysis has already been discussed in the previous paragraphs.

4.8.1 Static Stability Analysis using Slope/W for slope section SS1

Static stability analysis was performed by neglecting the effect of earthquake. Static stability analysis was conducted for three different cases; for dry condition, for the present condition and for the anticipated worst condition.

Static dry condition

For the slope stability analysis in the dry condition the groundwater level is considered to be much below the influence zone in the cross section i.e. the groundwater level is believed to be below the possible failure surface. Thus, the slope section was considered to be dry and it was assumed that there will not be any effect of the groundwater in inducing instability to the slope.

The FOS was computed by Slope/W for static dry condition for SS1 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The results thus obtained are presented in Table 4.5.

Perusal of Table 4.5 clearly indicates that the FOS values by BS and M-P methods is slightly greater than 1, which may be considered as critically stable. However, the safe FOS value for road cut slope is considered to be 1.2 (Hoek and Bray, 1989). Thus, it may be concluded that SS1 slope section is unstable for static dry condition.

Table 4.5 Input parameters and FOS for static conditions SS1slope section by Slope/W

Condition	**Unit weight**	**Cohesion**	**Friction angle**	**FOS**	
				BS	**M-P**
Static dry	16.464	31.3	15.6	1.062	1.055
Static present	19.404	31.3	14.4	0.919	0.912
Static worst	19.404	31.3	13.3	0.782	0.792
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

Static present condition

For the slope stability analysis, under the present condition, the groundwater table was determined from the three borehole measurement records. The average groundwater level as deduced was at half (1/2) of the slope height (SS1) and it was presumed that the slope section is moderately saturated.

The stability analysis results (Table 4.5) indicates, that the FOS of the slope section (SS1) under the present condition is 0.919 (BS) and 0.912 (M-P). These results clearly show that the slope is unstable during medium saturation condition as the FOS values are less than 1.0.

Static anticipated worst condition

For the anticipated worst condition, the slope stability analysis was conducted by assuming that the three fourth of the slope section is saturated. The stability analysis results (Table 4.5) indicates, that the FOS of the slope section (SS1) under the Static anticipated worst condition is 0.782 (BS) and 0.792 (M-P).

These results clearly indicate that the slope would be completely unstable during static anticipated worst condition as the FOS values are far less than 1.0.

4.8.2 Dynamic Stability Analysis using Slope/W for slope section SS1

For the dynamic stability analysis, the input parameters used were same as that were used for static case. However, in the dynamic stability analysis the effect of earthquake was also considered and a coefficient of Peak Ground Acceleration (PGA) equal to 0.24g was adopted.

Further, the FOS was computed by Slope/W for dynamic dry condition for SS1 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The input data used and the computed factor of safety for existing and anticipated conditions are presented in Table 4.6.

Table 4.6 Input parameters and FOS for dynamic conditions for slope section SS1 by Slope/W

Condition	**Unit weight**	**Cohesion**	**Friction angle**	**FOS**	
				BS	**M-P**
Dynamic dry	16.464	31.3	15.6	0.688	0.683
Dynamic moderately saturated	19.404	31.3	14.4	0.583	0.81
Dynamic worst	19.404	31.3	13.3	0.42	0.432
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

Dynamic dry condition

For this case the slope section was considered to be dry and it was assumed that there will be dynamic loading. Perusal of Table 4.6 clearly indicates that the FOS values by 0.688 (BS) and 0.683 (M-P) methods are less than 1, indicating that the slope section would be unstable for anticipated dynamic dry condition.

Dynamic moderately saturated condition

For the slope stability analysis, under the dynamic moderately saturated condition, it was presumed that the slope section is saturated up to half (1/2) of the slope height (SS1) and it was presumed that the dynamic loading occurs. The stability analysis results (Table 4.6) indicates, that the FOS of the slope section (SS1) under the dynamic moderately saturated condition is 0.583 (BS) and 0.581 (M-P). These results clearly show that the slope is unstable during dynamic moderately saturated condition as the FOS values are far less than 1.0.

Dynamic anticipated worst condition

For the dynamic anticipated worst condition, the slope stability analysis was conducted by assuming that the three fourth of the slope section is saturated and dynamic loading occurs. The stability analysis results (Table 4.5) indicates, that the FOS of the slope section (SS1) under the Static anticipated worst condition is 0.42 (BS) and 0.432 (M-P). These results clearly indicate that the slope would be completely unstable during dynamic anticipated worst condition as the FOS values are far less than 1.0.

4.8.3 Static Stability Analysis by using Slide software for slope section SS1

For the stability analysis by Slide software, the input parameters were same as used for Slope/W. Static stability analysis was performed by neglecting the effect of earthquake. Static stability analysis was conducted for three different cases; for dry condition, for the present condition and for the anticipated worst condition.

The FOS was computed by Slide Software for static condition for SS1 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The results thus obtained are presented in Table 4.7.

Table 4.7 Input parameters and FOS for static conditions for SS1 slope section by Slide

Condition	Unit weight	Cohesion	Friction angle	FOS	
				BS	M-P
Static dry	16.464	31.3	15.6	1.01	1.007
Static present	19.404	31.3	14.4	0.889	0.888
Static worst	19.404	31.3	13.2	0.597	0.614
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

Static dry condition

For the slope stability analysis in the dry condition the groundwater level is considered to be much below the influence zone in the cross section i.e. the groundwater level is believed to be below the possible failure surface. Thus, the slope section was considered to be dry and it was presumed that there may not be any possible effect of the groundwater in inducing instability to the slope. From the stability analysis results (Table 4.7), it was found that the FOS of SS1 section under dry condition will be 1.01 (BS) and 1.007 (M-P). These values of FOS clearly indicates that the slope section is unstable (Design FOS for road section must be > 1.2, Hoek and Bray (1989))

Static present condition

For the slope stability analysis, under the present condition, the groundwater table was determined from the three borehole measurement records. The average groundwater level was deduced to be half (1/2) of the slope section height and slope section was presumed to be moderately saturated. The analysis results indicate that the, the FOS for the slope section SS1 under the present condition is 0.889 (BS) and 0.888 (M-P). These values of FOS clearly show that the slope section is unstable as the values of FOS are less than 1.0.

Static anticipated worst condition

For the anticipated worst condition, the slope stability analysis was conducted by assuming that the three fourth (3/4) of slope section is saturated. This will be useful to estimate the factor of safety for the worst condition. Under this condition, FOS of SS1 slope section is computed to be 0.597 (BS) and 0.614 (M-P). These values of FOS clearly show that the slope section is unstable as the values of FOS are less than 1.0.

4.8.4 Dynamic Stability Analysis using Slide for slope section one

For the dynamic stability analysis, the input parameters used were same as that were used for static case. However, in the dynamic stability analysis the effect of earthquake was also considered and a coefficient of Peak Ground Acceleration (PGA) equal to 0.24g was adopted.

Further, the FOS was computed by Slide software for dynamic conditions for SS1 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The input data used and the computed factor of safety for existing and anticipated conditions are presented in Table 4.8.

Table 4.8 Input parameters and FOS for dynamic conditions for SS1slope section by Slide software

Condition	**Unit weight**	**Cohesion**	**Friction angle**	**FOS**	
				BS	**M-P**
Dynamic dry	16.464	31.3	15.6	0.694	0.697
Dynamic Moderately saturated	19.404	31.3	14.4	0.59	0.609
Dynamic worst	19.404	31.3	13.2	0.38	0.423
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

A perusal of Table 4.8 clearly indicates that for all three anticipated dynamic conditions the FOS values by both Bishop Simplified methods (BS) and Morgenstern-Price method (M-P) are far less than 1.0. Thus, it may be concluded that slope section SS1 would be completely unstable under dynamic loading with or without water saturation.

4.8.5 Static Stability Analysis using Slope/W for slope section SS2

Static stability analysis was performed by neglecting the effect of earthquake. Static stability analysis was conducted for three different cases; for dry condition, for the present condition and for the anticipated worst condition.

The FOS was computed by Slope/W Software for static condition for SS2 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The results thus obtained are presented in Table 4.9.

Table 4.9 Input parameters and FOS for static conditions for SS2 slope section by Slope/W

Condition	Unit weight	Cohesion	Friction angle	FOS	
				BS	M-P
Static dry	13.328	78.2	5.8	1.253	1.251
Static present	16.954	76.6	5.8	0.939	0.938
Static worst	16.954	75	5.8	0.873	0.869
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

A perusal of Table 4.9 clearly indicates that the Slope Section SS2 is stable for static dry condition as the FOS values are more than 1.2. However, for water saturation conditions; static present (moderately saturated) and static worst (3/4 th saturation) the slope becomes unstable as for these conditions the FOS is less than 1.0.

4.8.6 Dynamic Stability Analysis using Slope/W for slope section SS2

For the dynamic stability analysis, the input parameters used were same as that were used for static case. However, in the dynamic stability analysis the effect of earthquake was also considered and a coefficient of Peak Ground Acceleration (PGA) equal to 0.24g was adopted.

Further, the FOS was computed by Slope/W software for dynamic conditions for SS2 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The input data used and the computed factor of safety for existing and anticipated conditions are presented in Table 4.10.

Table 4.10 Input parameters and FOS of dynamic conditions SS2 by Slope/W

Condition	Unit weight	Cohesion	Friction angle	FOS	
				BS	M-P
Dynamic dry	13.328	78.2	5.8	0.817	0.815
Dynamic present	16.954	76.6	5.8	0.613	0.610
Dynamic worst	16.954	75	5.8	0.589	0.586
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

A perusal of Table 4.10 indicates that for all three anticipated dynamic conditions the FOS values by both Bishop Simplified methods (BS) and Morgenstern-Price method (M-P) are far less than 1.0. Thus, it may be concluded that slope section SS2 would be completely unstable under dynamic loading with or without water saturation.

4.8.7 Static Stability Analysis using Slide for slope section SS2

Static stability analysis was performed by neglecting the effect of earthquake. Static stability analysis was conducted for three different cases; for dry condition, for the present condition and for the anticipated worst condition.

The FOS was computed by Slide Software for static condition for SS2 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P). The results thus obtained are presented in Table 4.11.

Table 4.11 Input parameters and FOS of static conditions for slope section SS2 by Slide Software

Condition	Unit weight	Cohesion	Friction angle	FOS	
				BS	**M-P**
Static dry	13.328	78.2	5.8	1.214	1.212
Static present	16.954	76.6	5.8	0.91	0.909
Static worst	16.954	75	5.8	0.855	0.856
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

A perusal of Table 4.11 clearly indicates that the Slope Section SS2 is stable for static dry condition as the FOS values by both BS and M-P methods are more than 1.2. However, for water saturation conditions; static present (moderately saturated) and static worst (3/4 th saturation) the slope becomes unstable as for these conditions the FOS is less than 1.0.

4.8.8 Dynamic Stability Analysis by using Slide software for slope section SS2

For the dynamic stability analysis, the input parameters used were same as that were used for static case. However, in the dynamic stability analysis the effect of earthquake was also considered and a coefficient of Peak Ground Acceleration (PGA) equal to 0.24g was adopted.

Further, the FOS was computed by slide software for dynamic conditions for SS2 slope section by Bishop Simplified method (BS) and Morgenstern-Price method (M-P).

The input data used and the computed factor of safety for existing and anticipated conditions are presented in Table 4.12.

Table 4.12 Input parameters and FOS of dynamic conditions for SS2 slope section by Slide software

Condition	Unit weight	Cohesion	Friction angle	FOS	
				BS	**M-P**
Dynamic dry	13.328	78.2	5.8	0.782	0.779
Dynamic present	16.954	76.6	5.8	0.583	0.581
Dynamic worst	16.954	75	5.8	0.546	0.546
FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method					

A perusal of Table 4.12 indicates that for all three anticipated dynamic conditions the FOS values by both Bishop Simplified methods (BS) and Morgenstern-Price method (M-P) are far less than 1.0.

Thus, it may be concluded that slope section SS2 would be completely unstable under dynamic loading with or without water saturation.

4.9 Result and discussion

During the present study slope stability analysis was conducted for the different conditions (dry, present and worst anticipated conditions) under both static and dynamic states. For the computation of stability condition of critical slope sections deterministic approach using Slope/W and Slide software were used. In both slope/W and Slide FOS was computed by utilizing Bishop Simplified methods (BS) and Morgenstern-Price method (M-P). General discussion on result for two critical slope sections studied during present study is presented in the following paragraphs. The slope model pictures used for analysis in both Slope/W and Slide are presented in (Appendix C).

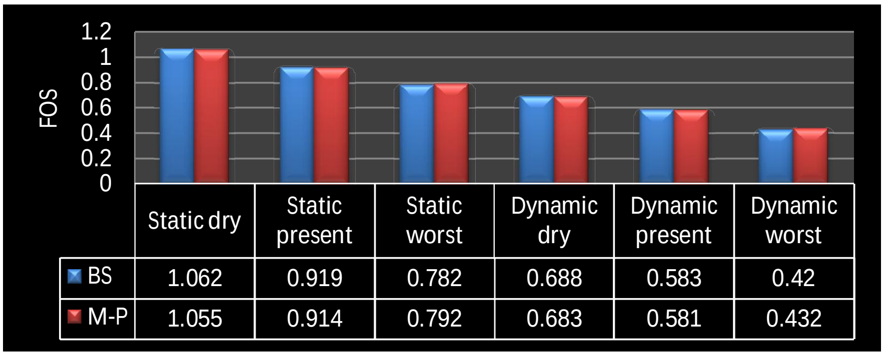
4.9.1 Results of Slope stability for slope section SS1 by Slope W and Slide

The results of stability analysis from the Slope/W and Slide were obtained as both visuals and numbers. The results (FOS) of the static and dynamic slope stability analyses by Slope/W and Slide software for critical slope sections by using Bishop Simplified method (BP) and Morgenstern-Price methods (M-P) are presented in Table 4.13 and Fig.4.3..6.

Table 4.13 Stability analysis result for Static and dynamic condition for SS1 and SS2 slope sections

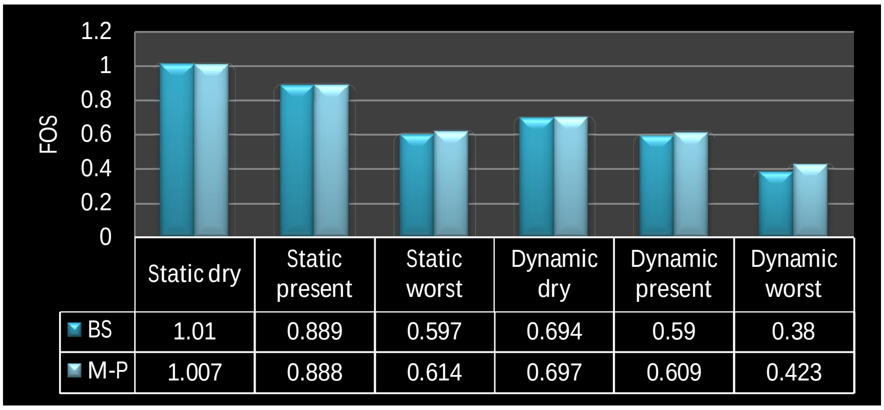
Conditions	Factor of Safety (FOS)							
	Slope Section SS1				Slope Section SS2			
	Slope W		Slide		Slope W		Slide	
	BS	M-P	BS	M-P	BS	M-P	BS	M-P
Static dry	1.062	1.055	1.01	1.007	1.253	1.251	1.214	1.212
Static present	0.919	0.914	0.889	0.888	0.939	0.939	0.91	0.909
Static worst	0.782	0.792	0.597	0.614	0.873	0.869	0.885	0.856
Dynamic dry	0.688	0.683	0.694	0.697	0.817	0.815	0.782	0.779
Dynamic present	0.583	0.581	0.59	0.609	0.613	0.610	0.583	0.581
Dynamic worst	0.42	0.432	0.38	0.423	0.589	0.586	0.546	0.546
BS: Bishop Simplified method, **M-P**: Morgenstern-Price method								

In the present study slope stability analysis was carried out for two critical slope sections (SS1 and SS2). Both the slope sections are soil slope sections and comprises of colluviums and silty clay type of soil. Since both the critical slope sections are composed of soils therefore the probable mode of failure in both the cases would be circular mode of failure. Further, the stability analysis was carried out by using the shear strength parameters that were adopted from the sensitivity analysis and the unit weight was derived from the laboratory bulk and dry density result. Slope stability analysis was conducted for the different conditions (dry, present and worst anticipated conditions) under both static and dynamic states following deterministic approach using Slope/W and Slide software by using Bishop Simplified method (BP) and Morgenstern-Price methods (M-P).



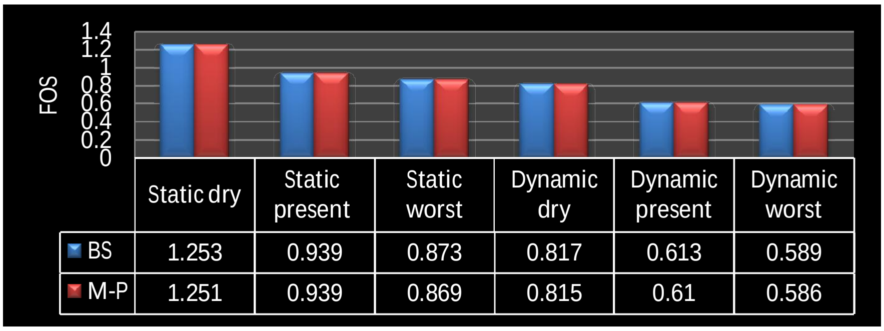
	Static dry	Static present	Static worst	Dynamic dry	Dynamic present	Dynamic worst
BS	1.062	0.919	0.782	0.688	0.583	0.42
M-P	1.055	0.914	0.792	0.683	0.581	0.432

SS1 Slope Section – FOS Results by Slope/W



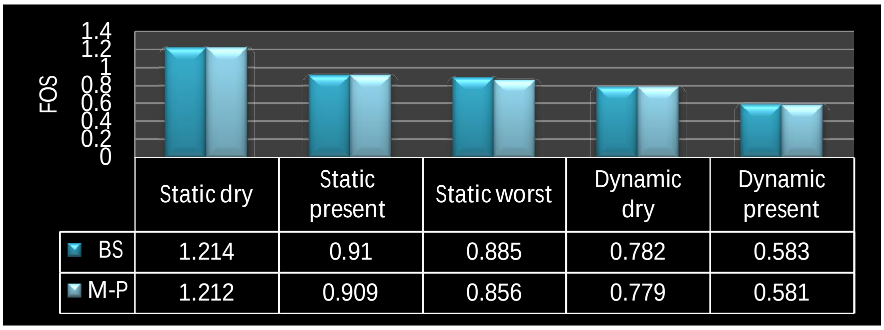
	Static dry	Static present	Static worst	Dynamic dry	Dynamic present	Dynamic worst
BS	1.01	0.889	0.597	0.694	0.59	0.38
M-P	1.007	0.888	0.614	0.697	0.609	0.423

SS1 Slope Section – FOS Results by Slide



	Static dry	Static present	Static worst	Dynamic dry	Dynamic present	Dynamic worst
BS	1.253	0.939	0.873	0.817	0.613	0.589
M-P	1.251	0.939	0.869	0.815	0.61	0.586

SS2 Slope Section – FOS Results by Slope/W



	Static dry	Static present	Static worst	Dynamic dry	Dynamic present
BS	1.214	0.91	0.885	0.782	0.583
M-P	1.212	0.909	0.856	0.779	0.581

SS1 Slope Section – FOS Results by Slide

FOS: Factor of safety, **BS**: Bishop Simplified method, **M-P**: Morgenstern-Price method

Fig4.3. Graphical presentation of FOS Results by BS & M-P results from Slope/W and Slide software indicating stability under varied condition (SS1 and SS2)

Slope Section SS1

A perusal of results presented in Table 4.13 and Fig.4.6 clearly shows that the slope section SS1 is critically stable for static dry condition as the FOS is slightly more than 1.0, however with respect to design FOS for road cut slope has to be more than 1.2. Thus, for static dry condition slope section SS1 may be considered as unstable. Further, perusal of FOS values for SS1 slope section for all static and dynamic conditions with or without water saturation are less than 1.0 which clearly shows that for all anticipated conditions, present or worst, the slope section SS1 is unstable. The stability results by both software Slope/W and Slide gave similar type of results. Further, when compared FOS results by Bishop Simplified method (BP) and Morgenstern-Price methods (M-P) in both Slope/W and Slide also gave comparable results. Thus, it may be concluded that the slope section SS1 is unstable for existing and anticipated adverse conditions.

The same observations were made during the field visit by visual observations of slope distress manifestations on account of heterogeneity in slope material, its morphology, past evidences of instability and other indicators of instability. As already discussed, this section is mainly characterized by a huge mass movement of colluvial material with circular/ rotational mode of failure. Further, the effect of rainfall significantly contributes towards the instability of the slope; and the behavior of the topography and engineering properties of colluvial material present in the slope section also plays a great role.

Pack, et al (1998) classified slopes based on FOS; as low landslide susceptibility (if FOS >1.5): moderate landslide susceptibility (1.5 > FOS > 1.25): high landslide susceptibility (1.25 > FOS > 1): and very high landslide susceptibility (FOS < 1). Thus, based on the susceptibility classification of Pack, et al (1998), the calculated FOS values for Slope section SS1 for all anticipated conditions fall in the class “very high landslide susceptibility”. Thus, there is a need to take all necessary stabilization measures to mitigate the problem of instability of SS1 slope section.

Slope Section SS2

A perusal of results presented in Table 4.13 clearly shows that the slope section SS1 is only stable for static dry condition as the FOS is slightly more than 1.2. Thus, for static dry condition slope section SS1 may be considered as stable. However, perusal of FOS values for SS2 slope section for all static and dynamic conditions with or without water saturation are less than 1.0 which clearly shows that for all anticipated conditions, present or worst, the slope section SS2 would be unstable.

From the field manifestations cracking of Mehmber Tseay School building, located on the top of this slope section (SS2), displacement of retaining wall constructed on the toe of the school building and sliding of soil within constructed structure indicate that the slope section SS2 is actually and potentially unstable. The same fact is validated by the slope stability results. Further, in general, what was observed in this critical slope section is that the failure surface shows scarp on the steep slope and probably these were developed into an arc segment that serves as a potential failure plane (Plate 4.2). Any available water body may likely pass through this scrap and probably reduces the shear strength of the soil and also it may lubricant the sliding surface, further facilitating the process of sliding.

Even with Peck et al., (1998) classification Slope section SS2 for all anticipated conditions fall in the class “very high landslide susceptibility”. Thus, there is a need to take all necessary stabilization measures to mitigate the problem of instability for SS2 slope section.

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CHAPTER FIVE REMEDIAL AND PREVENTIVE MEASURES

5.1 Introduction

If the result of the stability analysis indicate that the road cut slopes does not meet the minimum factor of safety requirement, then it may be necessary to use slope stabilization methods. Now a day, with the advance of technology and development of construction industry several slope stabilization methods to mitigate slope failure along the road and others civil structures has been developed.

In general, slope stabilization methods can be placed in one of two broad categories:

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

According to Abramson, (2001) the stability of any slope will be improved if certain actions are carried out. To be effective, first one must identify the most important controlling process that is affecting the stability of the slope; second, one must determine the appropriate technique to be sufficiently applied to reduce the influence of that process.

In the present study area slopes are vulnerable to instability problems. At many places the slopes material (rock and soil) has failed in to the road. The road slope instability disaster has not only created the enormous direct economic loss, but also the indirect economic loss which is difficult to estimate and the bad social impact because of the disaster interrupting traffic (Li et al., 2009).

Therefore, besides slope stability analysis of the study area, finding appropriate remedial and preventive measures for critical slopes will be very helpful to ease or reduce the economic and social problems in the area, particularly during rainy season.

From the present study, it was found that combination of different factors (rainfall, engineering properties soil, geology, topography, and others) are responsible for inducing slope instability in the study area.

Therefore, combination of different mitigation measures should be applied to reduce the instability condition induced in the study area. The remedial measures to be taken should also be cost effective and feasible. Stabilization of a slope depends on number of factors such as; its geometry, surface and groundwater conditions, strength of materials and the reason for stabilization. A number of techniques such as; modification of slope geometry, proper managements of drainages, retaining structures, internal slope reinforcement have been developed to stabilize slopes considering the above mentioned conditions (Abramson et al., 1996).

Depending on site condition and slope stability analysis result of the study area four types of suitable preventive and remedial measures are recommended namely; proper managements of drainages, retaining structures, gabions, and managing steeply cut slopes.

5.2 Proper management of drainage

Drainage is by far the most frequently used means of stabilizing slopes and less expensive than other methods of stabilization, and a large volume of ground can frequently be stabilized at a relatively low cost. Drainage is frequently used method, either alone or in conjunction with other methods.

Drainage improves slope stability in two important ways:

- ✓ It reduces pore pressures within the soil, thereby increasing the effective stress and the shear strength; and
- ✓ It reduces the driving forces of water pressures in cracks, thereby reducing the shear stress required for equilibrium.

Surface water is controlled to eliminate or reduce infiltration and to provide erosion protection. Cut slopes should be protected with interceptor drains installed along the crest of the cut, along benches and along the toe. The application of appropriate drainage is the most effective means of mitigating slope failure hazard regardless of the type of failure due to the important role played by pore-water pressure in reducing shear strength.

It is also relatively low cost method as compared with other methods. The seasonal groundwater seepages and surface water have serious impact on triggering the landslide in the area.

Hutchinson (1977) has indicated that drainage is the principal measure used in the repair of landslides, with modification of slope geometry.

For example, surface and groundwater drainage system can be one of the main mitigation mechanisms for the landslide along the main Road of Kombolcha- Desse site. The surface water can be collected into a collection chamber and connected to a drainage channel to remove the water safely from the slope face. Therefore, avoiding/reducing any water which infiltrate into the slopes and draining out from the slope without barrier, will reduce the pore pressure developed within the slope material.

Therefore, the shear strength of the slope material may not be excessively affected which will causes slope instability. Hence; proper management of the drainages in the study area will be key solution to reduce the instability of the slopes. Critical slope sections, those formed by colluvial soils in the study area are highly susceptible to slope failure. These areas are exposed for rapid saturation from infiltration of surface runoff and direct infiltration from precipitation, especially during rainy seasons.

Plate 5.1 Due to lack of proper drainage slope material collapsed into road by rainfall in study area.

In order to properly manage the drainage system of the slopes some of the general remedial measures are; construction of trench drain, interceptor drain and construction of collection chamber and diverting the water to the existing drainage systems. The drainage ditches which were constructed in most parts of the slopes in the present study area may allow continuous infiltration of water through soil mass. This may further add to the slope instability.

5.3 Retaining Structures

According to Abramson et al., (1996) retaining wall can be constructed through an unstable slope to provide additional resistance and raise the factor of safety for material behind the wall to an acceptable level. Retaining structures should be founded in stable earth materials. The retaining structure should be evaluated for possible sliding, overturning, and bearing failures using standard techniques. Failure surfaces that extend below the wall foundation and above the top of the wall also should be analyzed.

Analysis of walls that support bedded rock dipping toward the wall is facilitated by use of a computer program that also allows the use of anisotropic strength parameters. Consideration must be given to whether material in front of the wall that is assumed to provide passive resistance could be removed or excavated in the future. Loose materials, such as colluvial materials, slide debris, and broken rock, on the slope that could pose a hazard can be collected by a retaining structure capable of holding the volume of material that is expected to fail.

The retaining wall constructed at the side of the main road in the study area is filled by colluvial material. Because of this reason colluvial material easily flow into road. Therefore, the retaining structures which will be constructed should be properly designed in such a way which hold the collapsed material and should have sufficient drainage outlets.

5.4 Gabions

Gabions are wire mesh, boxlike containers filled with cobble-sized rock. A gabion can also be constructed from stacked gabions. Gabion walls usually are inexpensive and are simple and quick to construct. Due to their flexibility, they can withstand foundation movement, and they do not require elaborate foundation preparation. Because of their coarse fill, they are very permeable and thus provide excellent drainage.

Gabion walls work because the friction between the individual gabion rows is very high, as is the friction between the basal row and the soil underneath. When failure occurs, it is almost always in the foundation soil itself. Gabion walls built on clay soils require counterforts, which can be constructed as gabion headers extending from the front of the wall to beyond the slip circle. The counterforts serve as both structural components and as drains (Hutchinson, 1977).

5.5 Managing steeply cut slopes

The road construction companies have steeply cut some slopes in Kombolcha to Desse road and dumped the excavated material down the slope in unplanned manner. The poorly graded colluvial material, soils, and highly weathered basalts, become unstable in the steep rock mass cut. This loose material which is dumped down the slope in unplanned manner may easily be eroded and degraded. These make the overlying road to be affected by landslide. Therefore, it is necessary that slope cut must be properly designed and unplanned dumping of soil material should be avoided.

5.6 Remedial Measures for critical Slope Sections considered in present study

The study area is characterized by high relief and very steep slopes having a circular (rotational) mode of failure. In the stability analysis it was attempted to point out that the effect of rainfall is high for the instability of the slope section during summer. For this rainfall saturation has a major role for that specific section. Also, the colluvial material that comes from the steep slope parts of the block is responsible for the increment of the total weight on the lower slope section which in turn results into the instability of the slope section. This process is manifested by the collapse of the existing retaining wall that was found at the sides of the main road. Thus, these and other related problems intensify the instability condition of the slope.

As a principle there may be actions in the form of curative and preventive so as to solve the problems of the slope instability. However, for these sections a preventive work was recommended as it aims at prevention of the surface water from entering the potential slope failure area and also draining of surface water out from the area in a relatively safer way.

For this, based on the existing problem following recommendations are proposed;

- ✓ Soil removal from the upper portions of the block that is responsible for the increment of the soil mass of the lower portion. Also any soil mass that is available on the lower sections which are responsible for slides should be removed and attention should be given to this activities,

- ✓ Mechanism to drain surface water by making a proper design of open ditches and conduits which may drain any available water body on the slope section,

- ✓ Horizontal drainage mechanisms from the face of the slope that may reduce the existing water pressure that may develop. However, these works should be continued with other activities that may remove the water to a safer portion of the slope section,

- ✓ Retaining wall at the lower toe sections of the block may be recommended. However, it should strictly have a water passage holes at the faces of the wall so as to reduce the horizontal pressure from the water body at the back of the wall.

- ✓ The road construction companies have steeply cut some slopes in the study area and dumped the excavated material down the slope in unplanned manner. The poorly graded colluvial material, soils, and highly weathered basalts, become unstable on the steep rock mass cut. This loose material which is dumped down the slope in unplanned manner may easily be eroded and degraded. This may intern will make the overlying road to be affected by landslide. Therefore, it is necessary that properly slope cut should be designed and unplanned dumping of waste should be avoided.

5.6.1 Remedial measures for Slope Section one (SS1)

In this slope section colluviall material is characterized by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders located at the base of cliffs and steep slopes and the thickness increases towards the base of the slopes. Any available water body passes through this scarp reduces the shear strength of the soil and colluvial material. Further, this water body serves as a lubricant for the failure mass. Consequently, huge mass of colluvial material may collapse on to the main road which may pose difficulty in transport activity. Thus, to avoid such anticipated problems following measures may be adopted;

- ✓ Proper management of the drainages in this section will be key solution to reduce the instability of the slope. This section is exposed for rapid saturation from infiltration of surface runoff and direct infiltration from precipitation, especially during rainy seasons.

- ✓ Soil removal from the upper portions of the block that is responsible for the increment of the soil mass of the lower portion. Also, any soil mass that is available on the lower sections which may be responsible for slides should be removed and attention should be given to this activities,

- ✓ Retaining wall at the lower toe sections of the block may be recommended. However, it should strictly have a water passage holes at the faces of the wall so as to reduce the horizontal pressure from the water body at the back of the wall,

- ✓ Mechanism to drain surface/ sub surface water by making a proper design of open ditches and conduits which may drain any available water body on the slope section,

- ✓ Horizontal drainage mechanisms from the face of the slope reduce the existing water pressure that may develop. However, these works should be continued with other activities that may remove the water to a safer portion of the slope section,

5.6.2 Remedial measures for Slope Section two (SS2)

In this critical slope section Ethiopian Roads Authority previously used gabions along the road and retaining wall on toe of Menber Tseay School, but the retaining wall displaced and gabions were deformed. Slope failures in this section occur during, or soon after, heavy rain; therefore this vulnerable slope should always be inspected after heavy rain. A weight at the top of a bank tends to make it more unstable, while a weight at the bottom makes it more difficult for the toe to lift and increases stability. However, if the slope does slip, then structures sitting on the toe can be damaged. If the slip is deep seated (i.e. the radius of the slip circle is large) even building some distance from the case of the slope can be effected. This condition was shown in Menber Tseay School near to the main road.

Tension cracks can form at the top of the slip circle in soils that have some cohesion. The appearance of such cracks (which can run parallel to the top of the bank or form the arc of a circle) is often the first indication of slope instability. Any cracks like these should be closely monitored and appropriate action (e.g. relocating vulnerable footpaths) may be taken. The potential instability that the presence of these cracks represents will be exacerbated if the crack deepens as a result of erosion action. For this, based on the existing and anticipated problem following recommendations are proposed;

- ✓ Surface and subsurface drainage are paramount in the stabilization of slopes characterized by residual soil. Storm water from direct rainfalls and discharges originating outside the deposit should be trapped before they infiltrate the soil profile. A combination of horizontal and vertical drains, located along the toe of the slope, is an effective way of controlling infiltration

- ✓ Planting appropriate vegetation.
- ✓ Reducing the weight at the top by replacing heavy soil with lighter material.
- ✓ Decreasing the weight on the toe of slope that could damage constructed structures.
- ✓ Keeping people and animals away from areas that are vulnerable to landslide.
- ✓ Soil stabilization, e.g. with cement or, sometimes, a little clay to act as a binder.
- ✓ Using geotextiles and ground anchorage systems.
- ✓ Reducing the height of the bank.
- ✓ Reducing the angle of the slope.
- ✓ Repairing tension cracks at the top of the slope.

This critical slope section further needs a detailed investigation such as; geophysical and borehole logging to know the depth of soil, geotechnical studies to know engineering properties of soil and hydro geological condition of the soil. All these data would be essential to define slope geometry, to know behavioral material characteristics and to assess further subsurface conditions for a more accurate stability condition assessment.

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CHAPTER SIX CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

The roads, which passes on the hilly and mountainous terrains is characterized by variable topographical, geological, hydrological and land-use condition. All such conditions make such roads to frequently affect by the slope failures. This is because the road which crosses through hilly and mountainous terrains experiences both deep and shallow excavations at the construction stage that disturbs the inherent nature of rock and soil slopes.

Slope instability along Komobolcha and Desse road is becoming serious problem due to the presence of loose unconsolidated materials (colluvial materials), highly weathered and fractured basalt rocks, high relief, steep natural slopes, nature of geologic formations exposed along the road section, poor drainage conditions, occurrence of high seasonal rains, and seismically active nature of the region. For these reasons, present study was conceived and detailed slope stability analysis of selected critical slope sections was made. The main objective of the present study was to conduct detailed slope stability analysis of critical slope sections and to evolve and suggest appropriate remedial measures to stabilize the critical slope sections.

To meet out the objectives of the present research study, various activities were accomplished. These include; literature Review, collection of primary and secondary data, analysis of data by different graphical, empirical and analytical approaches through standard software, interpretation of results and finally thesis compilation. From thorough literature review a conceptual framework was developed which later helped in developing systematic methodology for the present study.

For the present study two critical slope sections were selected for further analysis. The selection of these critical slope sections was made based on the field manifestation of instability. In this study the slope stability analyses was performed to evaluate the factors of safety for critical slope sections under various anticipated conditions (i.e., static and seismically loaded with varied water saturations). Both Slope/W and Slide software was used under different conditions to evaluate slope stability. Analysis was made by utilizing Bishop, and Morgenstern- Price methods. Factor of safety for the selected slope sections was determined for the different anticipated conditions (i.e., static and dynamic with varied water saturations) using Slope/W and Slide software. Both static and seismic slope stability analysis was carried out and factor of safety was deduced for each anticipated conditions.

From the slope stability analysis it was observed that both the critical slope sections would be unstable for the existing and anticipated conditions. However, for static dry conditions slope section SS1 is critically stable and SS2 section is stable. For all other anticipated conditions defined by static and dynamic situations with varied water saturation both slope sections would be unstable. These findings validates to the visual observations made for various instability manifestations on both slope sections during the field work.

The results obtained from Slope/W and Slide software shows that for all conditions almost similar type of results (± 0.05 variation only.) were computed. This confirms that the input data used and the results obtained for stability condition are correct.

The causes of slope instability in the study area are wide and multi-dimensional, therefore integrated approach of remedial measures may be more appropriate to mitigate the possible landslide hazard in the area. Depending on site condition and slope stability analysis result of the study area four types of suitable preventive and remedial measures are recommended namely; proper managements of drainages, retaining structures, gabions, and managing steeply cut slopes.

6.2 Recommendation

Slope instability along Kombolcha to Desse road are triggered by natural factors including the presence of unconsolidated deposits (colluvial material, and weathered basalt), high relief and steep natural slopes, nature of geologic formations exposed along the road section, poor drainage conditions, occurrence of seasonal rains, and seismically active nature of the region. These factors suggest that slope instability in the area, particularly the large destructive colluvial material, will continue to occur and pose hazard to the highway despite the remedial measures.

The main aim of this study was to conducted slope stability analysis and to suggest remedial measures to control and minimize the potential hazard associated with slope instability. Thus, it was intended to suggest the most suitable remedial measures, based on analysis and site conditions. Although the hazards associated with slope failure cannot be completely eliminated, they can be significantly minimized by employing a variety of protective and remedial measures. In addition to remedial measures, continual maintenance of the highway is essential for smooth and safe flow of traffic.

The general recommendations based on the present study are;

- Although there are numerous instances of slope instability along the highway, many steep slopes have reached their natural state of equilibrium. Excavating the toes of these slopes for the purpose of widening the highway can result in instability, triggering new failure. This will be particularly true for the areas where unconsolidated materials are prevalent. Therefore, the best approach for widening the road would be to consider fill embankments supported by retaining structures before undertaking any excavations to gain the required width of the highway.

- From the present study it was deduced that the poor drainage condition on critical slopes have played a significant role in inducing instability of slopes therefore, improvement in drainage condition will be an effective measure to stabilize the slopes in the area.

- The magnitude of the slope instability problems along the road is so massive that in certain instances, the hazard has to be accepted and to be learnt as how to best live with the problem.

- Before implementing any remedial measure suggested through present study more detailed studies would be required to work out specific cost effective remedial measures for individual critical slopes.

- In order to evaluate better influence of critical factors for slope instability, further detailed geotechnical investigations, geophysical and topographic survey should be performed.

- The conclusions derived from this work should serve as a basis for further research, extending issues already dealt with in previous studies and contributing new information on slope instability in the study area.

Finally, in the present study maximum efforts were made to conduct the detailed slope stability analysis on the selected slope section in a systematic manner well supported by actual scientific data and realistic field observations. Such works require enough financial support, time and resources; this research work was conducted under such constraints. All these constraints might have affected the quality of the results. This study was limited to two critical slope sections only.

Further studies on slopes will provide additional knowledge on mechanisms of mass movement processes involved in slopes of the study area. Such, additional elaborate studies may be vital for landslide hazard management specific to critical slopes and in general for the entire area.

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APPENDIXES

APPENDIX A

LABORATORY TEST SUMMARY RESULTS

Company Name:		Document No.:
TCDSCo	TRANSPORT CONSTRUCTION DESIGN SHARE COMPANY	
Revision 0	Material Testing Result	OP/TCD/093 Page No.:

20/05/13

Code: 1404/2005 — Your Ref.=========
PROJECT: Slop Stablity analysis along Kombolcha - Desse Road
SUBMITTED BY : Biruk Wolde
SAMPLE OF: Soil
STATION: S1 CF
DATE SAMPLED: ============
TEST REQUESTED: Direct Shear Strength (AASHTO T-236)
REPORTED TO: Biruk Wolde — Date ON: 20/05/13

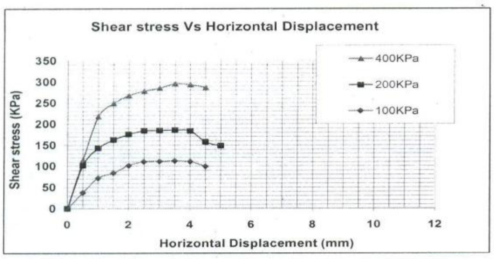
1. SPECIMEN DATA

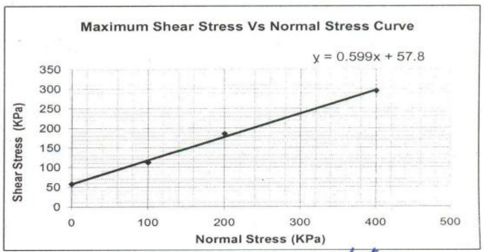
TP NO.	_
Depth (m)	_
Initial Height (mm)	20.00
Initial Area (cm^2)	36.00
Initial Weight (gm)	142.56
OMC (%)	18

Sample No.	_
Sample Condition	REMOLDED
Specimen Size (mm)	60 × 60 × 20
Initial Volume (cm^3)	72.00
Bulk Unit Weight (gm/cm^3)	1.98
Dry Unit Weight (gm/cm3)	1.68

2. SHEAR RESULT

Normal Stress (KPa)	100	200	400	C(KPa)	Ø(Degrees)
Shear Stress (KPa)	112	186	295	58	31





Remark:
Conducted/Reported by: Lab.Engineer/Technician — Checked by: Head,Laboratory division — Approved by Manager,Geotec Eng.Dept.

AMONG THE MAJOR SERCISES RENDERED BY THE DESIGN PROJECTS OF TCDSCo. ARE,
- Testing the engineering properties of various Construction materials, Such as soil, Aggregates, Asphalts, Cements, Water reinforcement steel bars, etc.

Tel.No. 155799 — P.O.Box 41726 — Fax:- 251-1-514231

The Federal Democratic Republic of Ethiopia Transport Construction Design S.Co.

	Company Name:	Document No.:
TCDSCo	**TRANSPORT CONSTRUCTION DESIGN SHARE COMPANY**	
Revision	**Material Testing Result**	OP/TCD/093
0		Page No.:

20/05/13

Code: 1404/2005 **Your Ref.:** =========
PROJECT: Slop Stablity analysis along Kombolcha - Desse Road
SUBMITTED BY : Biruk Wolde
SAMPLE OF: Soil
STATION: S3 CF
DATE SAMPLED: ===========
TEST REQUESTED: Direct Shear Strength (AASHTO T-236)
REPORTED TO: Biruk Wolde **Date ON:** 20/05/13

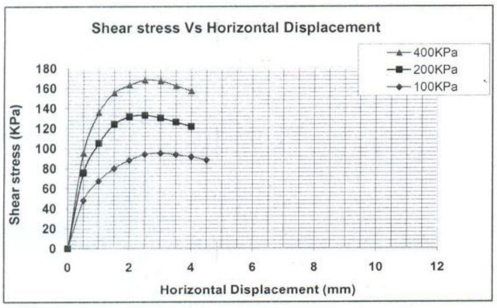
1. SPECIMEN DATA

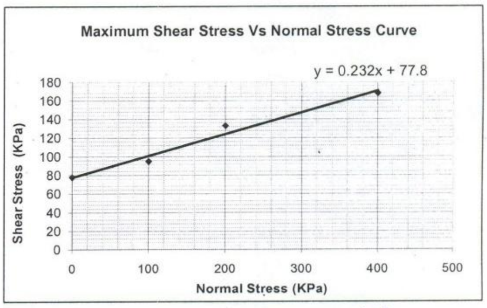
TP NO.	_
Depth (m)	_
Initial Height (mm)	20.00
Initial Area (cm^2)	36.00
Initial Weight (gm)	124.56
OMC (%)	27

Sample No.	_
Sample Condition	REMOLDED
Specimen Size (mm)	60 × 60 × 20
Initial Volume (cm^3)	72.00
Bulk Unit Weight (gm/cm^3)	1.73
Dry Unit Weight (gm/cm3)	1.36

2. SHEAR RESULT

Normal Stress (KPa)	100	200	400	C(KPa)	Ø(Degrees)
Shear Stress (KPa)	95	133	168	78	13





Remark: ______
Conducted/Reported by: ______ **Checked by:** ______ **Approved by** ______
Lab.Engineer/Technician Head,Laboratory division Manager,Geotec.Eng.Dept.

AMONG THE MAJOR SERCISES RENDERED BY THE DESIGN PROJECTS OF TCDSCo. ARE,
- Testing the engineering properties of various Construction materials, Such as soil, Aggregates, Asphalts, Cements, Water reinforcement steel bars, etc.

Tel.No. 155799 **P.O.Box 41726** **Fax:- 251-1-514231**

The Federal Democratic Republic of Ethiopia Transport Construction Design S.Co.

APPENDIX B

Desse area Monthly precipitation in mm (2000-2012)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
2000	0	0	13.5	87.7	58.3	18	352.6	302	108.4	77.5	60.9	172.2	1233.1
2001	21.7	0	200.5	79.8	53.1	53.7	410.2	308.8	231.9	63.2	2.4	5.6	1483.8
2002	3.2	9.4	50	13.6	8.4	56	342	354.3	102.3	16.5	4.6	102.3	1232.5
2003	78.3	7.8	56	45.7	76	78.9	326.7	372.4	121.3	43	9.2	52.2	1402.3
2004	0	41.6	45.8	164.2	54.4	67	371.1	290.3	114	32.1	0	73.1	1273.7
2005	136.5	44.3	77.6	148.1	39.5	64	362.5	376.6	89.4	31.5	2.1	44.4	1243.6
2006	121.4	70.4	86.3	175.5	75.6	79	301.3	270.2	97	24.6	6.8	0	1198.1
2007	6.3	0	201	152.3	97.2	90.3	199	313.6	47.8	30.8	30.7	0	1245.9
2008	57.4	60.8	58.2	138.6	134.5	28.3	328.8	296	75.6	48.9	3	32.1	1251
2009	67.3	45.2	0	106.9	23.8	56.9	341	281.4	123	32.7	82.1	0	1345.7
2010	2.6	0	6.7	127.3	39	87.7	299.3	283.1	111.3	67.5	47.2	0	1163.8
2011	0	3.8	35	121.2	44.6	67.9	432.1	352.4	162	41.3	23.7	32.1	1342.3
2012	4.5	0	48.9	120.8	34.7	71.4	321.4	391.2	90.8	49.5	47.2	3.5	1231.7

APPENDIX C

SLOPE MODEL AND FOS RESULT FOR BOTH SLOPE/W AND SLIDE SOFTWARE

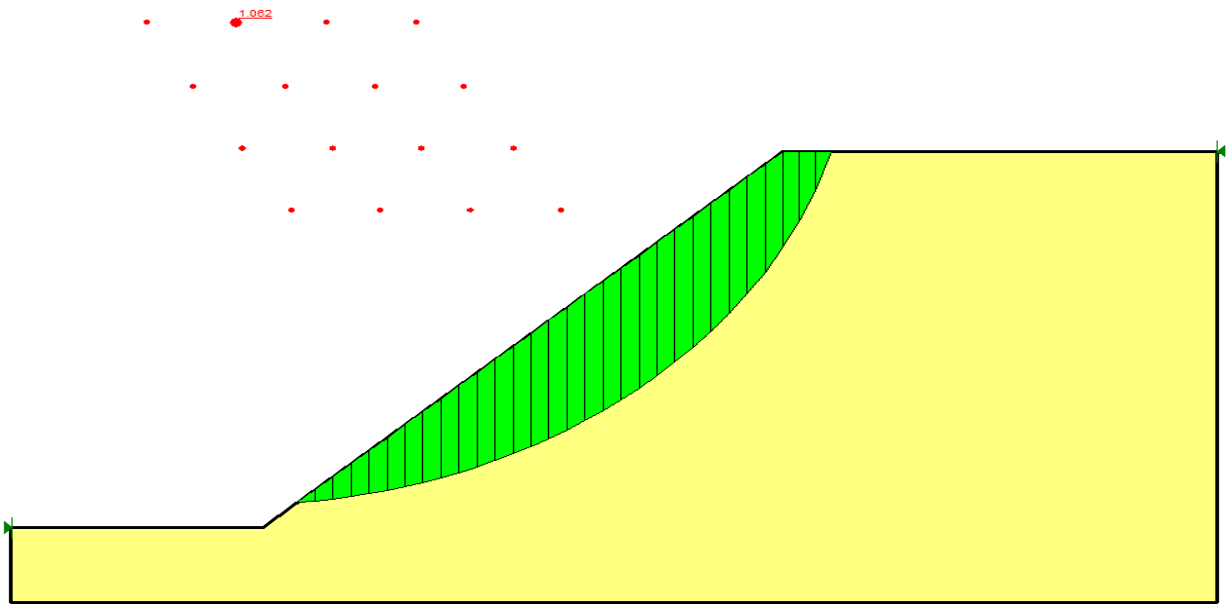


Fig1. Slope/W Slope model static dry condition SS1

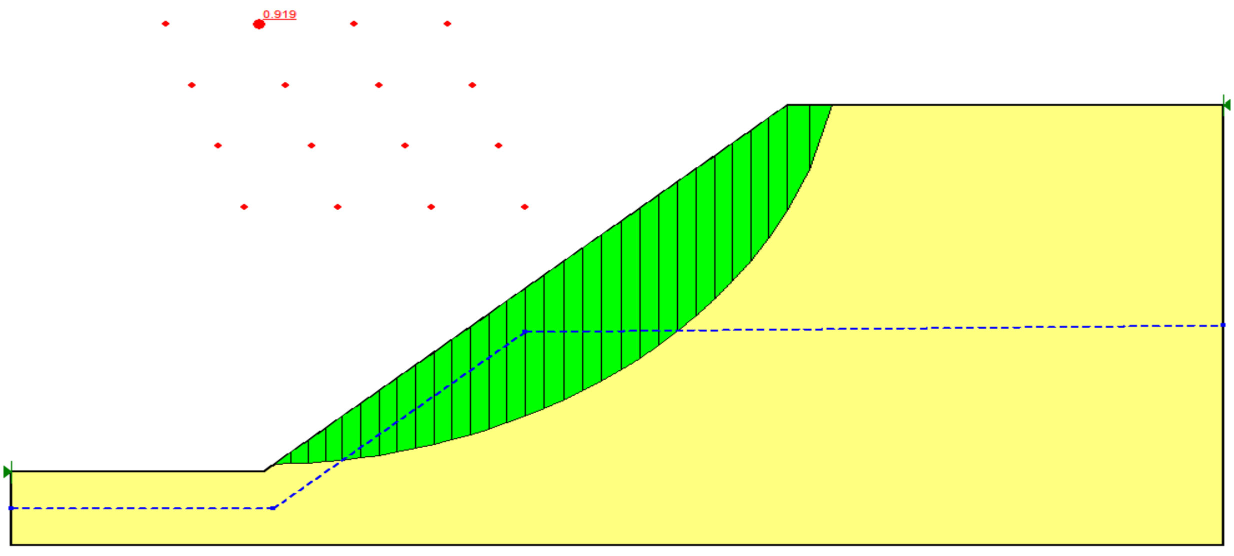


Fig2.Slope/W Slope model static present condition SS1

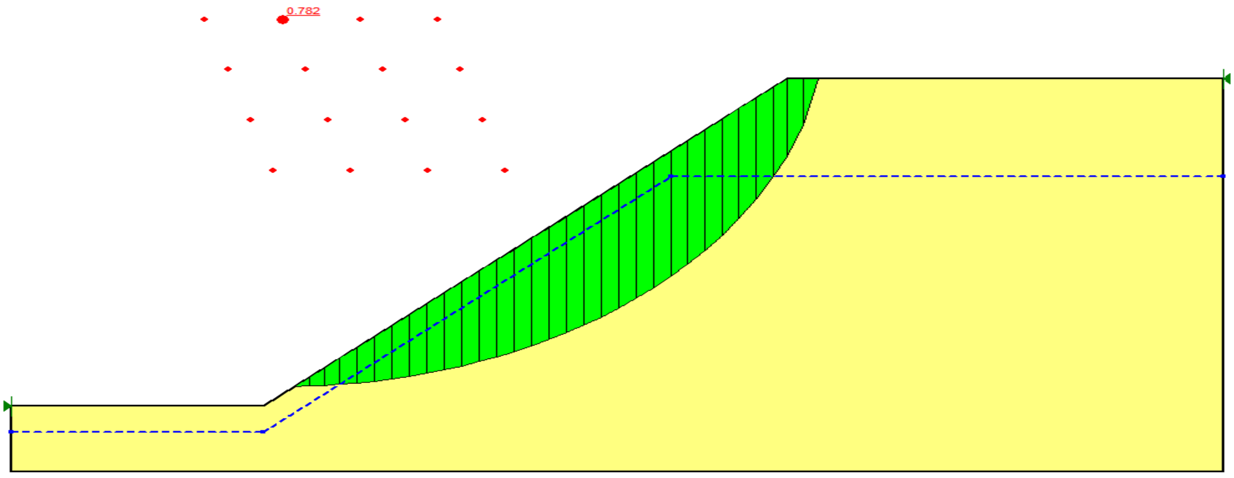


Fig3.Slope/W Slope model static worst condition SS1

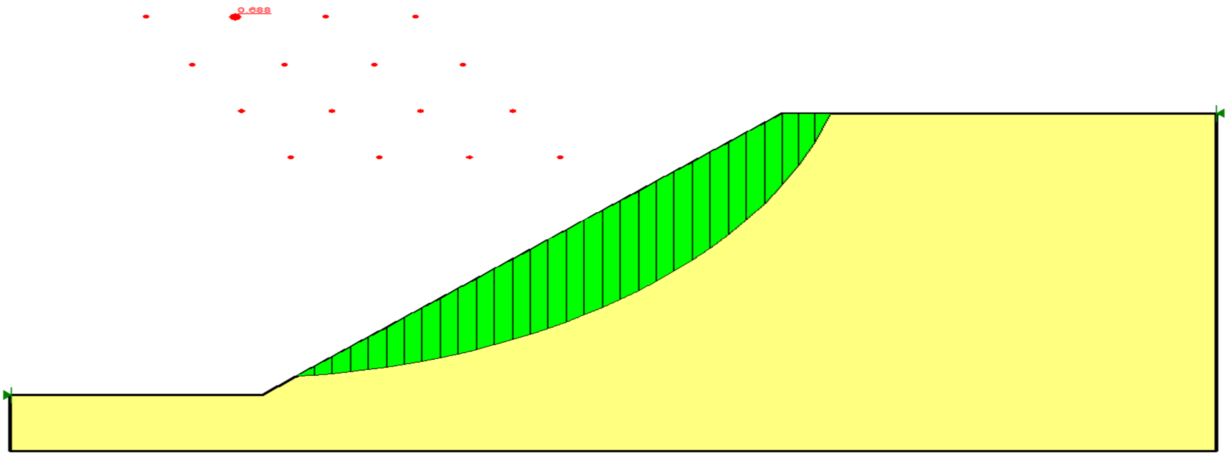
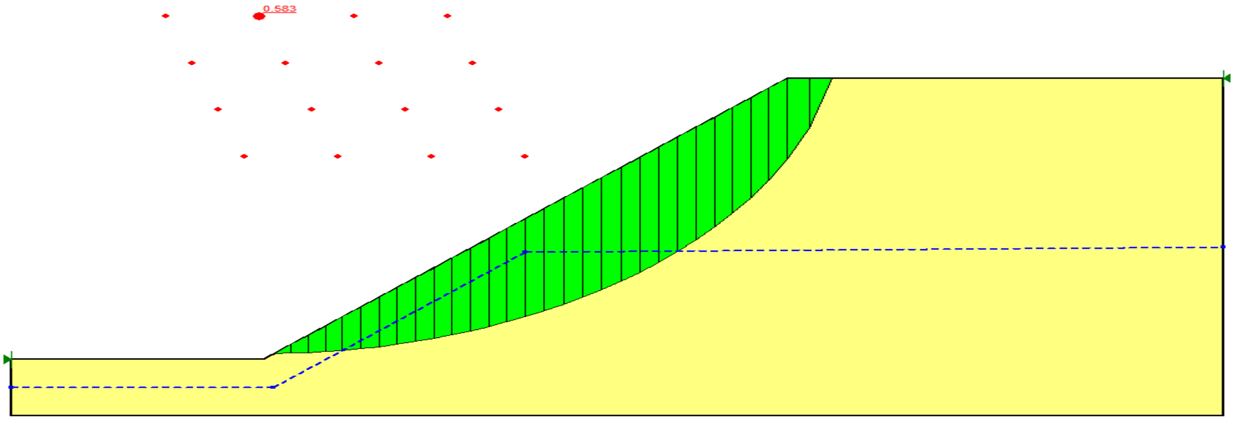


Fig4. Slope/W Slope model dynamic dry condition SS1



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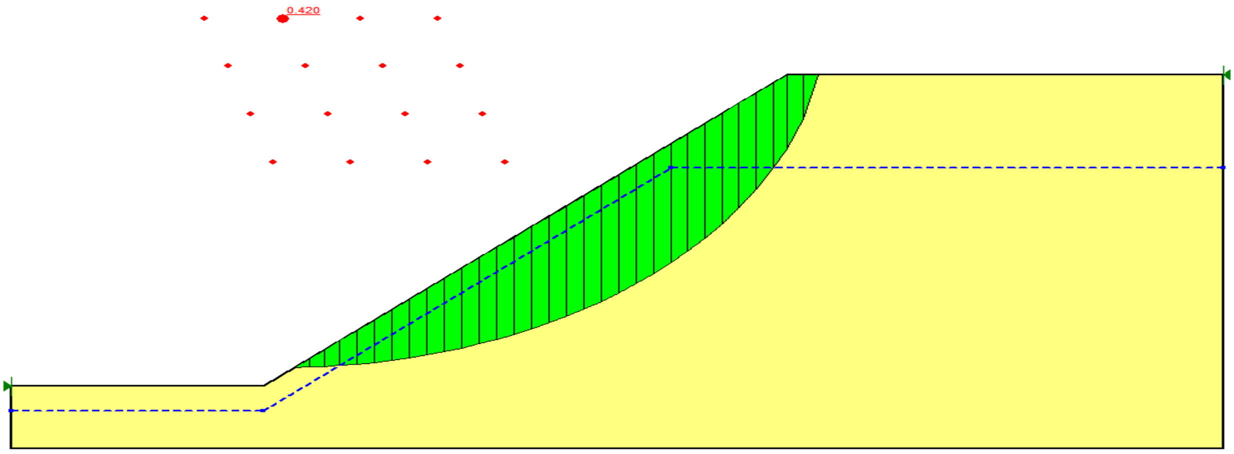


Fig6. Slope/W Slope model dynamic worst condition SS1

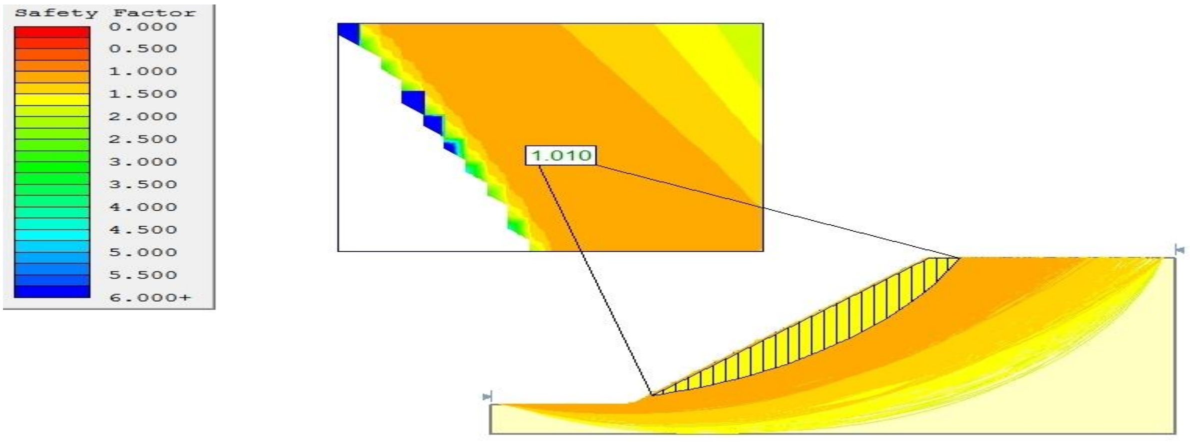
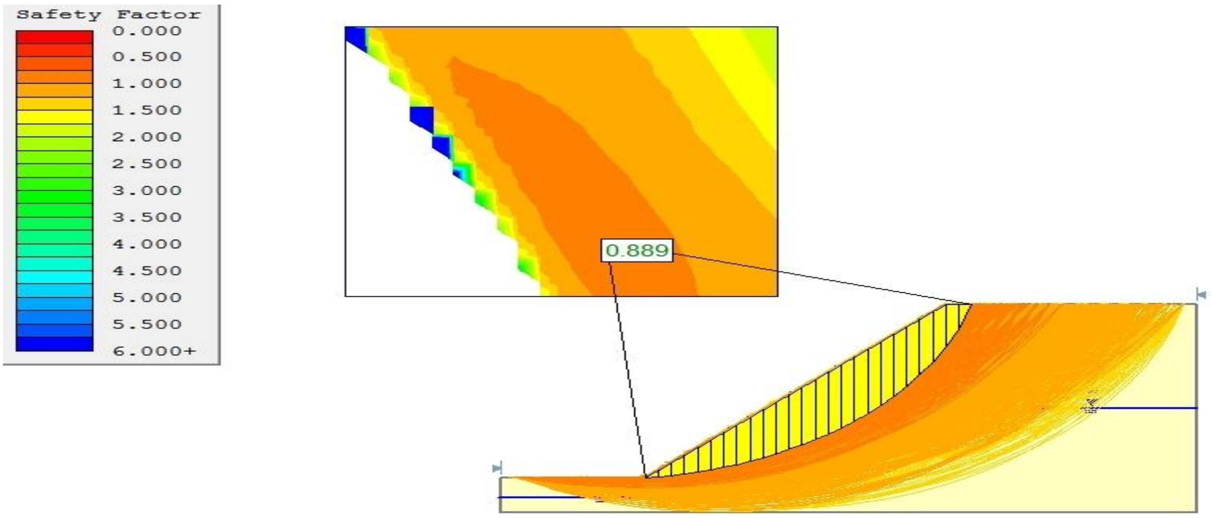


Fig7. Slide Slope model static dry condition SS1



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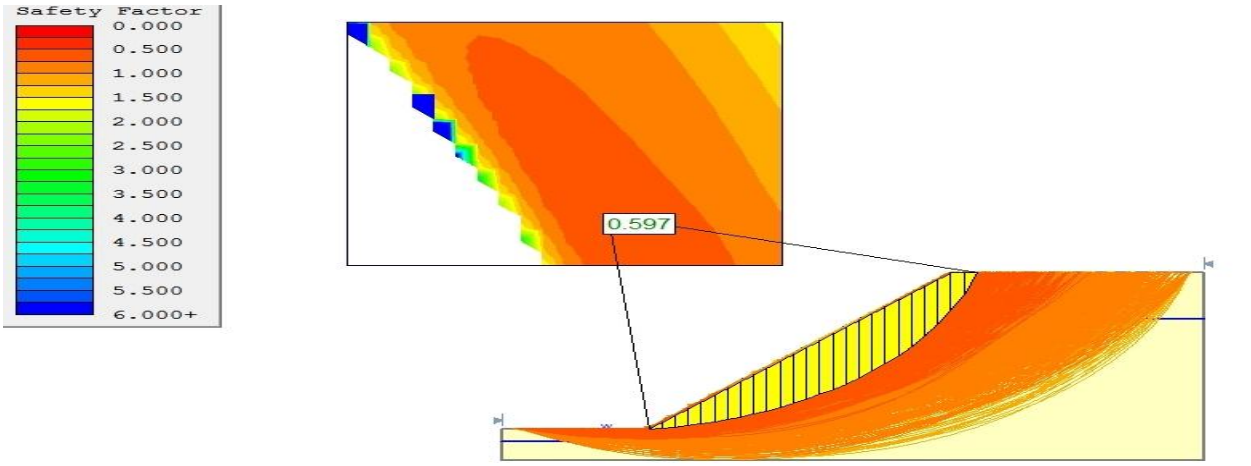
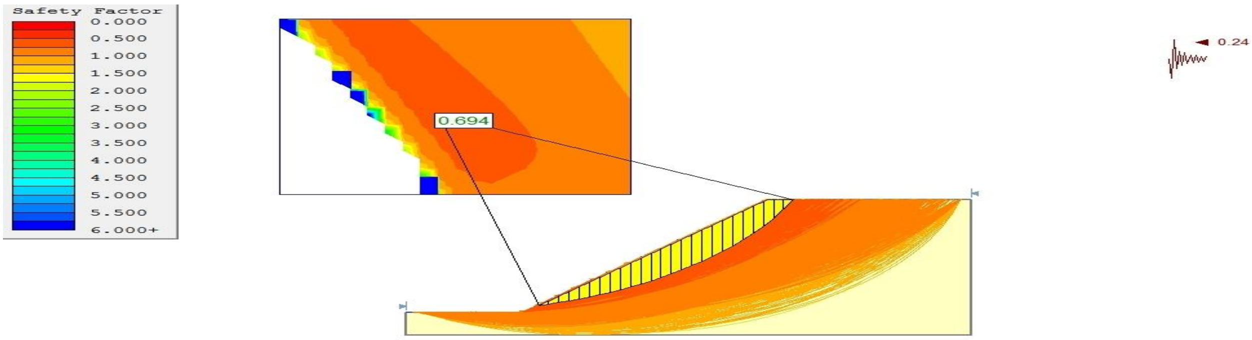


Fig9. Slide Slope model static worst condition SS1



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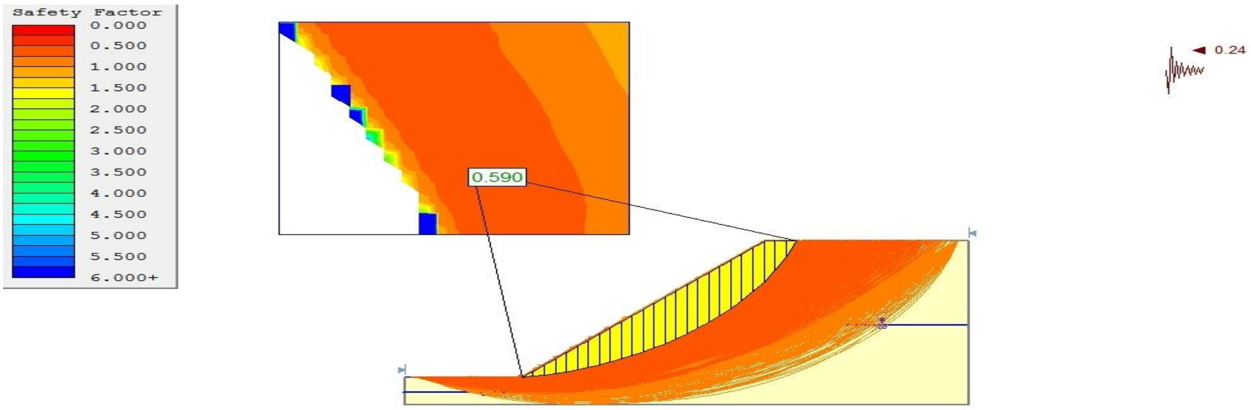

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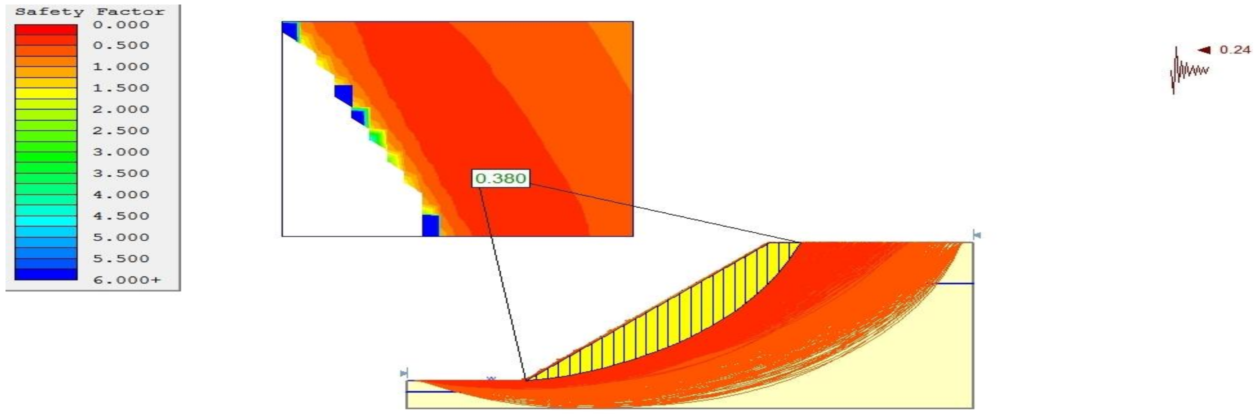

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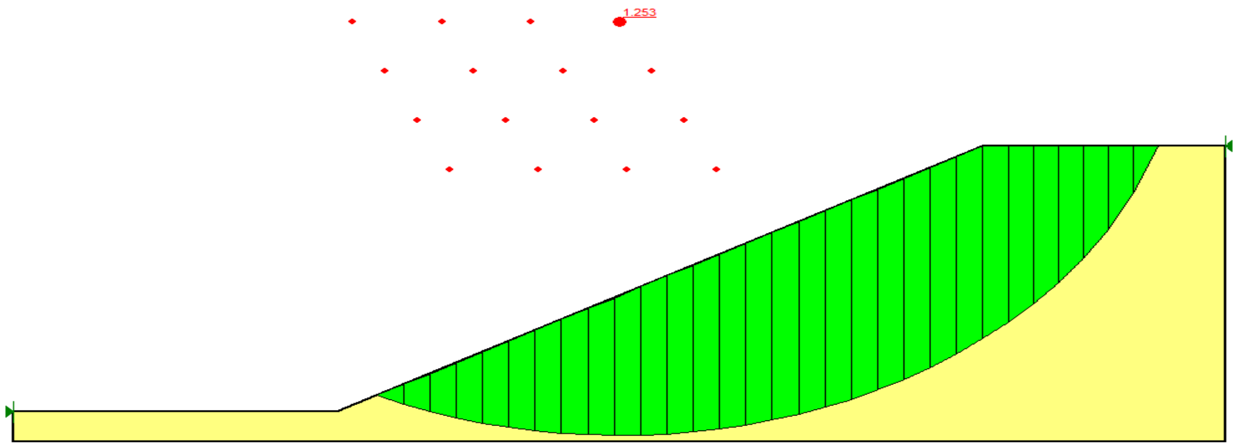
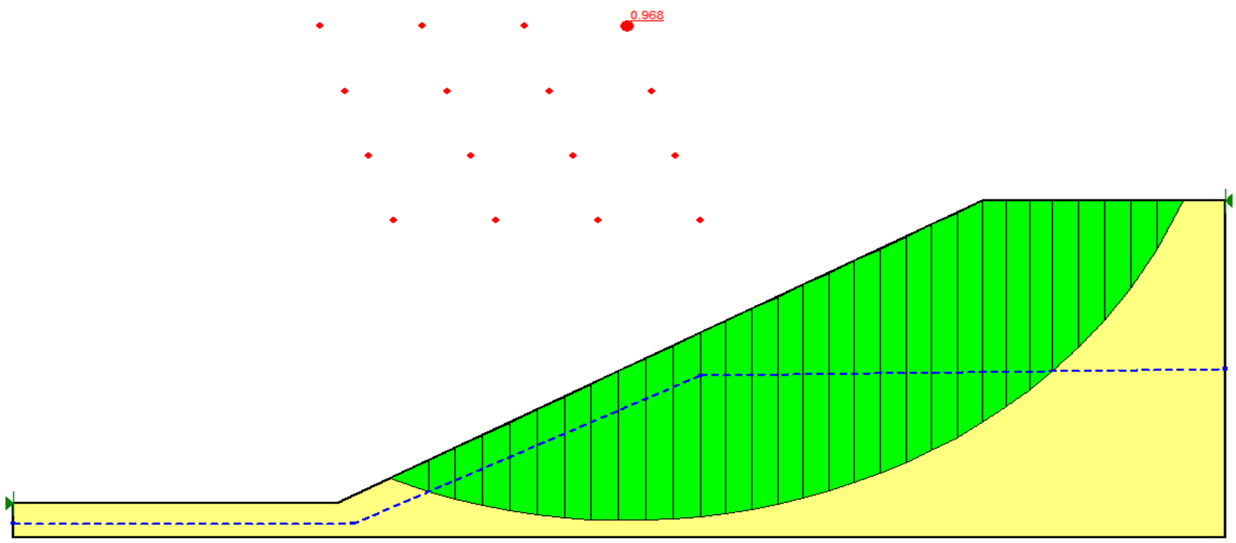


Fig13. Slope/W Slope model static dry condition SS2



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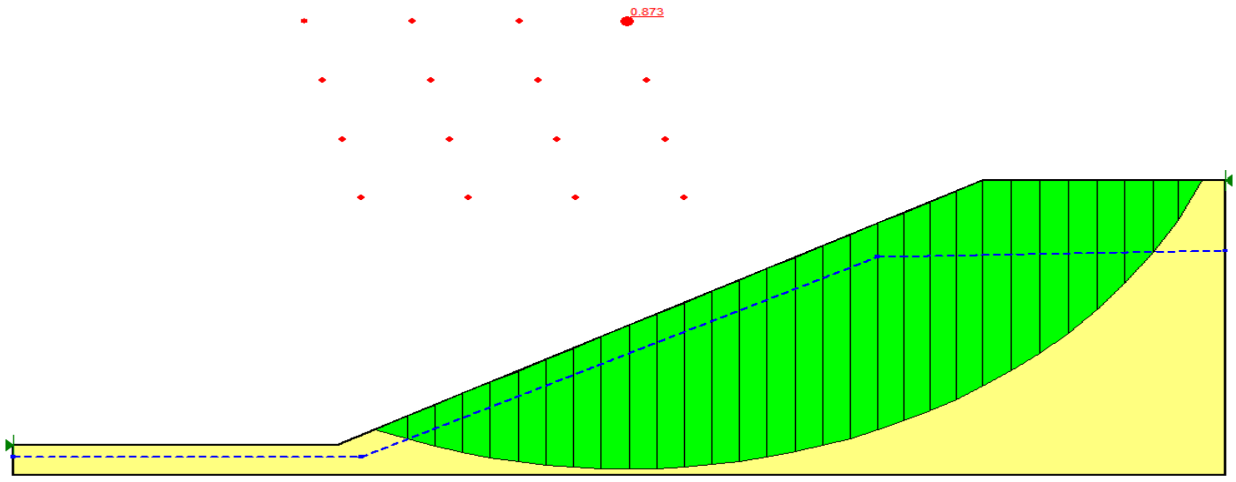

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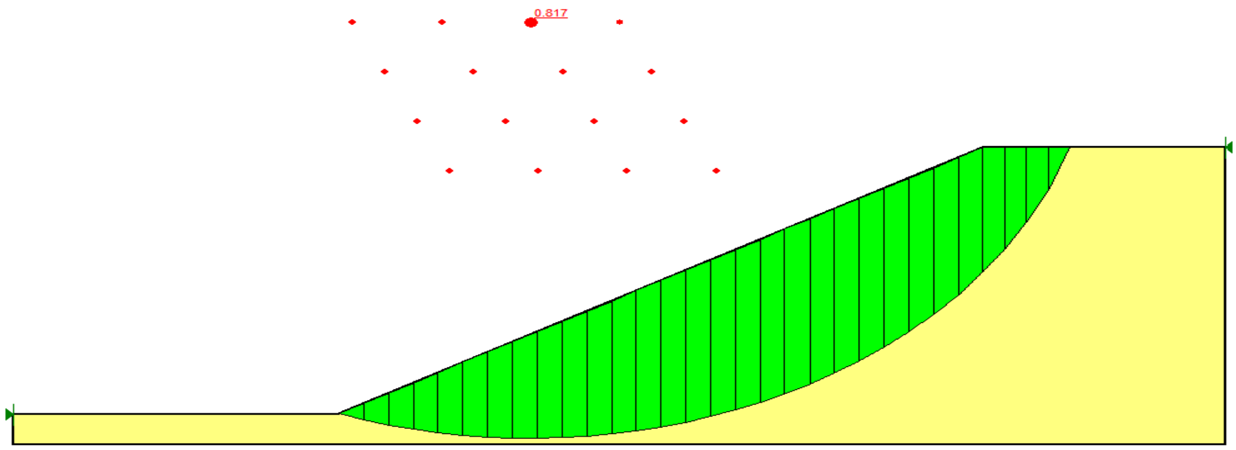

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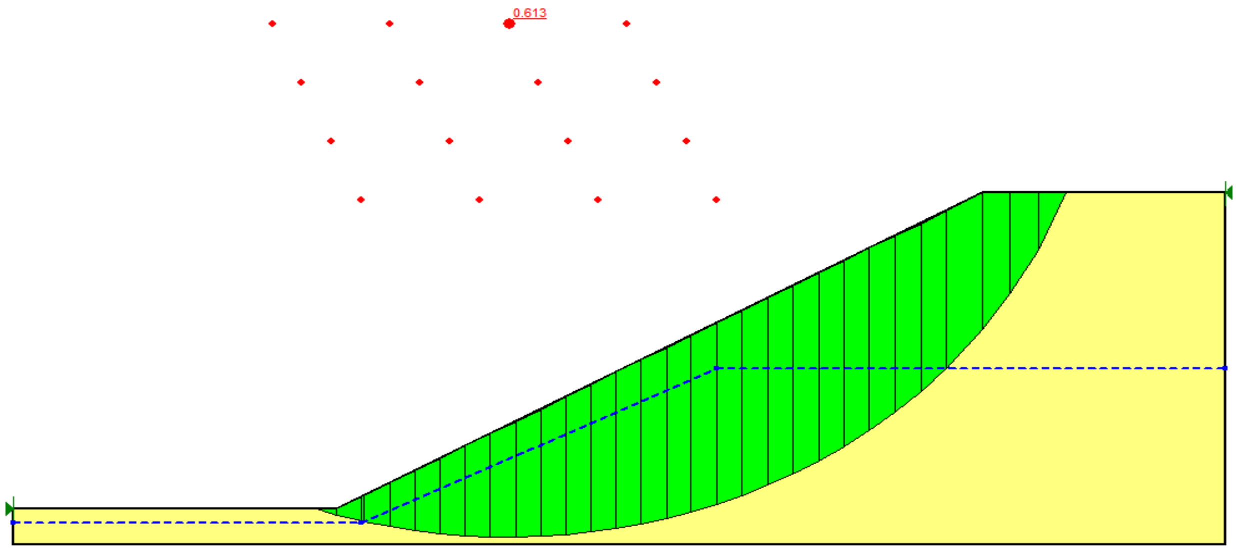

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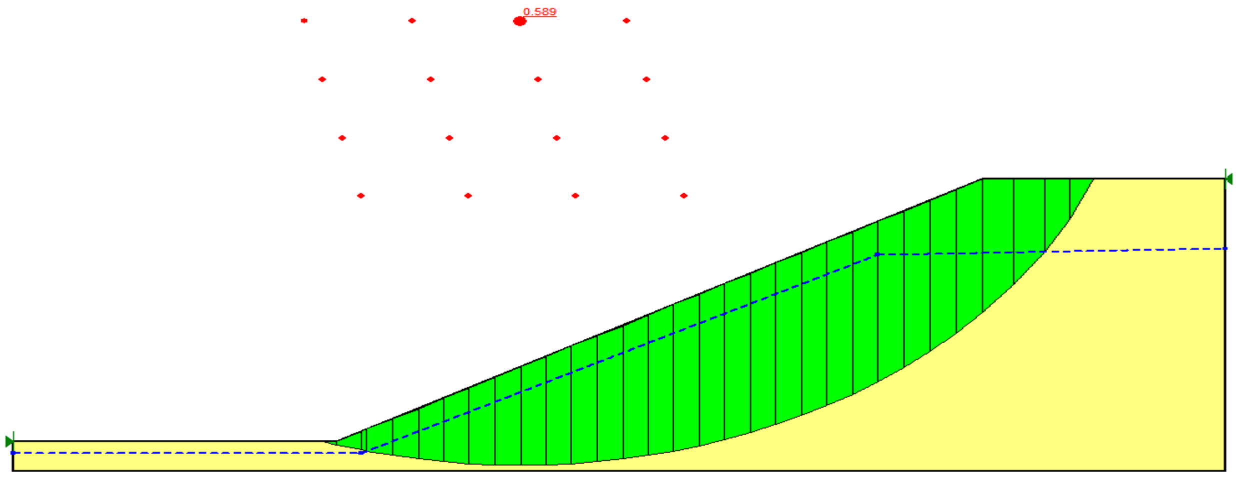

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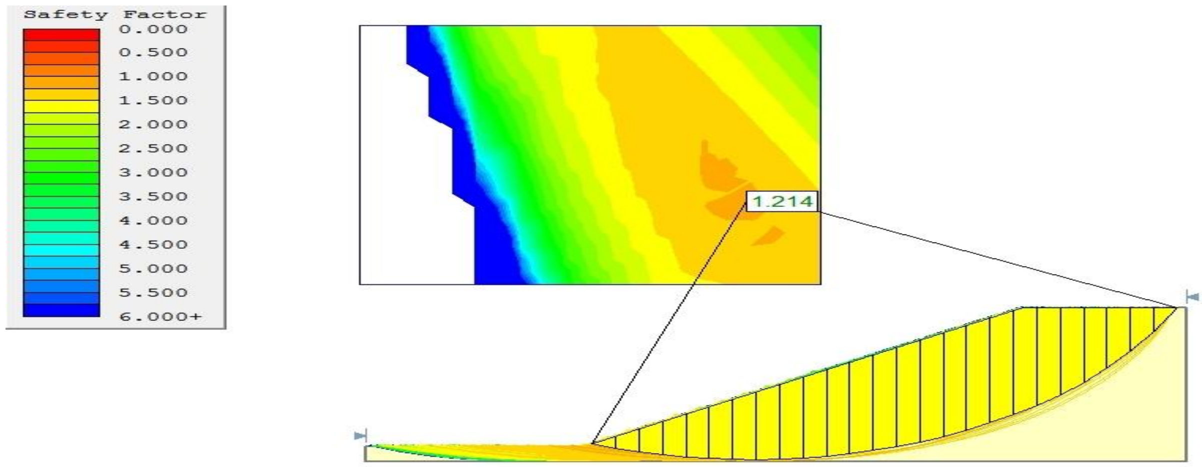

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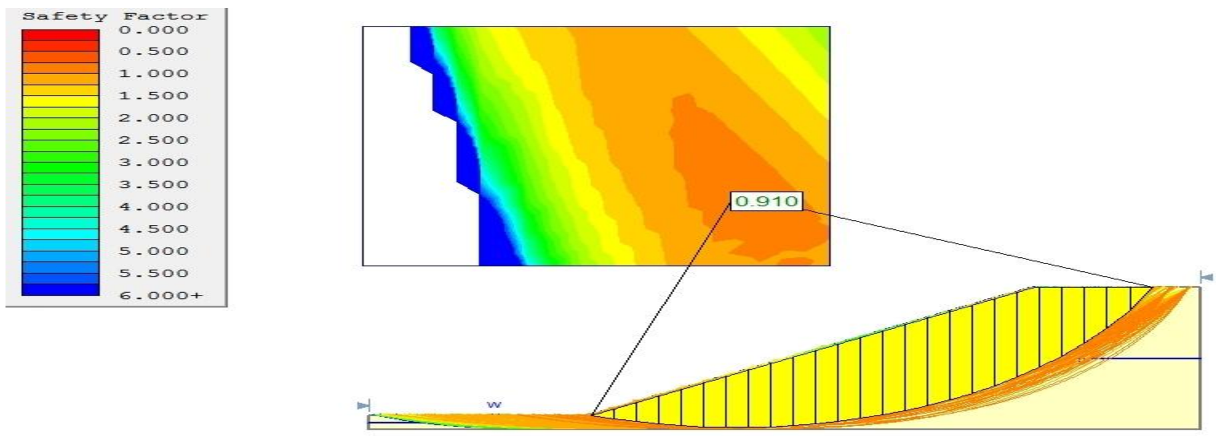

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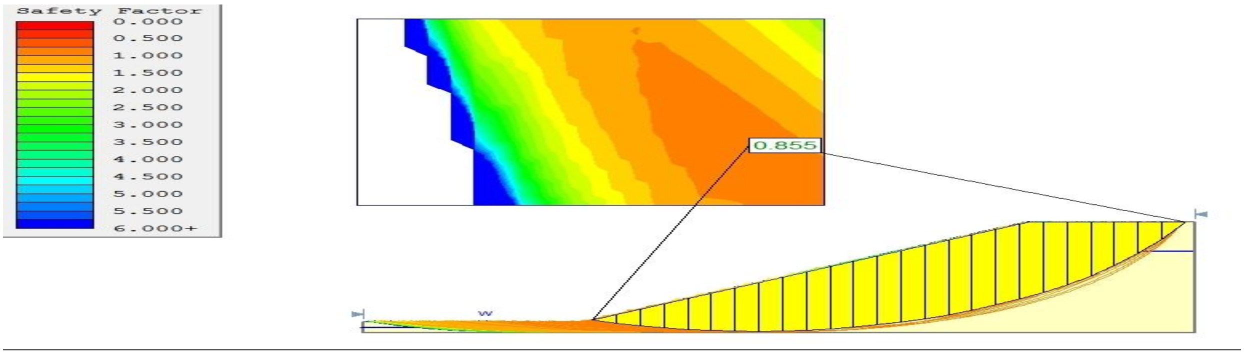

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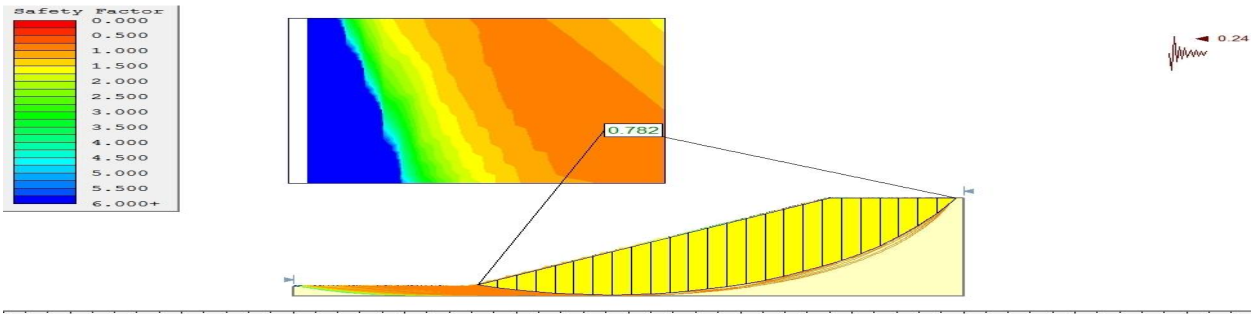

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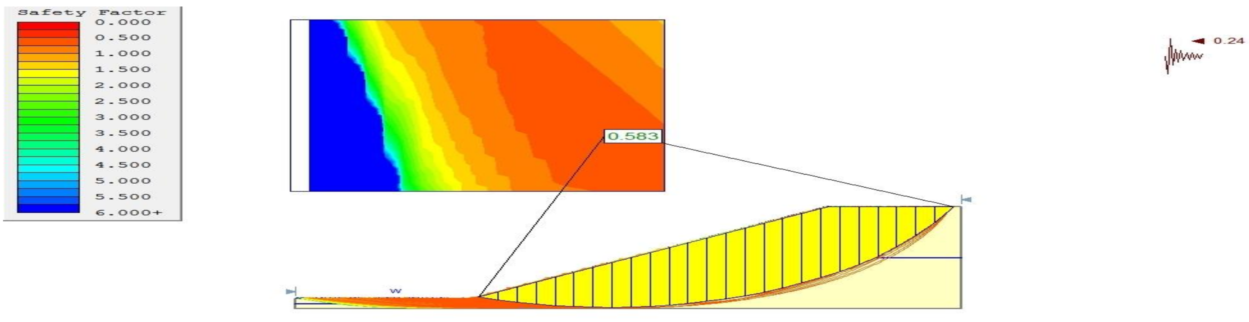

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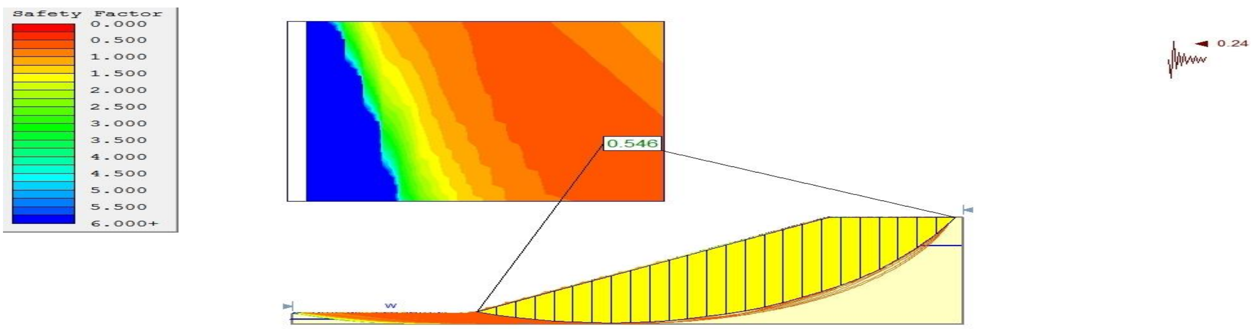

Fig24. Slide Slope model dynamic worst condition SS2