JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL ENGINEERING STREAM

Slope Stability Analysis and Mitigation Measures of the Agoro – Shahgubi Section of the Agulae - Berhale Road, Northern Ethiopia.

A Final thesis submitted to the School of Graduate Studies of JimmaUniversity in partial fulfillment of the requirements for the Degree of Masterof Science in Civil Engineering (Geotechnical Engineering)

By: Mengesha shiferaw Berhie

December, 2017

Jimma, Ethiopia

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By: Mengesha Shiferaw Berhie

Main -Advisor: Kifle Woldearegay (Ph.D.)

Co-Advisor: Damtew Tsige (Ph.D. student)

December, 2017

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Declaration

This thesis is my original work and has not been presented for degree in any other university

Mengesha Shiferaw Berhie__________ ________/_____/_____
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This thesis has been submitted for examination with my approval with university supervisors

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Acknowledgement

First of all I would like to extend my sincere and spiritual thank for God for his priceless and endless mercy. I wish to express my deep gratitude to my Advisor Kifle Woldearegay (Ph.D.) for his extensive, in valuable and steadfast guidance, for his support and patience, sharing his knowledge, experience and for his skillful and passionate lecture that inspired me greatly throughout my MSc studies. Having a chance to study under his supervision was one of the most fortunate things in my life.

Likewise, I thanks Mr. Damtew Tsige for his expert advice and feedback my co-advisor throughout the time of my studies, I benefited from the co-operative and friendly support, advice and guidance of many others. I would like to acknowledge the Mekelle University, Civil Engineering Department Laboratory Assistants Mr. Zenawi and Mr. Yonas for their patient discussions with me on my laboratory works

I would like also to thank my gentle class mates during my studies Mr. Hayelom Tsegay and Mr. Diriba Maru for making a vibrant research environments with pleasant casual chats as well as helpful discussion.

I am grateful to my friends, who gave me indispensable support through their strong moral support and advice to Jimma Institute of Technology staffs: MogesGetahun and YibasMamuye. Most importantly, I would like to thank to all my families for their love, support, and encouragement. I specially thank my wife Tigist Abraham. No words can express how grateful I am for her endless love and support on me.

Abstract

The general objective of this research is to evaluate the slope stability and erosion problems along Agoro-Shahgubi section of Agulae-Berahle route and recommend mitigation measures. Disturbed soil samples and dimensions of the failed slopes (length, width and slope height) were gathered from nine failed slopes spread through Agoro-shahgubi.

Direct shear tests were performed to determine the shear strength of the soil samples. Having the results of unit weight and direct shear tests the stability of the slope was checked. The slope stability analysis method using SLOPE/W software has been used to determine the factor of safety of the slope.

The results of the limit equilibrium methods shows that factor of safety is almost less than unity for almost all except for the Bishop method for the failed slopes for both sample codes ***A-1-01*** *and* ***SG(*** *this is why there are large boulders with the soil which acts as a supportive structure can may fail during rainy season). So that the numerical results of the Limit equilibrium method verify instability of the slope.*

The major causes of slope instability in the study area are method of excavation, over steepening of the slope, deforestation and surface water erosion which includes the effect of saturation on strength. Based on the findings the following mitigation measures had been recommended which includes Removing of unstable and over hanging boulders, cut and fills solution (slope flattering and incorporating of benches in the slope and providing support (construction of retaining walls have been considered as a remedial solution.

Key Words: Agoro-Shahgubi, Erosion, Mitigation Measures, Stability Analysis

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List of Symbols

B	Width of a slice
C	Cohesion
E	The horizontal interslice normal forces;
FF	Safety factor obtained by equilibrium of forces;
FM	Safety factor obtained by equilibrium of moments;
F	The perpendicular offset of the normal force (N) from the center of rotation or from the center of moments;
FOS	Factor of safety
F(x),	Scalar interslice force function;
H	The height of each interslice surface;
H	The height of each slice from the center of the base;
h_t	Height of line of thrust of interslice force from the base of eachslice
LEM	Limit equilibrium method;
ΔL	The length of the base of each slice;
N	The total normal force on the base of the slice;
CSS	Circular Slip surface
T	The vertical interslice shears forces;
U	The excess pore pressure acting at the base of each slice;
α	The angle between the tangents to the center of the base of each slice and the horizontal;
δ	Inclination angle of resultant interslice force;
λ	A scaling factor;
γ	Unit weight of soil
σ_n	Total normal stress acting at the base of each a slice

List of Acronomy

A	Agoro
K	Korha
AF	AdiFeham
AS	Adigrat sand stone
SG	Shahgubi
AASHTO	American Association of State Highway and Transport Officials
ASTM	American society for testing materials
USCS	Unified Soil Classifications standards
UNESCO	United Nation, Educational, Science and Cultural Organization
UTM	Universal Transverse Mercator Grid
ERA	Ethiopian road authority
SLOPE/W	One component of a complete suite of geotechnical products called geo-Studio that has been designed and developed to be a general software tool for the stability.

Dedication

This work is dedicated to my late father:-Shiferaw Berhie (Fanno),

Who raised me with love, fun, and care.

Who loved just to see me, to cheer me, make me laugh, see me progress

Who will always be missed and remembered with love.

May GOD bless your soul!!!

CHAPTER ONE

INTRODUCTION

1.1Background

Understanding the importance of the road sector, Ethiopia has been involved in massive road constructions since the last two decades. The country has been undertaking further road developments in order to connect local administration regions (Tabias/Keble) (Woldearegay, 2014).

A well planned and implemented road development has a number of benefits among which is fueling growth process through different activities of the development endeavors of a nation including: creating market access opportunities for agricultural products, access to services (health, schools, etc.) and contribute to the socio-economic development of an area (Wu T.H,200 and Abdel-Latif,2000)**.**

The Agulae - shahgubi road project has been constructed by the contractor; Defense Construction Enterprise and consulting firm under the client of Ethiopian Roads Authority.

This road section includes asphalt concrete standard carriage way, bridges, culverts, retaining walls and pavement marking. It was commenced on 29/Sep/2010 and completed on 25/Dec/2014 and its total length is about 76 km.

The Agoro – shahgubi section of the Agulae – Berhale road is a **27** km long Figure 3.1 and passes through topographically and geologically challenging terrain among which escarpments, adverse geological formations (loose overburden materials), complex structural features and dense to sparse vegetation cover. And hence this route is usually endangered to such geo – hazards such as differential erosions (flood) as well as slope failures. Even if the pavement is not failed entirely, the route is highly affected by geo – hazards like surface water erosions (debris/mud flows, scouring of road sections), (rock fall, rockslide, earth slide etc.). These geo – hazards have caused road section damage, destruction of certain retaining structures and blockage of surface drainage lines.

1.2Statement of the Problem

The Agoro – Shahgubi section of the Agulae - Berhale road connects Tigray and Afar Regional State. It passes through steep slopes and highly dissected topography, adverse geological formations (loose overburden materials), complex structural features and dense to sparse vegetation cover.

The route is highly affected by surface water erosions as well as slope instabilities. It is generally characterized by poor and at times with no construction of retaining structures and inadequate and/or blockage of surface drainage systems like ditches, culverts etc.

It is common to observe debris/earth slides, scouring of road sections, rock fall, and rockslides.Consequently, this route is frequently affected by a number of problems which include (a) traffic delays (reduction of highway capacity) due to road way blockage mainly during rainy season, (b) damage to road section including retaining structures leading to additional maintenance costs. As the result of siltation problems, ditches eroded and culverts are blocked which leads to further erosions and initiate slope failures.

The above problems have been causing both direct and indirect impacts. The direct impacts are cost of reconstruction for the destroyed infrastructure and for resettlement of the displaced communities while the indirect impacts are disruption of economic activities and other social services.This research focuses on the geotechnical problems (slope failures), and possible solutions of the issue along the Agoro-shahgubi section of the Agulae-Berahle road with main focus on erosion and slope stability along the selected route.

1.3Objective

1.3.1 General Objective

The general objective of this research is to evaluate the slope stability and erosion problems along Agoro-Shahgubi section of Agulae-Berahle route and recommend mitigation measures.

1.3.2 Specific Objective

The specific objectives of research work are:

1. To evaluate slope stability conditions and erosional issues along the route.
2. To evaluate the causes and mechanisms of slope failures and the factors that affect erosional process along the selected road section.
3. To undertake slope stability analysis and recommend mitigation measures which could be implemented to address the problems.

1.4Research Questions

The research questions that this study will attempt to clarify are as follows:

1. What are the mechanisms of slope failures, erosion processes and related road damages along the route?
2. What are the stability conditions of the slopes and what factors are causing erosional problems along the route?
3. What are the engineering solutions for the above problems?

1.5Scope of the Research

This particular research has focused on evaluation of the slope stability and erosional issue along the Agoro-Shahgubi section of the Agulae-Berahle route that connects Tigray and Afar regional states in northern Ethiopia. The study involved; (b) field evaluations (measuring size of the damaged section of the road which includes scouring depth, width, and length of the gully as well as dimension of slop failures, laboratory testing and stability analysis. Finally recommendations are provided on the mitigation measures which could be implemented. Despite limited financial resources for the research all efforts were done to generate quality data.

1.6 Significance of the Research

The result of this research could be used for farther maintenance of the Agoro-Shahgubi road section as well as for farther infrastructure developments in similar environments. The output of this research could be used by research institutions, government organizations, private sector and other interested body on slope stability and erosion hazard assessments and remediation works.

1.7 Structure of the Thesis

This thesis has five chapters and their contents are outlined below:-Chapter one is an introductory chapter that gives an overview of the background of the research and the study area. Chapter Two deals with the literature review about the occurrences, influencing factors and mitigation measures of landslide. Chapter Three deals with the materials and methods. Chapter Four results and discussions. Chapter Five presents the conclusions and recommendations drawn from the study.

CHAPTER TWO

LITERATURE REVIEW

This chapter gives a review of previous literatures on definition, causes, failure mechanisms, and erosion and mitigation options of landslides.

2.1 Concepts of slope stability

Slopes can be man-made or natural. The study and quantification of their safety has been recognized as very essential for the economical prevention of life and property loss. Civil engineers and in particular Geotechnical Engineers have devoted much effort and study to the understanding of the mechanisms leading to failure of slopes. The failure of soil in a down ward and out ward movement of a slope is called a slide or slope failure (Abramson, and Boyce, 1996).

Slides occur in almost every conceivable manner, slowly or suddenly and with or without any apparent provocation .They are usually caused by excavation, by undercutting the foot of an existing slope, by a gradual disintegration of the structure of the soil, by an increase of the pore water pressure in a few exceptionally permeable layers, or by a shock that liquefies the soil Problems associated with failures of natural and artificial slopes often pose formidable challenges in geotechnical engineering (Nelson, 2010).

In general, an exposed inclined ground surface that is unrestrained may be prone to mass movement due to gravitational forces. The resulting shear stresses, induced along a potential or known failure surface, slope failure occurs when the shear stress along failure plane exceeds the shear strength of the soil. The ratio of available shear strength to induced shear stress in a potential failure surface is referred to as the factor of safety (Nelson, 2010).

2.2 Causes and Triggering Factors of Landslides

In discussing the various causes of slope failures, it is useful to begin by considering the fundamental requirement for stability of slopes. This is that the shear strength of the soil must be greater than the shear stress required forequilibrium. Given this basic requirement, it follows that the most fundamental cause of instability is that, for some reason, the shear strength of the soil is less than the shear stress required for equilibrium (Nelson, 2010)**.**

2.2.1 Due to the Increase or Decrease in Shear Stress

Slope failure occurs when the downward movements of material due to gravity and shear stresses exceeds the shear strength. Therefore, factors that tend to increase the shear stresses or decrease the shear strength increase the chances of failure of a slope. Different processes can lead to reduction in the shear strengths of rock mass such as Increase in pore pressure, cracking, swelling, decomposition of clayey rock fills, creep under sustained loads, leaching, strain softening, weathering and cyclic loading are common factors that decrease the shear strength of rock mass. In contract to this the shear stress in rock mass may increase due to additional loads at the top of the slope and increase in water pressure in cracks at the top of the slope, increase in soil weight due to increased water content, excavation at the bottom of the slope and seismic effects (Nelson, 2010).

2.2.2 Effect of Pore pressure

The effect of water on the slope can be considered into two fold. One is ground water or aquifer below the surface that generates pore water pressure and the other is rainwater infiltration that seeps through surface and flows along the slope generating water pressure. It is related to the surrounding precipitation levels, topography, nearby water masses, and the geo-hydrological characteristics of the rock mass (Varnes, 1978.)

In medium to hard rock, water occupying the fractures within the rock mass can significantly reduce the stability of a rock slope. Water pressure acting within a discontinuity reduces the effective normal stress acting on the plane, thus reducing the shear strength along that plane. If a load is applied at the top of a slope, the pore pressure increases. Such a load can lead to immediate failure of the slope if it exceeds its shear strength of slope. Water filling in discontinuities can result in lowering of stability condition for natural or artificial slopes (Varnes, 1978).

2.2.3 Geological Discontinuities

The stability of rock slopes is significantly influenced by the structural discontinuity in the rock in which the slope is excavated (Varnes, 1978). A discontinuity can be in the form of a bedding plane, schistosity, foliation, joint, cleavage, fracture, fissure, crack, or fault plane. This discontinuity controls the type of failure which may occur in a rock slope. The properties of discontinuities such as orientation, persistence, roughness and infilling are play important role in the stability of jointed rock slope. Discontinuity makes a soil or rock mass anisotropic. The orientation of a major geological discontinuity relative to an engineering structure also controls the possibility of unstable conditions (Varnes, 1978)

2.2.4 Geotechnical Properties of Material

The important geotechnical properties affecting stability of a slope are shear strength of material, particle size distribution, density, permeability, moisture content, plasticity and angle of repose. The strength of rock mass is a very important factor that affects the stability of slopes. Strength of rock mass is a function of strain rate, drainage condition during shear, effective stresses acting on the soil prior to shear, the stress history of the soil, stress path, and any changes in water content and density that may occur over time. It consists of cohesion and friction angle of material. Cohesion results from a bonding between the surfaces of particles and is dependent upon many factors, including material properties, magnitude and direction of the applied force and the rate of application, drainage conditions in the mass, and the magnitude of the confining pressure (Rahman, 2012).

2.2.5 Erosion

Two aspects of erosion need to be considered from slope stability point of view. The first is a large scale erosion, such as a river erosion occurring at the base of a slope. The second is a relatively localized erosion caused by groundwater or surface runoff. In the first type, erosion changes the geometry of the potentially unstable rock mass. The removal of material at the toe of a potential slide reduces the confining stress that may be stabilizing the slope. Localized erosion of joint filling material, or zones of weathered rock, can effectively decrease interlocking between adjacent rock blocks (Oyedepo O.J and Oluwajana, 2013)

. Loss of such interlocking significantly reduces the rock mass shear strength. The resulting decrease in shear strength may allow a previously stable rock mass to move causing slope failure. In addition, localized erosion may also result in increased permeability and ground-water flow thus affecting the stability of rock slope (Oyedepo O.J and Oluwajana, 2013).

2.2.6 Seismic Effect

Seismic waves passing through rock increase stress which could causes fracturing in the rock mass. As a result, friction is reduced in unconsolidated masses as they are tarred apart which may induce liquefaction. Blasting and earthquakes events affect rock slopes in two distinct ways with different time scales. The first effect is in the form of immediate co-seismic detachment of rock from a slope face. The second effect occurs over a longer timeframe involving opening of fissures and rock fracturing that may result in rock dislodgements in the future. Such effects of seismicity on rock slopes strongly depend on local conditions of the rock mass (Nelson, 2010).

2.2.7 Vegetation

Plant roots provide a strong interlocking network to hold unconsolidated materials together and prevent flow. Furthermore, plants are very effective in removing water from the soil, thus increasing the shear strength. Although, the extra weight of plants may cause a slight destabilizing effect if the root network is of limited extent, the overall vegetation increases stability of a slope. Different types of vegetation like grasses, herbs, shrubs and trees are used to stabilize the slope stability and reinforcement of the soil (Liu, 2016).

2.2.8 Influence of Slope Geometry

Slope steepness is a principal factor in the determination of the intensity and character of landslides. It has direct as well as indirect influences. Direct influences encompass slope, steepness, and river valley morphology. The most important relief characteristic is the steepness, which affects the mechanism as well as the intensity of the landslides. The greater the height, steepness and convexity of slopes, the greater the volumes of the landslides. On a steeper slope, the shear stress or tangential component of gravity increases and the perpendicular component of gravity decreases. The forces resisting movement down the slope are grouped under the term shear strength which includes frictional resistance and cohesion among the particles that make up the object as shown in figure 2.1 below. When the shear stress becomes greater than the shear strength then the slope fails (UNESCO/UNE, 1988).

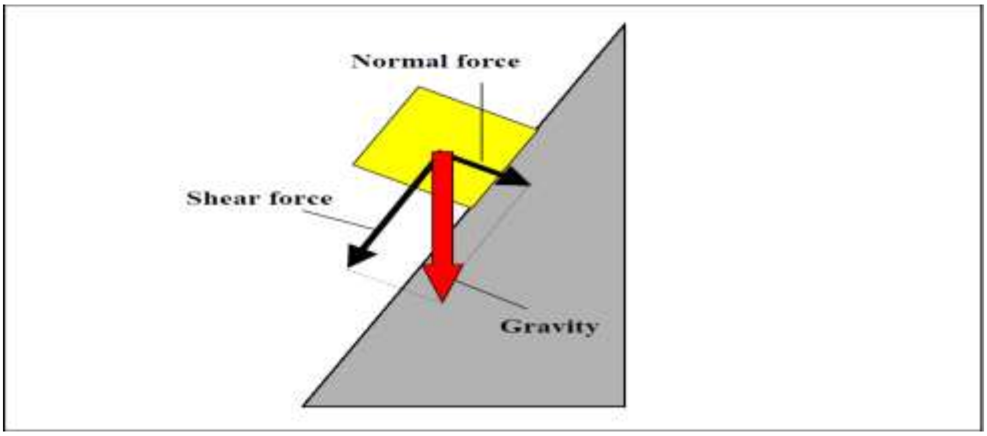

Figure 2.1 Shear force and normal force to gravity on a steep slope (UNESCO,1988).

2.3 Landslide Mechanics

Slope instability is the condition which gives rise to slope movements. In every slope there are forces (stresses) which tend to promote movement (shear stress), and opposing forces which tend to resist movement (shear strength). Sliding occurs when the forces tending to cause movements are greater than those resisting it. In normal circumstances, the shear stress is balanced by shear strength and a state of equilibrium is maintained. However, this equilibrium can be disturbed by stress increments or the weakening of frictional force (Alexander, 1993)**.**

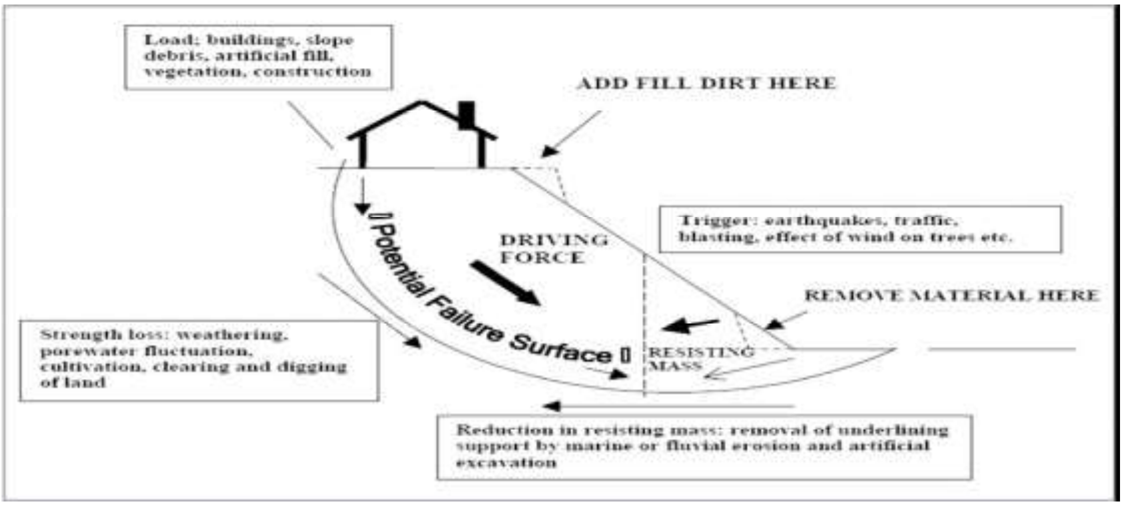

Figure 2.2 Landslide mechanics(after Alexander, 1993)

2.3.1 Shear Strength of Soil

The way the soil particles, behave as a group or mass depends not only upon the inner cohesion of the particles but also upon the friction generated between individual soil grains. The latter characteristic is termed internal friction or shearing resistance. How much shear stress a soil or regolith can withstand is given by the Mohr Coulomb equation (Msilimba, 2002).

2.4 Classification of Slopes

Slopes can be divided into three classes according to: - Stable slopes are those whose margin of stability is sufficiently high to withstand all destabilizing forces. (2) Marginallystable slopes are those that will fail at some time in response to the destabilizing forces attaining a critical level of activity, and (3) Active unstable slopes are those in which destabilizing forces produce continuous or intermittent movement (Msilimba, 2002).

2.5Landslide Hazard Mitigation Measures

Slope stabilization methods generally reduce driving forces, increase resisting forces. Driving forces can be reduced by excavation of material from the appropriate part of the unstable ground and drainage of water to reduce the hydrostatic pressures acting on the unstable zone. Resisting forces can be increased by Drainage that increases the shear strength of the ground by Elimination of weak strata or other potential failure zones

,Building of retaining structures or other supports Provision of in situ reinforcement of the ground and Chemical treatment (hardening of soils) to increase shear strength of the ground (Msilimba, 2002).

For remediation of Land slide or slope instability one should answer the following questions the cause of the cut slope instability and the amount of remediation needed to maintain stability for reasonably foreseeable future. The methods for correcting and prevention of slope instability related problems are quite many, however a chosen few methods which are related to cut slope instability problems are briefed below (Msilimba, 2002).

2.5.1 Avoidance

Relocation of the highway or road stretch affected by the slide from the limits of the slide area by provision of realignments. This method is cost effective if the relocation is for short stretches of the road section (Msilimba, 2002).

2.5.2 Movement of Earth

Reduction in shear stresses on the slip surface can be achieved by removal of slide material. This can be achieved by the removal of slide material from the head (upper part) and by flattening of the slide mass (Nelson, 2010).

2.5.3 Improving of Stability by Geometric Methods

The increase in slope angle can disturb the balance of forces, therefore, slope reduction is recommended to create a gentler slope and this might reduce the component of the gravitational force acting along the slope (Alexander, 1993). In areas where slopes are collapsing on their own, the construction of embankments, whose size can be determined by the selection of gradient that will provide a stable slope given the local hydraulic conditions can provide stability. The embankments can be of stone and wire mesh **(**Misilimba, 2007).

2.5.4 Stabilization by Drainage

Stabilization by drainage has been noted as a very effective means of protecting unstable hill slopes from sliding. Water was noted to have infiltrated into the weathered and jointed beds which in turn increase both pore and cleft water pressures. The construction of concrete embankments may result in a buildup of water in the slope, creating further instability. Stone embankments, using wire meshes, will reduce significantly the buildup of water in the slope such structures are less expensive and more applicable to developing countries (Pilot, 1988)**.**

2.5.5 Engineering Structures to Mitigate Damage

Where movement is inevitable, engineering structures can reduce the damage. These include (a) the use of cable nets and wire fences that catch rock blocks before they cause damage (b) reinforcing structures such as rock sheds and tunnels which allow the mass to pass over without collapsing; and (c) embankments whose size is determined by the selection of gradient that produces a stable slope given the local hydraulic conditions (Crozier, 1999).

2.5.6 Retaining Structures

These structures are basically constructed at base and toe of the sliding surface. Accordingly, support at the base can be achieved by the provision surcharge at toe by construction of rock or earth fill while common or crib retaining wall structures are constructed at the base/toe of the slide mass to restrict movement of the slide mass. Furthermore, piles (concrete or steel) are fixed in the slip surface in order to increase the strength of the slip surface. Alternatively speaking, the strength of the failure surface is increased by the amount of the stress required to make the piles fail (Abramson, et al., 1996).

The most common use of retaining walls for slope stabilization is when a cut or fill is required and there is not sufficient space or right- of -way available for just the slope itself. The wall should be deep enough so that the critical slip surface passes around it with an adequate FOS (Abramson, et al., 1996).

2.5.7 Slope Modification

Modification of a slope either by humans or by natural causes can result in changing the slope angle as indicated below in figure 2.3, so that it is no longer at the angle of repose. A mass movement event can then restore the slope to its angle of repose. Undercutting - streams eroding their banks or surf action along a coast can undercut a slope making it unstable (Nelson, 2010)**.**

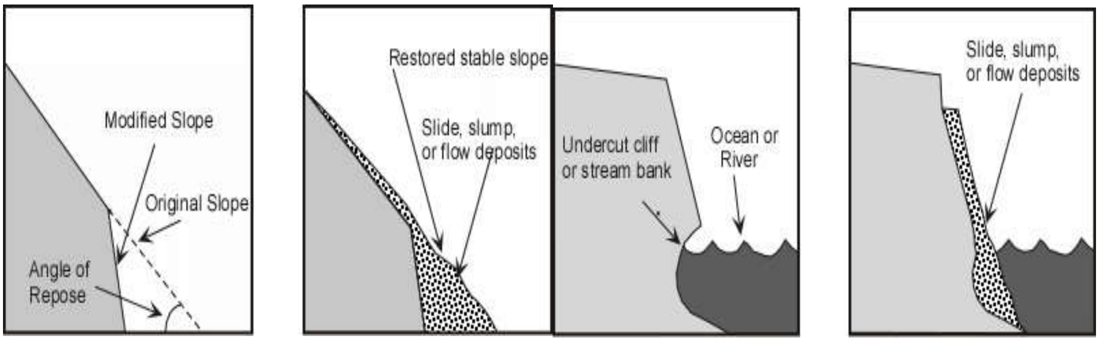

Figure 2.3 Slope modification

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

2.6 Modes of Failures in Rock Slopes

The stability of rock slopes is essentially governed by the: Joint sets and their orientation, Joint characteristics, Seepage pressure, Depth and steepness of the excavated slope face and Orientation of slope face with respect to the joint set. Common modes of failures in rock slopes are (Spang, 1995).

(A) Circular/rotational or quasi-rotational failure:-:

In this type of failure Sliding along a curved surface is expected in a heavily jointed, crushed and weathered rock mass and often take place at a larger scale. Often to simplify the analysis, the sliding surface is approximated to a circular arc and referred to as circular (Spang, 1995).

(B) Plane slide: this type of failure forms under gravity alone when a rock block rests on an inclined weakness plane that "daylight" into the free space. Inclination of failure plane greater than friction angle of the plane (Spang, 1995).

(C) Wedge failure/slide**:**

Wedge failure occur when two planes of weakness intersect to define a tetrahedral block. This type of Failure can occur if the line of intersection of the two discontinuities daylights into the excavation and Friction angle less inclination of the line of intersection of the two planes (Spang, 1995).

(D)Toppling failure: this type of failure involves overturning of rock layers: When the vector representing the weight of a block falls outside of the base, the block will topple; it will rotate about its lowest contact edge (Spang, 1995).

(E) Buckling failure**:** Buckling type of failure occurs when the excavation is carried out with its face parallel to the thin weakly bonded and steeply dipping layers, depending upon the depth of the excavation, these layers may buckle and fracture near the toe and sliding of the upper portions of the layers may result (Spang, 1995).

2.6.1 Mechanics of Rock Falls

Rock falls are generally initiated by some climatic or biological event that causes a change in the forces acting on a rock. These events may include pore pressure increases due to rainfall infiltration, erosion of surrounding material during heavy rain storms, freeze-thaw processes in cold climates, chemical degradation or weathering of the rock, root growth or leverage by roots moving in high winds (Azzoni,1995).

Once movement of a rock perched on the top of a slope has been initiated, the most important factor controlling its fall trajectory is the geometry of the slope. In particular, dip slope faces, such as those created by the sheet joints in granites, are important because they impart a horizontal component to the path taken by a rock after it bounces on the slope or rolls off the slope (Spang, 1995).

2.7 Causes of Rock Falls

2.7.1 Weak Materials and Structures

Weak materials and structures (bedding planes, weak layers, joints and fractures, and foliation planes) have their own impact on rock falls (Scharpe, 1938).

2.7.1.1 Bedding Planes

These are basically planar layers of rocks upon which original deposition occurred. Since they are planar and since they may have a dip down-slope, they can form surfaces upon which sliding occurs, particularly if water can enter along the bedding plane to reduce cohesion, In the diagram below, note how the slope above the road on the left is inherently less stable than the slope above the road on the right (Varnes, 1978).

2.7.1.2 Weak Layers

Some rocks are stronger than others. In particular, clay minerals generally tend to have a low shear strength. If a weak rock or soil occurs between stronger rocks or soils, the weak layer will be the most likely place for failure to occur, especially if the layer dips in a down-slope direction as in the illustration above. Similarly, loose unconsolidated sand has no cohesive strength. A layer of such sand then becomes a weak layer in the slope (Varnes, 1978).

2.7.1.3 Joints and Fractures

Joints are regularly spaced fractures or cracks in rocks that show no offset across the fracture (fractures that show an offset are called faults). Joints form as a result of expansion due to cooling, or relief of pressure as overlying rocks are removed by erosion. Joints form free space in rock by which water, animals, or plants can enter to reduce the cohesion of the rock. If the joints are parallel to the slope they may become a sliding surface. Combined with joints running perpendicular to the slope (as seen in the sandstone body in the illustration above), the joint pattern results in fractures along which blocks can become loosened to slide down-slope (Varnes, 1978).

2.8 Effects of Rock Fall on Roads

Rock falls are a major hazard in rock cuts for highways and railways in mountainous terrain. While rock falls do not pose the same level of economic risk as large scale failures which can and do close major transportation routes for days at a time, the number of people killed by rock falls tends to be of the same order as people killed by all other forms of rock slope instability (Badger, and Lowell, 1992).

2.9 Possible Measures Which Could Be Taken to Reduce Rock Fall Hazards

2.9.1 Identification of Potential Rock Fall Problem

It is either possible or practical to detect all potential rockfall hazards by any techniques currently in use in rock engineering. In some cases, for example, when dealing with boulders on the top of slopes, the rockfall hazards are obvious. However, the most dangerous types of rock failure occur when a block is suddenly released from an apparently sound face by relatively small deformations in the surrounding rock mass (Badger, and Lowell, 1992).

2.9.2 Reduction of Energy Levels Associated With Excavation

Traditional excavation methods for hard rock slopes involve the use of explosives. Even when very carefully planned controlled blasts are carried out, high intensity short duration forces act on the rock mass. Blocks and wedges which are at risk can be dislodged by these forces.Hence, an obvious method for reducing rockfall hazards is to eliminate excavation by blasting or by any other method, such as ripping, which imposes concentrated, short duration forces or vibrations on the rock mass. Mechanical and hand excavation methods can used and, where massive rock has to be broken, chemical expanding rock breaking agents may be appropriate (Badger, and Lowell, 1992).

2.9.3 Physical Restraint of Rockfalls

If it is accepted that it is not possible to detect or to prevent all rockfalls, then methods for restraining those rockfalls, which do occur, must be considered. Berms are a very effective means of catching rockfalls and are frequently used on permanent slopes. However, berms can only be excavated from the top downwards and they are of limited use in minimizing the risk of rockfalls during construction (Badger and Lowell, 1992).

Rock traps work well in catching rockfalls provided that there is sufficient room at the toe of the slope to accommodate these rock traps. In the case of very narrow roadways at the toe of steep slopes, there may not be sufficient room to accommodate rock traps. This restriction also applies to earth or rock fills and to gabion walls or massive concrete walls. Catch fences or barrier fences in common use are estimated to have an energy absorption capacity of 100 KN/m^2. The mesh is draped over the rock face and attached at several locations along the slope. The purpose of the mesh is not to stop rockfalls but to trap the falling rock between the mesh and the rock face and so to reduce the horizontal velocity component which causes the rock to bounce out onto the road way (Badger, andLowell, 1992)

Figure 2.4 Geo-brugg ring net restraining after(Spang, R.M., So¨nser, T., 1995)

2.10 Common Limit Equilibrium Based on 2D Slope Stability Analysis Methods

2.10.1 Introduction

Slope stability analysis using software is an easy task for engineers when the slope configuration and the soil parameters are known. However, the selection of the slope stability analysis method is not an easy task and effort should be made to collect the field conditions and the failure observations in order to understand the failure mechanism, which determines the slope stability method that should be used in the analysis. Therefore, the theoretical background of each slope stability method should be investigated in order to properly analyze the slope failure and assess the reliability of the analysis results.

Two dimensional slope stability methods are the most common used methods among engineers due to their simplicity**.** An analysis of slope stability is result of downward or shear force (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the shear force in order for a slope to be stable (Abramson, 1996)**.**

The relative stability of a slope is based on factor of safety Fs defined as:

$$Fs = \frac{\sum R}{\sum M} \quad \text{..........} \quad (2.1)$$

The equation states that the factor of safety is the ratio between the forces/moments resisting (R) moment and the forces/moments motivating (M) moment (Abramson, 1996).

2.10.1.1 The Ordinary Method (OM)

The Ordinary method (OM) satisfies the moment equilibrium for a circular slip surface, but neglects both the interslice normal and shear forces. The advantage of this method is its simplicity in solving the FOS, since the equation does not require an iteration process. The FOS is based on moment equilibrium and computed by (Abramson, 1996).

$$F_m = \frac{\sum(c'l + N\tan\Phi')}{\sum W\sin\alpha} \quad (2.2)$$

$$N' = (W\cos\alpha - ul) \quad (2.3)$$

Where, u= pore pressure, l= slice base length, α = inclination of slip surface at the middle of slice N=base normal force

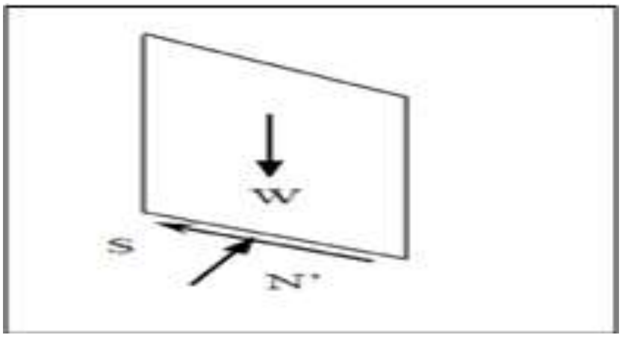

Figure 2.5 Forces acting on typical slice of natural slope (after (Abramson, 1996))

2.10.1.2 Bishop's Methods

Bishop's simplified method (BSM) is very common in practice for circular shear surface (cSS). This method considers the interslice normal forces but neglects the interslice shear forces (Abramson, 1996). It further satisfies vertical force equilibrium to determine the effective base normal force (N'), which is

$$P = \frac{1}{m_\alpha}\sum\left(W - \frac{c'l\sin\alpha}{F} - ul\cos\alpha\right) \quad (2.4)$$

Where,

$$m_a = cos\alpha(1 + tan\alpha\frac{tan\Phi\prime}{F}) \ldots\ldots (2.5)$$

Since the BSM also assumes a circular failure surface, the same Eq. (3.1) is utilized to determine the FOS. However, computation requires an iterative procedure because of the nonlinear relationship as the FOS appears on both sides.

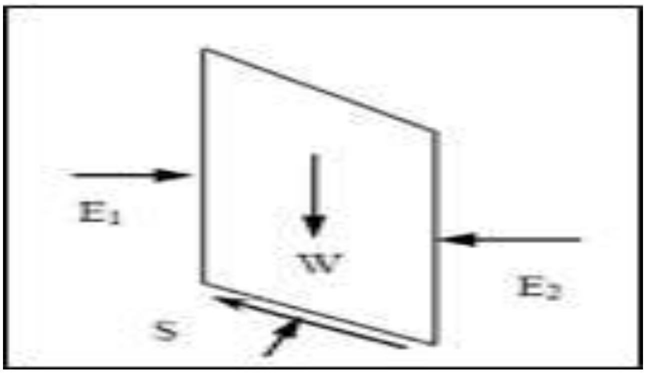

Figure 2.6 Forces acting on typical slice of natural slope (after (Abramson, 1996)

The Bishop rigorous method (BRM) considers the interslice shear forces (T) in addition to interslice normal forces (E). The method further assumes a unique distribution of their resultant forces and satisfies moment equilibrium of each slice. The interslice T and E forces, and hence the FOS are determined by an iteration procedure (Abramson, 1996)**.**

2.10.1.3 Janbu's Method

The simplified method, generalized method (GPS) and direct method developed by Janbu are very common in stability analysis. The fundamental differences in these methods are briefly reviewed below **(**Janbu, 1973)**.**

A. Janbu's Simplified Method

Janbu's simplified method (JSM) is based on a composite SS (i.e. non circular) and the FOS is determined by horizontal force equilibrium. As in BSM, the method considers interslice normal forces (E) but neglects the shear forces (T). The base normal force (N) is determined in the same way as in BSM and the FOS is computed by **(**Janbu, 1973).

$$\boldsymbol{F_f} = \frac{\sum(c'l + (N - ul)\tan\Phi')\sec\alpha}{\sum W\tan\alpha + \sum\Delta E} \ldots\ldots (2.6)$$

Where, $\sum \Delta E = E_2 - E_1$ = net interslice normal forces (zero if there is no horizontal force) **(**Janbu, 1973). Eq. 2.6 is written as

$$F_o = \frac{\sum\left(\frac{b(c' + (P-u)\tan\emptyset')}{n_a}\right.}{\sum p.btan\alpha} \text{......} (2.7)$$

$$n_a = cos^2\alpha(1 + tan\alpha\frac{tan\emptyset'}{F}) \text{......} (2.8)$$

Where, W/b = total vertical stress and b = width of slice

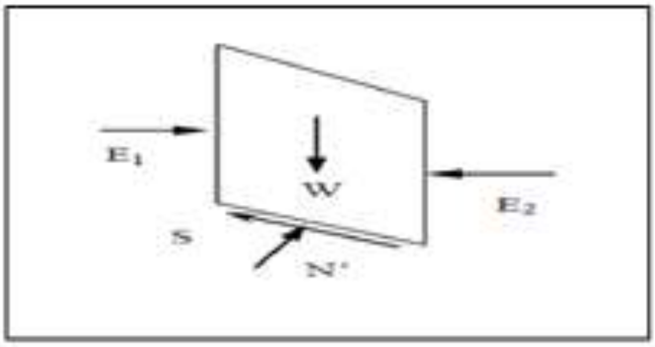

Figure 2.7 Forces acting on typical slice of natural slope(after Abramson, 1996)

Janbu introduced a correction factor (fo)), in the original FOS (fo), to accommodate the effects of the interslice shear forces. With this modification, Janbu's corrected method (JCM) gives higher FOS, as:

$$F_f = F_o * FOS \text{......} (2.9)$$

The correction factor depends on the depth to length ratio (D/L) of the failure surface. The FOS, with this correction factor, can increase by 5 12%, giving the lower range in friction only soils, i.e. the soils without cohesion and the higher range for clayey soils (Abramson, 1996).

B Janbu's Generalized Method

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

$$\boldsymbol{F_f} = \frac{\sum[(c'l+(N-ul)tan\emptyset')sec\alpha]}{\sum(W-(T_2-T_1))tan\alpha+\sum(E_2-E_1)} \text{...} (2.10)$$

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

$$N = \frac{1}{m_a}\left(W - (T_2 - T_1) - \frac{1}{F}(c'l - ultan\emptyset')sin\alpha\right) \text{......................................} (2.11)$$

This is the first method that satisfies both force and moment equilibrium. The moment equilibrium for the total sliding mass is explicitly satisfied by considering an infinitesimal slice width (dx) and taking moments about the midpoint of the slice base (Janbu, 1973).

The infinitesimal slice width was introduced to avoid the confusion about the point of application of base normal force (Janbu, 1973). This equilibrium condition shown in equation 2.12 below gives the relationship between the interslice forces (E and T) as:

$$T = tan\alpha_t E - \frac{dE}{dx}h_t \text{..} (2.12)$$

Where, $tan\alpha_t$ = slope of the line of thrust, and h_t = height from the midpoint of the slice base to dE.

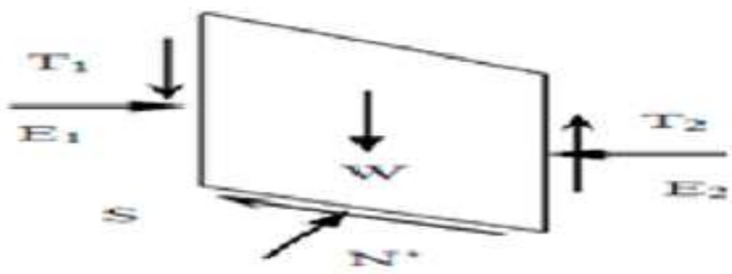

Figure 2.8 Forces acting on typical slice of natural slope (after Abramson, 1996)

2.10.1.4 Morgenstern Price Method

According Morgenstern, N.R and Price, 1965 the Morgenstern Price method (M PM) also satisfies both force and moment equilibriums and assumes the interslice force function. The interslice force inclination can vary with an arbitrary function (f(x)) as

$$T = f(x).\lambda.E \text{...} (2.13)$$

Where, F(x) = interslice force function that varies continuously along the slip surface,

λ = scale factor of the assumed function.

The method suggests assuming any type of force function, for example half sine, trapezoidal or user defined. The relationships for the base normal force (N) and interslice forces (E, T) are the same as given in JGM. For a given force function, the interslice forces are computed by iteration procedure until Ff (equation 2.12) is equals to Fm which is shown below in equation 2.17 (Janbu, 1973).

$$F_m = \frac{\sum(c'l+(N-ul)tan\emptyset')}{\sum Wsin\alpha} \qquad (2.14)$$

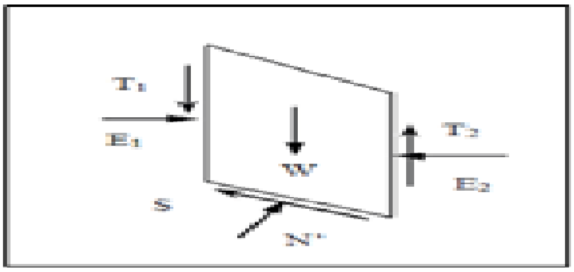

Figure 2.9 Forces acting on typical slice of natural slope (after Abramson, L.W., 1996)

2.11 Choice of LEM of Analysis

The various analysis methods discussed in the previous sections share some common fundamental features and limitations. In general, when interpreting the factor of safety (FOS) computed using any one of these methods, reliability of the input data that define the conditions under analyses should be considered. Choice of method of analysis could be regarded with respect to the following (Spencer, 1967).

(A) Accuracy: Various studies have been conducted over the years to evaluate the computational accuracy of limit equilibrium methods. The accuracy of the various methods was evaluated by comparison with what are believed to be correct values for a range of conditions where the slope geometry, water pressures, unit weights and shear strengths are precisely defined. Such studies include Spencer (1967), Wright et al. (1973), Chen and Snitbhan (1975), Huang and Avery (1976), Fredlund and Krahn (1977), Garber and Baker (1979), Sarma (1979), Fredlund et al. (1981.

(B) Ease of Application

Suitability of a particular method of analysis could be regarded with respect to 'ease of application. This refers to the relative ease with which one can understand the methods and employ it to solve practical problems. Ease of application could be regarded with respect to the ease in making reasonable assumptions, Amount of computational effort and time required and Frequency of numerical problems encountered And Ease in making assumptions **(**Spencer, 1967).

According spencer (1967) assumptions employed by the common limit equilibrium methods can be grouped into three. These are: (Assumption type I) this involves Assumptions on interslice shear force distribution (Bishop's Rigorous Method), (Assumption type II) indicates Assumptions on interslice force inclinations (Spencer's Method, Morgenstern and Prices Method). While (Assumption type III) describes assumptions on the locations of point of action of interslice forces (Janbu's Method).

2.12 Ground Investigations

Before any further examination of an existing slope, or the ground onto which a slope is to be built, essential borehole information must be obtained. This information will give details of the strata, moisture content and the standing water level. Also, the presence of any particular plastic layer along which shear could more easily take place will be noted (Rahman, 2012)**.** According Rahman 2012) Ground investigations includes:

- ✓ In-situ and laboratory tests
- ✓ Aerial photographs
- ✓ Study of geological maps and memoirs to indicate probable soil conditions
- ✓ Visiting and observing the slope for the study in this thesis.

2.13 Soil Classification

Geotechnical engineers classify soils, or more properly earth materials, for their properties relative to slope stability, foundation support or use as building material. These systems are designed to predict some of the engineering properties and behavior of a soil based on a few simple laboratory or field tests (ASTM, 2004)**.**

Two soil classification system are popular among geotechnical engineers, which are the unified soil classification system (USCS) and the American Association of State

Highway and Transport Officials (AASHTO) soil classification system. The USCS has two main steps. First of all the soil is designed into either the group of gravels and sands or the groups of silts and clays. Then, particular methods are employed to sort the soil in to one of 36 granular soil types. For the group of silts and clays, the Atterberg limit test result can lead the soil to one of 35fine-grained soil types (Braja, 2007).

The AASHTO system also possesses two dominant steps. first, sieve analysis can confirm if the soil is mostly coarse-grained or fine-grained .the first three groups,A-1,A-2,and A-3,consist mainly of granular particles .the remaining four groups,A-4,A-5,A-6and A-7-6,have at least35% of their particles belonging to silts and clays. Secondly, liquid limit and plasticity index values can help engineers to classify the fine –grained soils further into four groups based on the AASHTO soil classification system (Braja, 2007)**.**

For more information we can see the table below for AASHTO classification for both fine and coarse grained soils.

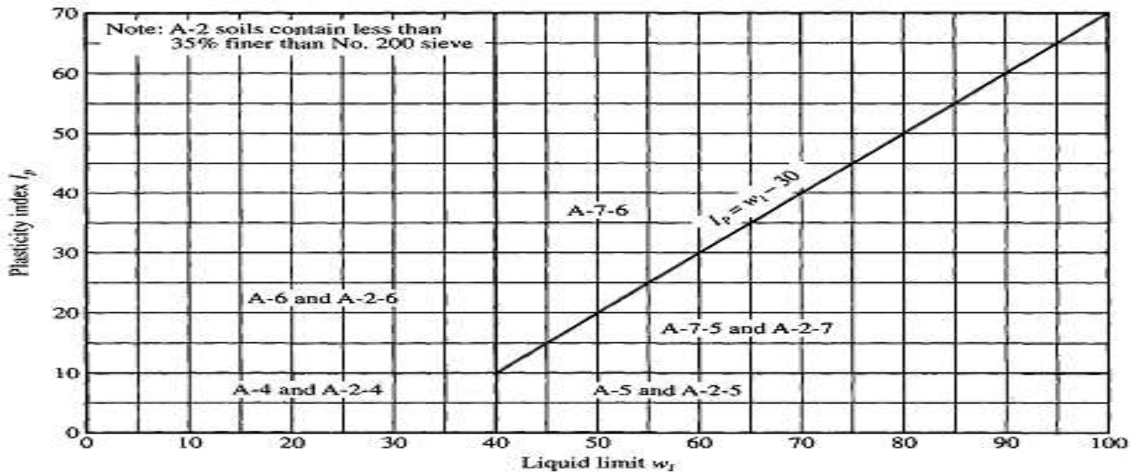

Figure 2.10 Shows-range of LL and PI for soil groups (AASHTO)

Table 2.1 Soil Classification for Fine Grained Based on AASHTO Classification

Soil group		Passing No 200 sieve	Liquid limit (LL)	Plastic index(PI)	Material type	Subgrade rating
A-4		36 mini	40 max	10 max	Silty soil	Fair to poor
A-5		36 mini	41 mini	10 max	Silty soil	Fair to poor
A-6		36 mini	40 max	11mimi	Clay soil	Fair to poor
A-7	A-7-5	36 mini	41 mini	11mimi and PI≤ LL-30	Clay soil	Fair to poor
	A-7-6	36 mini	41 mini	11mini and PI≥LL-30	Clay soil	Fair to poor

Table 2.2 AASHTO Classification for Coarse-Grained Soils

Soil group		Passing No 10 sieve	Passing No 40 sieve	Passing No 200 sieve	Liquid limit	Plasticity index	Material type	Sub grade rating
A-1	A-1-a	50 max	30 max	15 max		6 max	Stone fragments, gravel and sand	Excellent to good
	A-1-b		50 max	25 max		6 max		
A-3			51 min	10 max		Non - plastic	Fine sand	
A-2	A-2-4			35 max	10 max	10 max	Silty and clayey gravel and sand	
	A-2-5			35 max	41 min	10 max		
	A-2-6			35 max	40 max	11 min		
	A-2-7			35 max	41 min	11min		

2.14 Shear Strength Tests

2.14.1 Introduction

The slope stability analysis methods like limit equilibrium methods are used for evaluating the stability of slopes require 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 state of stress, drainage, over consolidation ratio, loading rates, and soil compaction (Abramson, 1996).

Measured soil strength parameters are not unique. Their magnitudes depend on a number of factors such as structure of the soil, stress history, drainage condition, method of loading, definition of failure, degree of disturbance, rate of loading (Robin Chowdhury, 2010).Generally, quality of laboratory test results depends on type of soil, quality and size of test sample, testing method and skill of testing personnel (technicians)Jean-Pierre Bardet, 1997).

The shear strength of soil is one of the most important aspects of geotechnical engineering. The bearing capacity of shallow and deep foundations, slope stability, retaining wall design and pavement design are all influenced by the shear strength of the soil. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads. There are two components of shear strength, a cohesive element (expressed as the cohesion, c, in units of force/unit area) and a frictional element (expressed as the angle of internal friction, ϕ). These parameters are expressed in the form of total stress (c, ϕ) or effective stress (c', ϕ') (Bishop, 1955).

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

$$\tau = c + \sigma \tan\varphi \quad \ldots\ldots (2.15)$$

Where τ is shear strength (i.e., shear at failure), c is cohesion, σ is normal Stress on shear plane, and φ is angle of internal friction. The above equation 2.20 represents a straight line on shear strength versus normal stress plot. Figure 2.11the intercept on the shear strength axis is the cohesion c and the slope of the line is the angle of internal friction φ.

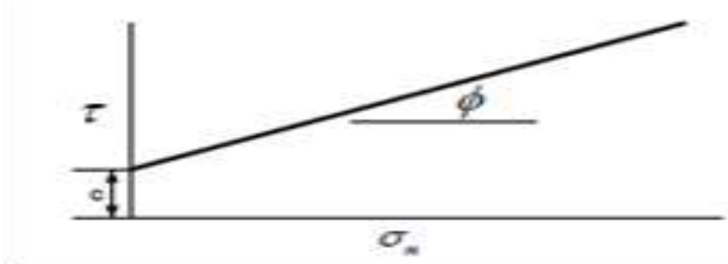

Figure 2.11 Graphical representation of Coulomb shear strength equation (after Bishop, 1955).

The failure envelope is often determined from direct shear tests and the results are presented in terms of half-Mohr circles, as shown in Figure below. Hence the failure envelope is referred to as the Mohr-Coulomb failure envelope.

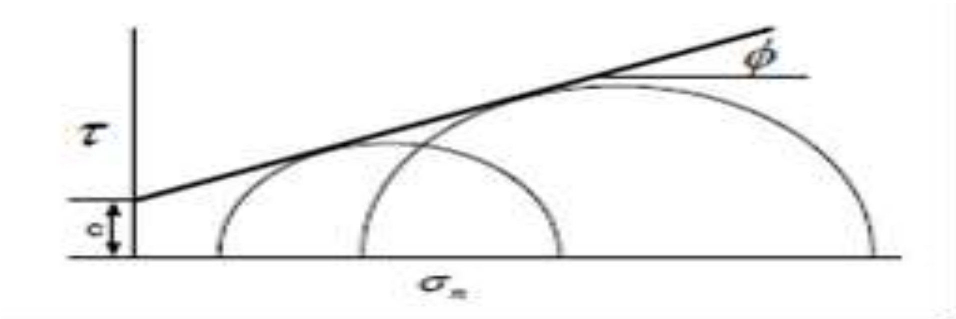

Figure 2.12 Mohr-Coulomb failure envelope (afterBishop, 1955).

2.15 Factor of Safety

For most limit equilibrium methods, the factor of safety for slope stability analysis is usually defined as the ratio of the ultimate shear strength divided by the mobilized shear stress at incipient failure. There are several ways in formulating the factor of safety, FS. The most common formulation for FS assumes the factor of safety to be constant along the slip surface, and it is defined with respect to the force or moment equilibrium (Bishop, 1955).

According equilibrium (Bishop, 1955), Moment Equilibriumgenerally used for the analysis of rotational landslides. Considering a slip surface, the factor of safety F_m, defined with respect to moment is given by:

$$F_m = \frac{M_r}{M_d} \qquad (2.16)$$

Where:M_r is the sum of the resisting moments

M_d is the sum of the driving moments

For a circular failure surface, the center of the circle is usually taken as the moment point for convenience. For a non - circular failure surface, an arbitrary point for the moment consideration may be taken in the analysis equilibrium (Bishop, 1955).

Bishop (1955) states, Force Equilibriumgenerally applied to translational or rotational failures composed of planar or polygonal slip surfaces. The factor of safety F_f defined with respect to force is given by;

$$F_f = \frac{F_r}{F_d} \qquad (2.17)$$

Where:F_r is the sum of the resisting forces

F_d is the sum of the driving forces

For 'Simplified methods' which cannot fulfill both force and moment equilibrium simultaneously, these two definitions will be slightly different in the values and meaning, but a single factor of safety is specified for designs. A slope may actually possess several factors of safety according to different methods of analysis (Bishop, 1955).

CHAPTER THREE

MATERIAL AND METHODS

3.1Location and Description of the Study Area

The route is about 27 km long and traversing between Agoro - Shahgubi section of the Agulae- Berhale road. It is found partly in the Atsbi-wenberta Wereda, Tigray Regional State and partly in the Berhale Wereda, Afar Regional State in Northern Ethiopia. Geographically, the study area is bounded by UTM coordinates of 58°41′33′′ - 59°31′42′′ E and 150°78′16′′ - 151°27′07′′N.

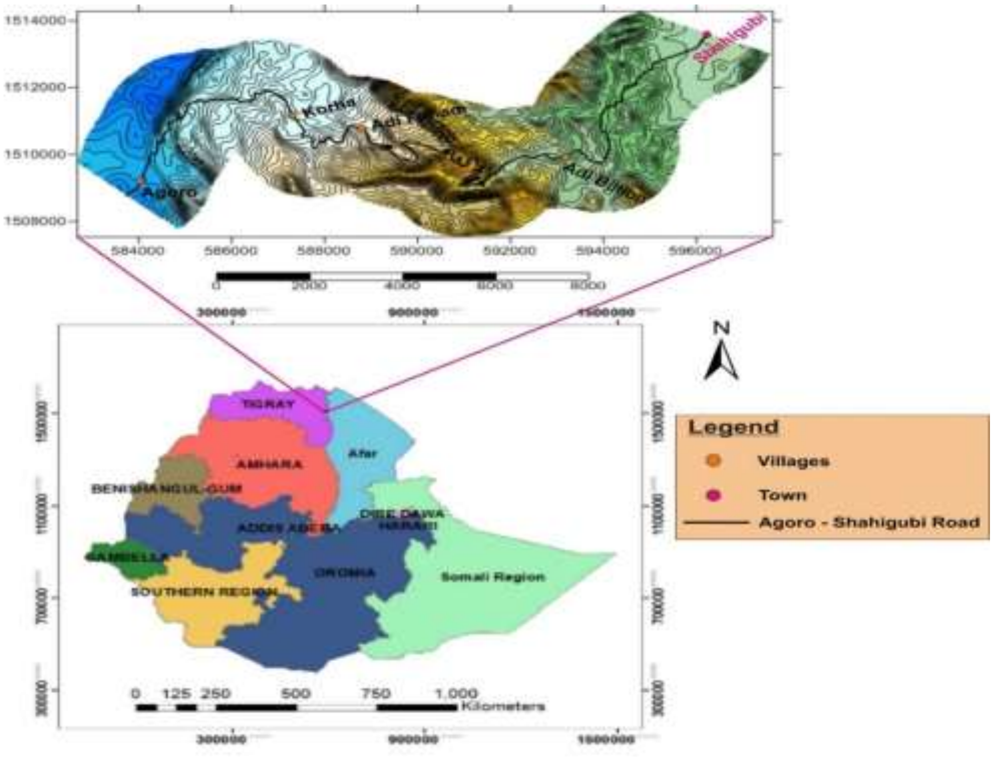

Figure 3.1 Location of the study area

3.2Topography of the Study Area

The study area is part of the western afar margin (Ethiopian escarpment) and is generally characterized by highly variable topography which is a reflection of the past geological and erosional process. This landscape includes plateaus, steep hill slopes, and deeply incised valleys and gorges. The lithological units are downthrown towards east direction which is controlled by the N-S trending rift scarps and its elevation ranging from 1150 to 2450m.

3.3Drainage Pattern

A drainage pattern is strongly influenced by topography, geology and/or structures, climate and vegetation cover of an area (Barry, 1992). The watershed of the study area is largely covered by the Shale and Sandstone units. The Shale generally covers extensive portion of the study area as compared to the other geological formations so that the impervious nature of the shale material usually promotes surface run-off (less infiltration) as opposed to the Sandstone unit. The streams of the study area are running from the Ethiopian escarpment (high rainfall) through deep and narrow gorges towards the marginal basin. The drainage system of the area is well - defined by dendritic pattern. "Gaharta and Salahue" are the two major rivers in the area and there are numerous small tributaries joining at angles to those major rivers. There are also several seasonal streams having large flow during wet season and drying up in the dry season. Finally, the major rivers in the study area drain towards eastward.

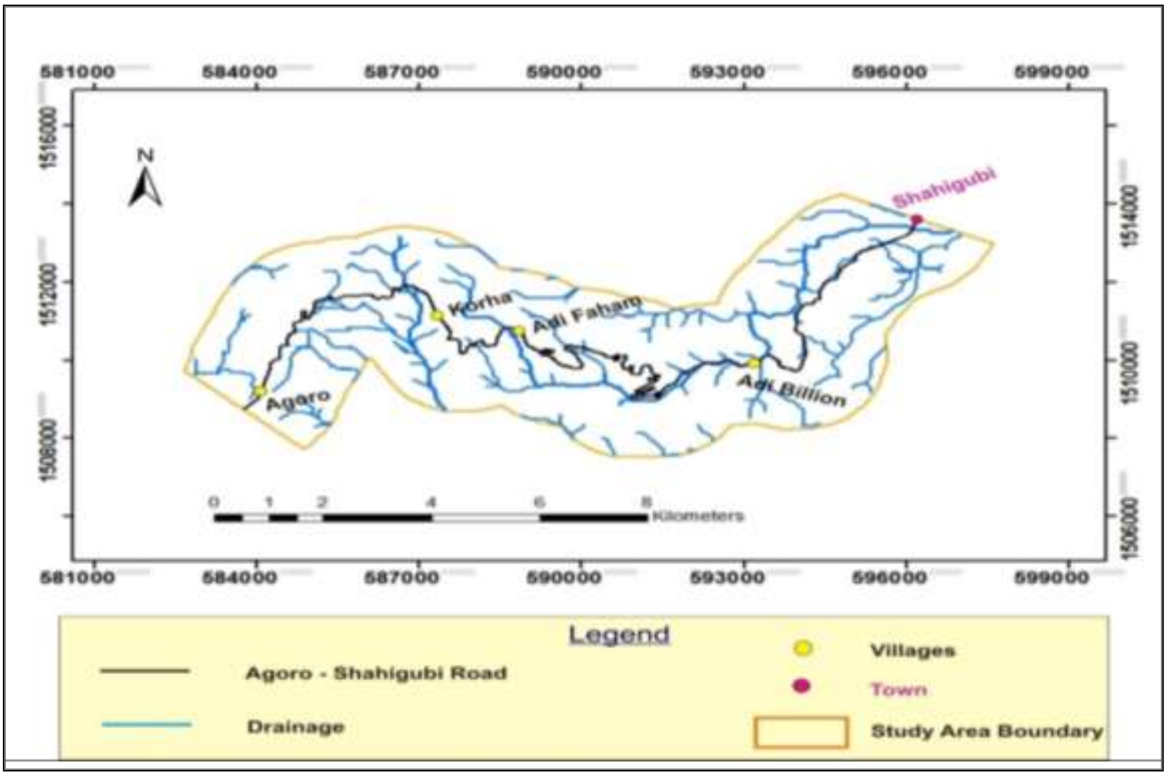

Figure 3.2 Drainage map of the study area

3.4Climatic Condition of the Study Area

Climate is generally controlled by altitude, latitude and cloud cover (Barry, 1992). Rainfall and temperature determine the climate of an area. Although Ethiopia is located in the tropics, temperature and rainfall vary greatly with altitude and depicts large climate variation starting from desert to temperate. Based on the most commonly used, the traditional climatic classifications of Ethiopia, the study area which belongs to the Atsbi wenberta Wereda, Tigray Regional State is classified under temperate climatic zone (ET. Weinadega) and the study area which belongs to the Berhale Wereda, Afar Regional State is classified under tropical to sub - tropical climatic zone (ET. Kolla). The mean monthly rainfall, and mean monthly minimum and maximum temperature data in Atsbi & Berhale stations (2006 – 2016) are shown in Figure 3.3

3.5Rainfall

The rainfall of Ethiopia is dominantly controlled by the north-southward displacement of the Inter-tropical Convergence Zone (ITCZ). As the study area is found partly in the Atsbi wenberta Wereda, Tigray Regional State and partly in the Berhale Wereda, Afar Regional State, the amount of rainfall and temperature are erratically distributed. The rainy season in the Atsbi wenberta Wereda runs from June to September what is known as **summer** (ET. Kiremt) forming a uni- modal rainfall pattern, when the ITCZ moves to the north. The dry season is also runs from December to March, **winter** (ET. Bega) when the ITCZ moves to the south. Although, a high rainfall distribution is from June to September, short rains are also recorded in (April, May, October and November). However, the amounts of rainfall in Berhale Wereda, the intense rainfall are recorded in (November, August, September and March) and the dry season is also recorded in (June, December, January and February). The annual mean monthly rainfall (2006–2016) of Atsbi and Berhale meteorological stations are (51.32 mm and 15.10 mm) respectively. The mean monthly rainfall distribution of Atsbi and Berhale meteorological station (2006 – 2016), based on the data acquired from Ethiopian National Meteorological Agency (NMA) is presented in table 3.3 below.

Table 3.1 Mean monthly rainfall data in Atsbi and Berhale respectively (after EMNSA, 2016).

Month	Jan	Feb	Mar	Apr	May.	Jun.	Jul.	Au	Sep	Oct.	Nov	Dec
Mean monthly RF (mm)	2.2	1.29	23	39	35	51	168	193	75	14	15	1.66
	5.0	5.4	23	19	8	2.7	16.5	27	25	19	28	3.9

3.6 Temperature

Temperature is largely influenced by latitude, altitude, distance from large water bodies and ocean current (Barry, 1992). The annual average monthly maximum and minimum temperature (2006 - 2016) of Atsbi and Berhale are (20.12 ^{0}c, 9.39 ^{0}c) and (37.94^{0}c, 23.53^{0}c) respectively. The mean monthly minimum and maximum temperatures of Atsbi and Berhale (2006–2016) as per Ethiopian National Meteorological Agency are presented in Table 3.4 below.

Table 3.2 Mean monthly minimum and maximum temperature data in Atsbi & Berhale) respectively (after EMNSA, 2016).

Month	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug	Sep.	Oct.	Nov.	Dec.
Mean monthly mini ($T^{0}c$)	7.2	7.7	9.24	10.5	11.5	12.2	11.3	11.0	10.	8.17	7.28	6.41
	19	21	22	23	25	26	27	28	27	23.7	20.9	19.6
Mean monthly (T_{max0c})	19	21	21	21.4	21.7	22.8	20.5	20.1	20.3	18.6	17.8	18
	31	33	34	37	39	61.5	35.5	38.2	43.0	38.0	34.	33

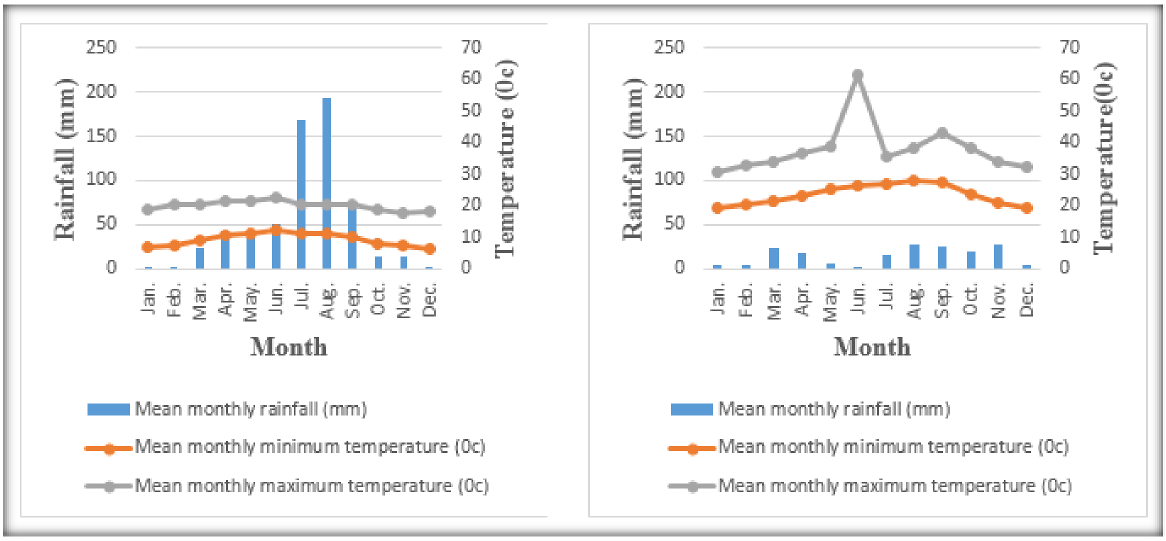

Figure 3.3 Mean monthly rainfall, and mean monthly minimum and maximum temperature data of Atsbi (left) and Berhale (right) (after EMNSA, 2016)

3.7 Research Design

In order to achieve the objectives of the study, the following methodologies were adopted:-

- Visual inspections for the whole stretch of Agoro- Shahgubi road section were made and the failed cut slope sections were identified and selected for further investigations. Which includes geotechnical investigation of soils and rocks (c) measurement of landslide features which includes (length, width, and depth) and failure mechanisms, and (d) selection of appropriate slope stability analysis methods.
- For the selected failed slope, different samples were taken and Laboratory tests were performed accordingly such as moisture content, Atterberg limit, sieve analysis and direct shear tests.
- A detailed literature review to study the theoretical background of the most widely used 2D slope stability analyses methods and the well documented research works were made.
- With obtained shear strength parameters for critical conditions the stability of the slope was checked with the numerical results of LEM by using geoslope software.

For the failed slope, causes of failure and possible remedial measures were pointed out.

- For the finalization of the field work results, the data analyzed according to the objective of the research. During the field observation of this specific site, it was found necessary to start by visual inspection of the whole stretch of Agoro - shahgubi road. During initial visit the whole portion of the road was covered and the failed cut slope sections were identified visually and selected for further detailed investigations. The following photo shows the failed slope which was selected for further investigation.

3.8 Parameters for Slope Stability Evaluation

Grain size, specific gravity, and shear strengthen parameters(cohesion, angle of internal friction and unit weight of soil) thickness of sliding mass, size of slide (lengthen, and width) were the parameters used.

3.9 Field Sampling and Laboratory Test Preparation

Sampling of soils was undertaken in order to determine physical characteristics. Core sampling were carried out, using standard procedures of ASTM. For each failed slope, undisturbed and disturbed samples were collected. The sampling coordinates are given in Table as shown below. The sampling of the failed slopes were taken depends on the geology, critical problems, in areas where landslides hadoccurred, the samples were collected from the sides of the escarpment.

From the sampling area, samples were collected at their moist condition using plastic bags in order to keep the moisture of the sample. In-situ moisture contents were determined immediately, after the samples were brought to the laboratory, using oven temperatures of 105^{o}C for every test sample.

Each sample was dried in oven until continuous weighing gives constant weight. For 105^{o}c oven drying temperature, every sample was dried for 24 hrs. Soil samples from 9 failed slopes were taken   and analyzed to know the properties like cohesion, angle of friction, unit weight of soil, water content, liquid limit, plastic limit and plasticity index, specific gravity and grain size analysis (sieve).

Then these properties were used in classification of soil and Slope stability analysis of landslide. The analysis was conducted through Numerical modeling software package

Geo-Studio-Slope/W 2012. Soil samples collected from the study area were sent to Mekelle university geotechnical and material laboratories. Then after having all the results of the above properties, analysis was done.

Table 3.3 Location depth sampling at the study area.

S/no	Failed slope code	Depth of excavated failed slope, sample taken)	Sample type	Location		
				Easting	Northing	Elevation
1	A-1-01	7.0	Disturbed	584425	1510333	2429
2	A-1-02	6.5	Disturbed,	585392	1511567	2316
3	K	6.4	Disturbed	586605	1511791	1938
4	AF	7.0	Disturbed	589034	1510378	1872
5	AS-1-01	6.5	Disturbed	590667	1510131	1420
6	AS-1-02	5.5	Disturbed	590933	1509680	1224
7	AS-1-03	8.0	Disturbed	591278	1509301	1086
8	AS-1-04	7.5	Disturbed	591352	1509603	1024
9	SG	5.5	Disturbed	594300	1511561	8610

3.10 Software and Instruments

In this research, the following instruments and software were used:

a. GPS Garmin:-to determine landslide position and collecting slope profile data for slope stability analysis.
b. MS word and Excel to analysis laboratory data and display research data
c. Geo-studio 2012/Wfor slope stability analysis

3.11 In-situ Density (Field density)

An in-situ density measurement was conducted by field in -situ apparatus methods at each failed slope where samples have been recovered for laboratory tests. The bulk density was computed upon completion of each test. The field dry densities were later computed based on the natural moisture content from field. The field dry densities and the field moisture contents conducted on the failed slope are tabulated below.

Table 3.4 Field density of soil by using field density apparatus.

Station(km)	Soil Type	Bulk density(g/cc)	Moisture content (%)
A-1-01	Clay	1.646	8.4
A-1-02	Clay	1.686	9
K	Clay	1.675	10.8
AS-1-01	Silty	1.675	9.5
AS-1-02	Clay	1.621	11.8
AS-1-03	Silty and Clayey	1.625	9.7
AS-1-04	Silty	1.615	8.8
AF	Silty	1.5	12.3
SG	Silty and Clayey Gravel and Sand	1.598	12.9

CHAPTER FOUR

Results and discussions

4.1 Laboratory Test Results

Nine samples were collected from each failed slope to perform the necessary tests. Samples were collected and labeled immediately after the field test is performed.The necessary tests are conducted for all the samples and the summary of the results is presented in a tabulated form below. The laboratory data analysis is given in their respective appendices

4.1.1 Moisture Content

This test was conducted to determine the water (moisture) content of soils. Water content tests were run on nine failed slope. (Appendix A).As can be noted from Table 4.1 the water content of the soils varied from 16.8-24.4%.

Table 4.1 Water content of the soils from landslide affected areas

Failed slope profile	Depth of excavated failed slope ,sample taken)	Average Water content (%)
A-1-01	9.0	20.14
A-1-02	8.0	24.4
K	6.4	17.6
AS-1-01	6.5	20.14
AS-1-02	5.5	24.4
AS-1-03	8.0	21.99
AS-1-04	7.5	17.6
AF	7.0	18.9
SG	5.5	16.8

4.1.2 Specific Gravity of Soil

Specific gravity is the ratio of the mass of a unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature (ASTM, 2004). AASHTO: 100-95(1995) manual is followed to determine the specific gravity.

Specific gravity tests were run for the nine samples. Results shows that specific gravity varies from 2.55 to 2.81 and all results of the tests are summarized in Table 4.1. Detail laboratory tests results are attached at the Annex A.

Table 4.2 specific gravity result of the study area

Failed slope profile	Depth of excavated failed slope ,sample taken)	Specific gravity
A-1-01	7.0	2.55
A-1-02	5.2	2.77
K	12	2.81
AS-1-01	6.5	2.61
AS-1-02	4	2.52
AS-1-03	8.0	2.61
AS-1-04	7.5	2.68
AF	7.0	2.72
SG	5.5	2.68

4.1.3 Unit Weight of Soils

This test was performed to determine the in-place density of undisturbed soil obtained by field density apparatus. The bulk density is the ratio of mass of moist soil to the volume of the soil sample (ASTM, 2004). A total of nine tests were carried out to determine the unit weight of soils (following ASTM standards). the dry unit weights of the soil was found to vary from 15 to 16.9 but for the moist unit weight it varies from 16 to 16.9 and results are presented in table 4.3.

$$\gamma_m = \gamma_d(1 + m_c) \qquad (4.1)$$

$$\boldsymbol{\gamma_d} = \frac{\boldsymbol{\gamma_{total}}}{\mathbf{1}+\boldsymbol{m_c}} \qquad (4.2)$$

Where, γm - the moist unit weight of the soil sample.

γ_d - The dry unit weight of the soil sample.

m_c - The moisture content of the soil at the field.

Table 4.3 Unit weight of the soils samples

Failed slope profile Code	Moisture content (%)	Bulk Unit wt. (KN/m^3)	Dry unit wt. (KN/m^3)
A-1-01	8.4	16.5	15.22
A-1-02	9	16.9	15.5
K	10.8	15	14.9
AS-1-01	9.5	16.8	15.34
AS-1-02	11.8	16.2	14.5
AS-1-03	9.7	16.3	14.9
AS-1-04	8.8	16.2	14.9
AF	12.8	16	14.2
SG	10.8	16.8	15.16

4.1.4 Atterberg's Limits Test Result

The liquid limit value for soil samples were determined in accordance to AASHTO T89-96. Plastic limit and plasticity index of the soil were determined according to the procedure stated on AASHTO T90-96. This laboratory is performed to determine the plastic and liquid limits of soils. From the laboratory results, it can be seen that almost all soil types are clay and silty soil with lower degree of swelling, which covers most part of The cross section of the slope is non-plastic.

Table 4.4Liquid Limit, Plastic Limit and Plasticity Index of soils from landslide affected areas

Failed slope profile	Depth of excavated failed slope sample taken)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI
A-1-01	7.0	20.3	9.8	10.5
A-1-02	5.2	25.0	10.6	14.4
K	12	16.2	16.2	0
AS-1-01	6.5	20.3	10.8	9.5
AS-1-02	4	25	14.4	10.6
AS-1-03	6.5	22.6	10.4	12.2
AS-1-04	7.5	17.5	8.0	9.5
AF	7.0	17.5	8.0	9.5
SG	5.5	24.7	16.7	8.0

4.1.5 Shear Strength Parameter Determination

The result from direct shear test (Table 4.5) indicates that the internal friction angle of soil materials and cohesion of the soils from the nine failed slope profile are shown below in the table. From the direct shear test the following shear parameters were obtained which helps in analyzing of slope stability. From this result cohesion value of the soil sample ranges from 1.7581 kN/m^2 to 42.853 kN/m^{2}and the value of internal angle of friction ranges from 11.89^O to 17.79 ^{O}.

Table 4.5 Summary Shear strength parameter of soils from the instable slopes along the route.

Slope profile	Depth of excavated failed slope sample taken)	Cohesion of soil (kN/m^2)	Angle of internal friction(^{o})
A-1-01	7.0	31.16	17.79
A-1-01	5.2	42.9	15.51
K	12	13.0	12.94
AS-1-01	6.5	31.46	12.23
AS-1-02	4	37.1	11.89
AS-1-03	6.5	17.06	15.19
AS-1-04	7.5	1.76	13.78
AF	7.0	31.18	12.24
SG	5.5	38.32	16.91

4.2 Slope Stability Analysis

Cut Slope stability analysis was conducted using 2D Limit equilibrium method using SLOPE/W and this analysis was done using the shear strength parameters that were taken from direct shear test results and moist unit weight derived from the field bulkdensity resul**t.** The safety factors was calculated by slope/W utilizing the Morgenstern Price methods, Ordinary method, modified Bishop Method and Janbu method.

The factor of safety for the moist conditions, which were obtained from Slope/W, is summarized in the table above (table 4.6).

Table 4.6 Input data for slope stability analysis

section	Input data				
	Dry unit weight KN/m^3	Moist unit weight (KN/m^3)	Cohesion (KN/m^3)	Angle of internal friction	Ground water condition
A-1-01	15.22	16.5	31.16	17.79	At great depth
A-1-02	15.5	16.9	42.85	15.51	At great depth
K	14.9	15	13.0	12.94	At great depth
AS-1-01	15.34	16.8	31.5	12.23	At great depth
AS-1-02	14.5	16.2	37.1	11.89	At great depth
AS-1-03	14.9	16.3	17.0	15.19	At great depth
AS-1-04	14.9	16.2	1.76	13.78	At great depth
AF	14.2	16	31.1	12.24	At great depth
SG	15.16	16.8	38.3	16.91	At great depth

From the slope stability analysis (geoslope software) using the above input parameters (cohesion, angle of internal friction and moist unit of the soil) the following results are obtained. Here are Results from Failed slope for selected samples and the others are in appendix Results from failed slope code A-1-01.

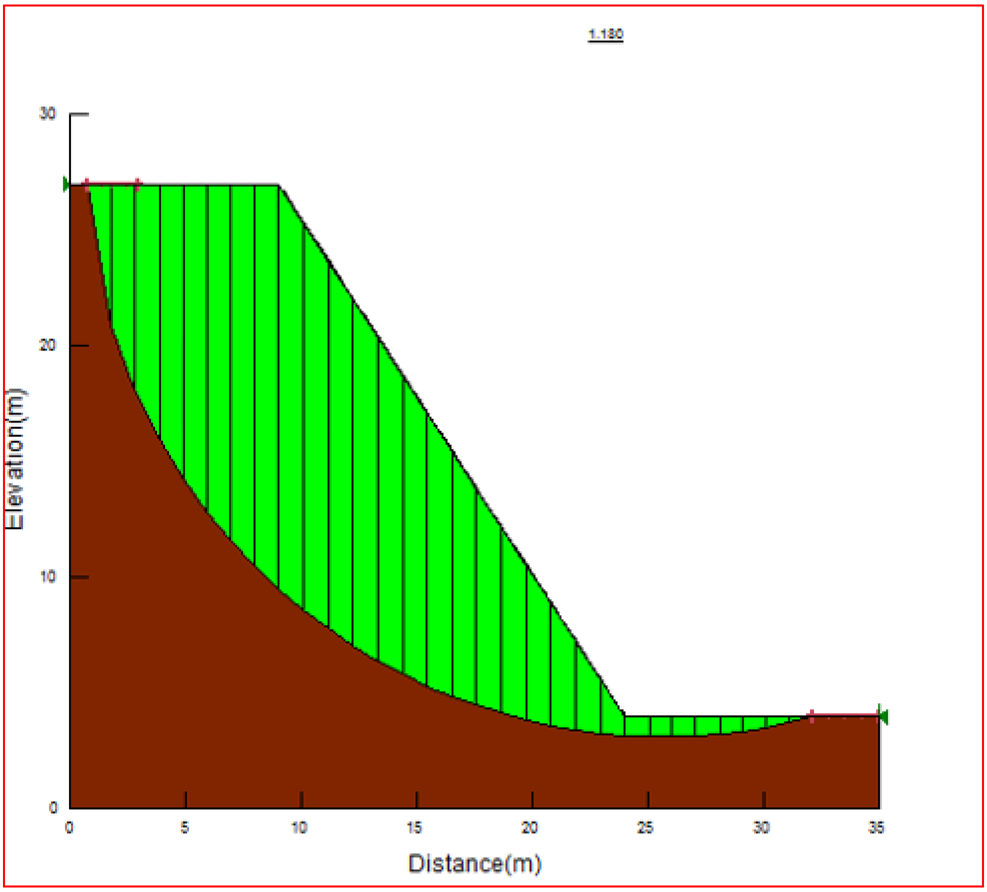

Figure 4.1 Most critical failure circle for homogenous failed slope A-1-01

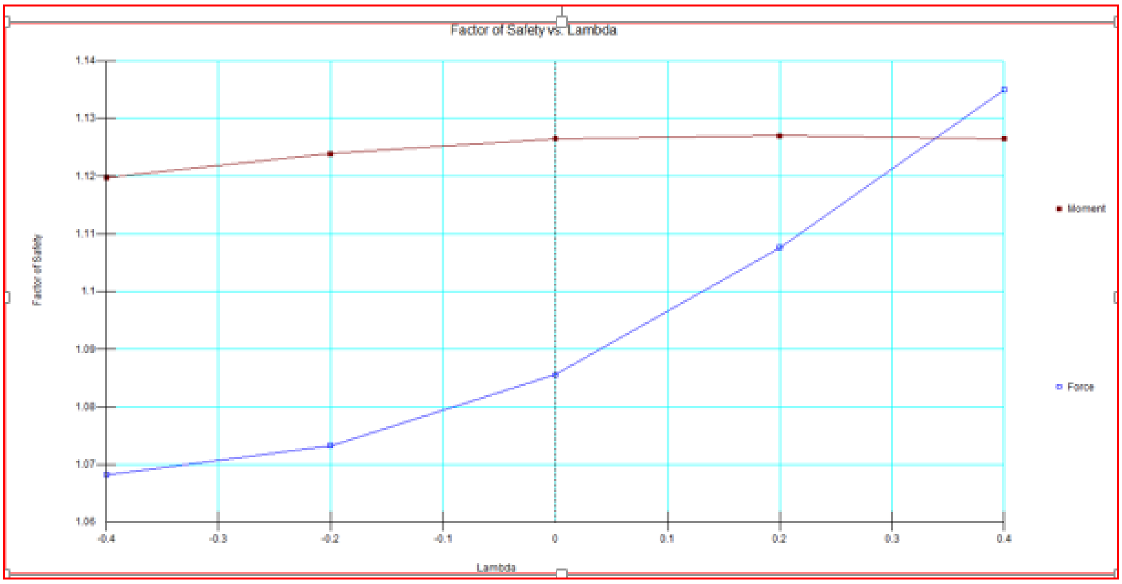

Figure 4.2 lambda vs. factor of safety

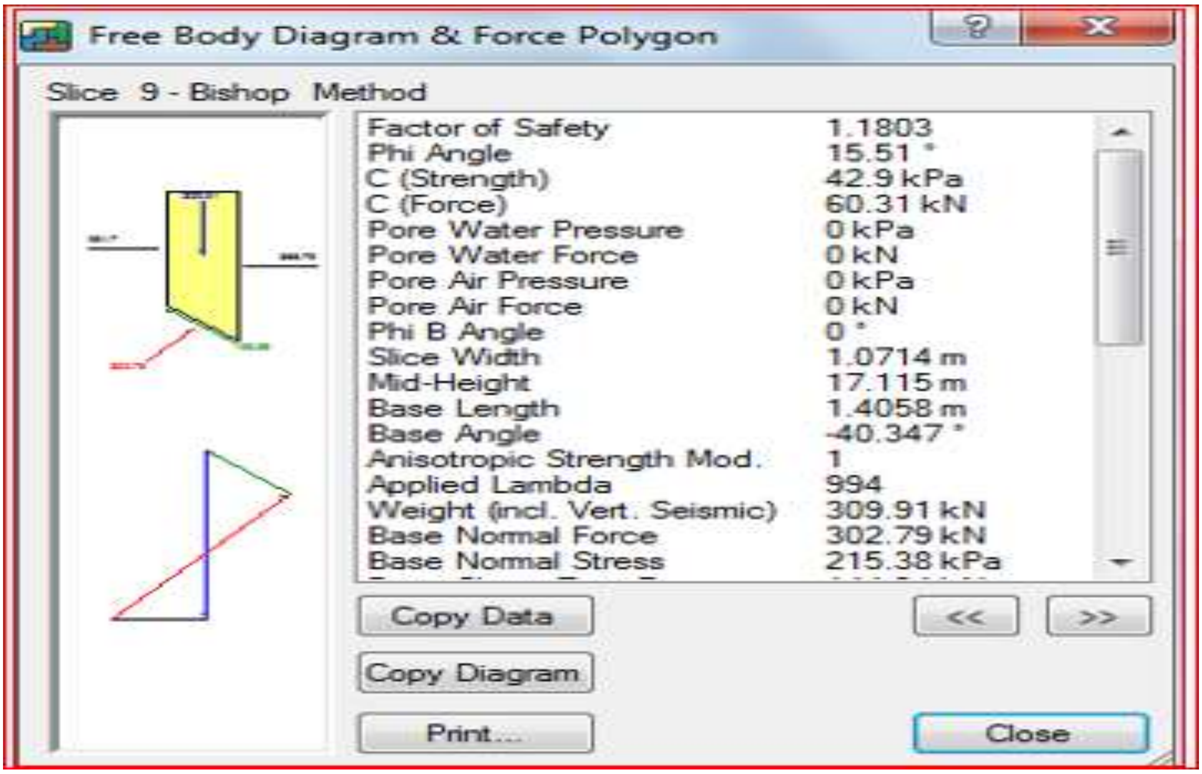

Figure 4.3 Stress acting on slice shaded fig above from Bishop Method

The units of the stresses given in the above figure are all newton per square meter (N/m^2). The width of this slice is 1.07 m, and the middle height is 17.115 m. The base length is 1.4m, and the base angle is -40.347degree.

From slope stability analysis of the nine failed slope profiles, the resisting force/moment and activating force/moment are summarized in Appendix B and the factor of safety of the slopes are indicated below in table 4.7.

Table 4.7 -- FOS values after geoslope program By Using Moist Unit Weight

Failed slope code	Methods	Factor of safety(Fos)	Status
A-1-01	Morgenstern	1.1	Marginally Stable
	Bishop	1.13	Marginally Stable
	Janbu	1.0	Failed
	ordinary	1.14	Marginally Stable
A-1-02	Morgenstern	1.05	failed
	Bishop	1.18	Marginally Stable
	Janbu	1.22	Marginally Stable
	ordinary	0.853	failed
K	Morgenstern	0.492	Failed
	Bishop	0.511	Failed
	Janbu	0.49	Failed
	Ordinary	0.476	Failed
As-1-01	Morgenstern	0.94	Failed
	Bishop	0.693	Failed
	Janbu	0.73	Failed
	Ordinary	0.708	Failed
As-1-02	Morgenstern	0.987	Failed
	Bishop	0.872	Failed
	Janbu	0.912	Failed
	Ordinary	0.86	Failed
As-1-03	Morgenstern	0.672	Failed
	Bishop	0.556	Failed
	Janbu	0.604	Failed

	Ordinary	0.602	Failed
As-1-04	Morgenstern	0.944	Failed
	Bishop	0.322	Failed
	Janbu	0.316	Failed
	Ordinary	0.316	Failed
AF	Morgenstern	0.944	Failed
	Bishop	0.908	Failed
	Janbu	0.925	Failed
	Ordinary	0.93	Failed
SG	Morgenstern	1.22	Marginally stable
	Bishop	1.06	Failed
	Janbu	1.08	Failed
	Ordinary	1.03	Failed

From the above result, it can be seen that the factor of safety is less than unity for almost of the soil samples and it can be concluded that with the prevailing condition the cut slope is unstable which actually witnessed by the site situation.

4.3 Landslide Inventory

There are several indicators to recognize the presence of landslide in the study area during site investigation. Such as Gully erosion due to culvert, failed slopes, road side ditch scouring problems, blocked and damaged culvert due to road side induced gully erosion.in addition to this damaged earth retaining structures due to rock fall and observation of toppling and wage failures along the route. The measured size of the sliding of the area and damaged asphalt as well as score dimension of the road side are indicted in table below and in Appendix D.

Table 3.5 Damaged Road Side Shoulder Dimensions

	Road Shoulder							
	Location			Size (dimension)		Remark		
Sr No	Easting	Northing	Elevation	Length (m)	Width (m)	Upslope	Downslope	Status of road shoulder
1	591393	1509604	1458	17	1.75		Downslope	scouring problem
2	591384	1509498	1420	22	1.5		Downslope	scouring problem
3	593207	1509893	1179	1.4	0.6		Downslope	scouring problem
4	594775	1512281	1082	20	1		Downslope	scouring problem
5	596058	1513268	983	22	1.6		Downslope	scouring problem

Table 3.6 Dimension of damaged road section along the route

	Road Section					
	Location			Diameter (m)	Length (m)	Possible Cause
No	Easting	Northing	Elevation			
1	585046	1511478	2171	5.4	16	Limestone (Rockfall)
2	590921	1509742	1510	1.2	3.9	Adigrat Sandstone (Toppling Failure)
3	591278	1509302	1365	10	15	Adigrat Sandstone (Rockfall Failure)
4	591287	1509311	1367	3.4	2.8	Adigrat Sandstone (Rockfall Failure)
5	591301	1509315	1361	6	5.4	Adigrat Sandstone (Rockfall Failure)

4.4 Figure road side ditches and culverts scouring problems at the study area.

4.4 Factors Controlling Slope Stability and Erosion Problems along the Route

On the basis of field observation and the results obtained, the following factors were the causes of failure along the route.

4.4.1 Erosion Problem

The Agoro- shahgubi route is affected by erosions caused due to blockage of culverts which results sedimentation. Erosion along this route changes the geometry of the potentially unstable soil /rock mass. Due to the removal of material at the toe of a potential slide reduces the confining stress that stabilizes the slope. Localized erosion of joint filling material, or zones of weathered rock effectively decrease interlocking between adjacent rock blocks. Loss of such interlocking significantly reduces the rock mass shear strength. The resulting decrease in shear strength allows a previously stable rock mass to move causing slope failure. In addition, localized erosion also results in increasing permeability and infiltration thus affecting the stability of rock slope.

Most importantly erosion along the route has the following negative impacts. Rendering of roadway is endanger due to the creation of gullies, slope instability on drains ditches for both lined and unlined, Flooding of roadways , Destruction of pavement infrastructure i.e. Pavement, culvert and drainage.

Therefore for the above problems the following are taken as solutions, Maintenance of proper road section for good drainage, ensuring that ditches are properly lined to prevent erosion, Regular maintenance to keep ditches, drains and culverts clear, Inspection of culverts on regular basis, installing diversions at all drain and culvert where runoff velocity can cause erosion.

4.4.2 Method of Excavation

During construction of the road traditional excavation methods like explosives and mechanical excavations were used for hard rock slopes. Even when very carefully planned controlled blasts are carried out, high intensity short duration forces act on the rock mass. The excavation has disturbed the slopes in the area and make it unstable. Road construction inevitably decrease the site stability by(1) Adding weight to the slope in the embankment fill, (2) Steepening the slope on both cut and fill surfaces,(3) Removing support of the cut slope, and (4) Rerouting and concentrating road drainage water.

Dormant earth flows are reactivated by cutting grades through the toe slope, which removes down slope support. Exposed soil and parent material on steep eroded by rain fall**.** In addition to thi**s** drainage water routed in to potential or inactive slump-earthflow areas down slope of roads decrease their stability. Drainage into slope depressions and other wet areas also initiate shallow, rapid failure

4.4.3 Over Steepening of the Existing Slope

The Agoro –shahgubi road embankment was cut with slope height H of maximum of **30** m for failed slope code (A-1-01) and slope angle of very stepper which was over stepped and unsafe because most of the soil was covered by clayey silty with big boulders which can easily fail and loos its strength during rainy season.

Therefore, cutting the slope with slope angle of more than 45^0 was one of the major causes of the slope failure. The major cause of the cut slope failure is related to release of stress up on excavation. This includes undermining the toe of the slope and over steeping the slope angle. Therefore, cutting the slope with slope angle of more than 45^0 was one of the major causes of the slope failure.

4.4.4 Effects of Vegetation and Deforestation

The Agoro –shahgubi section of the Agulae Berahle road is covered by different types of vegetation like grasses, herbs, shrubs and trees up to the Adi-Billion section of the route but during road construction these trees were removed. These vegetation are used to stabilize the slope stability and reinforcement of the soil, provide strong interlocking network to hold unconsolidated materials together and prevent flow in addition to this they are very effective in removing water from the soil, thus increasing the shear strength. The loss or removal of slope vegetation can result in either increased rates of erosion or higher frequencies of slopefailure. Therefore deforestation was considered as one of the cause which aggravate land slide problem in the area especially the first two failures.

4.5 Design of Remedial Measures for the Failed Slope (Cut Slope Instability)

From the slope stability analysis it is clearly observed that the slope is unstable for the current condition. Almost all the limit equilibrium, except for the first and last sample codes A-1-02and SG (except Bishop) the factor of safety for the other samples is less thanunity. Therefore, before fatal landslide problem occurs in this route it is necessary to adopt suitable remedial measures.

As already discussed, the route is characterized by a big mass movement with toppling, wedge, circular/rotational mode of failure. In the stability analysis, it was clearly described that the effect of rainfall significantly contributes towards the instability of the slope and the behavior of the land and engineering material/colluvial soil present in the slope section also plays a great role. Such conditions with other human impacts can cause a disastrous problem if measures will not be taken as soon as possible.

Since the material of the slope section is colluvial soil, the remedial measures to be taken are different from those of the rock slopes. The best remedial measures to be taken are mentioned below by taking the different causative factors into account. To reiterate once more, the following are major counter measures for landslide preventions.

A. Control Works

Slope flattering and earth removal: since the slide material has occupied the road section during failure earth removal is necessary to open the road. Furthermore, slope flattening is performed starting from the top of the slide mass in order to reduce the surcharge load on the slip surface. Surface drainage: provide furrow ditch at the top of the crown to collect surface drainage out of the slide area.

B. Support Systems (Retaining Walls)

Construction of retaining wall along the problematic slopes of roads is a very common practice. Indiscriminate use of such retaining structure as a protective measure without estimating lateral earth pressure and the passive force required to stabilizing the slope causes un-necessary increase in cost. Beside its cost, using a retaining wall has been considered as effective to prevent movements (failure).

C. Cut and Fill Solution

In order to improve the stability of unstable or potentially unstable slopes, the profile of slopes should change by excavation or by filling at the toe of the slope. The main slope excavation remedial measures involve: Flattening the slope (reducing the slope angle), removing unstable or potentially unstable materials, incorporating benches in the slope, and Drainage Control. For analysis, the slope angle should change from steeper to flatter. For example **(**1H: 1V) to **(**1.5H: 1V, 2H: 1V and 3H: 1V**),**A decrease in slope height and slope angle increases the stability of the slope.

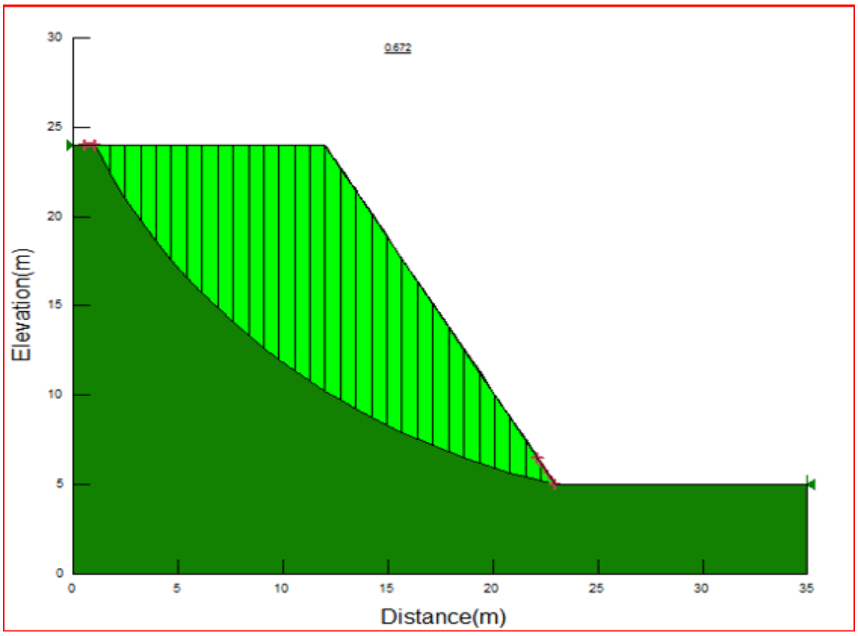

Figure 4.5 Most critical failure circle for Homogenous slide mass failed slope (before flattering)

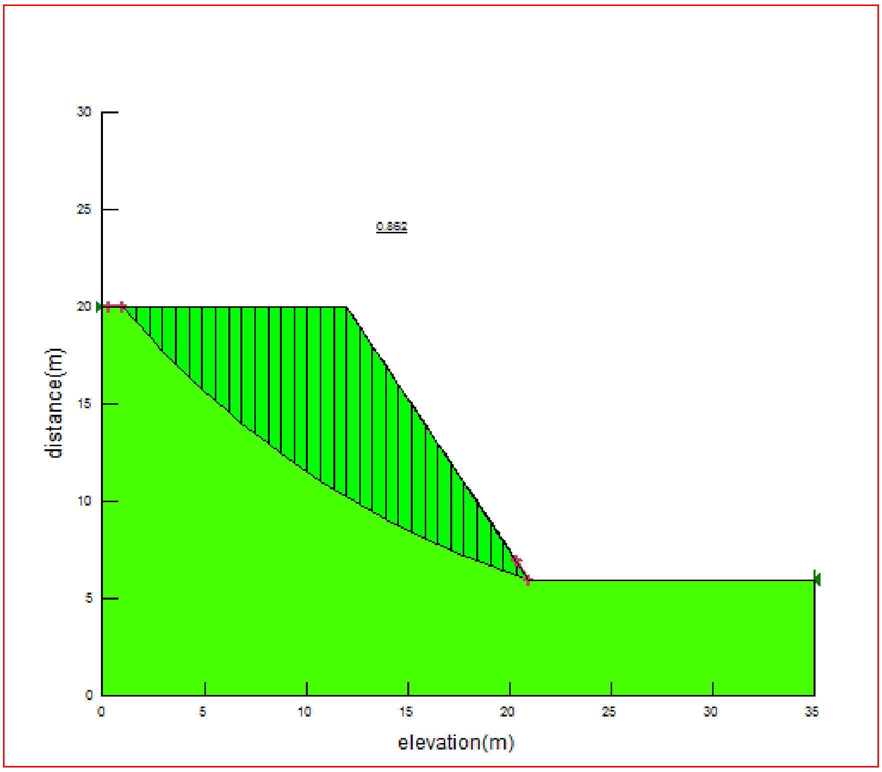

Figure 4.6 Most critical failure circle for Homogenous slide mass failed slope (after flattering (Slope changed from IV: 1H to 1V:1.5H)

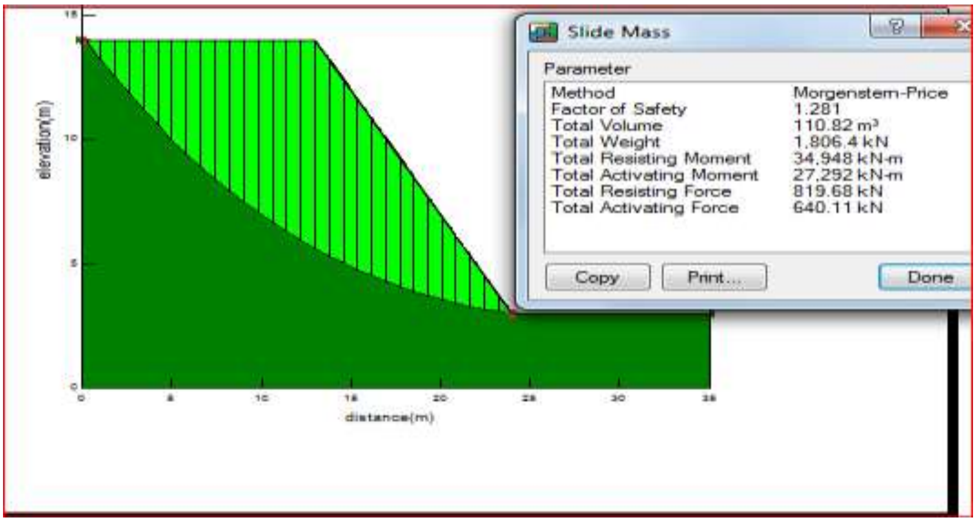

Figure 4.7 Most critical failure circle for Homogenous slide mass failed slope after flattering Slope changed from (IV: 1H to 1V:2H)

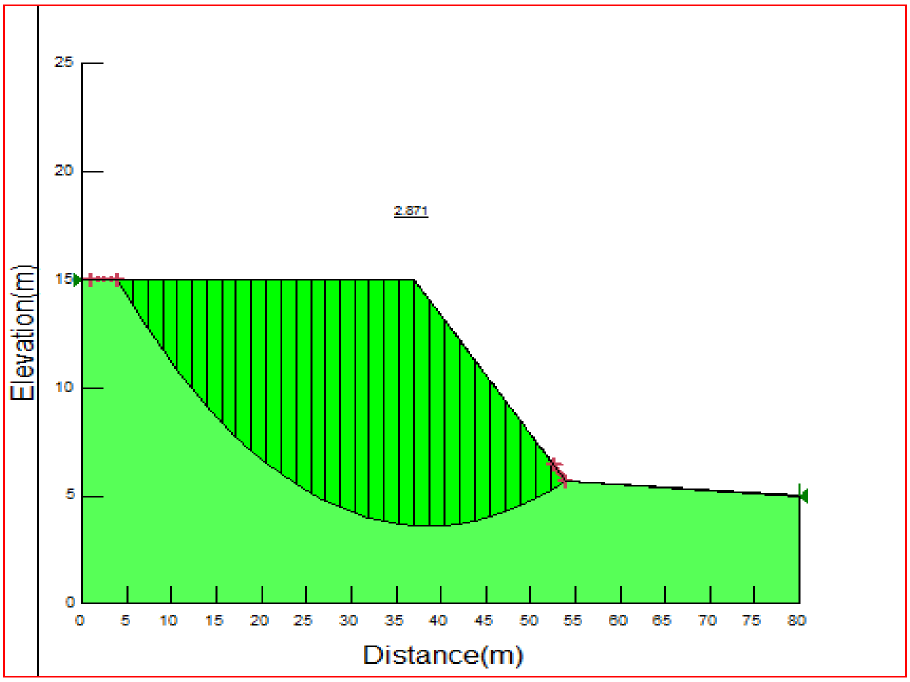

Figure 4.8 Most critical failure circle for Homogenous slide mass failed slope (after flattering slope angle changed from IV: 1H to 1V:3H)

Before flattering the slope the factor of safety was 0.55 but the factor safety increases to 0.67, 1.2 and 2.87 after changing the slope angle from 1V: 1H to 1.5H: 1V,1V:2H and 1V:3H. Slope angle can be graded into gentle ones; and if there is not enough room for such extensive grading, terraces or benches may be excavated into the slope.

D. Removing Unstable or Potentially Unstable Materials

Weak soils ,geologically fractured rocks and potentially unstable overhanging blocks was expected to be one of the cause for instability of the slope specially when they are in contact with water during rainy seasons this is why the stability of the blocks depends on the shear strength of the soil. Therefore, removing of such soils and rocks will stabilize the slope, even if it is a difficult task to remove all of them because lack of construction equipment's.

E. Incorporating Benches in the Slope

The purpose of benching in the slope is to transform the behavior of one high slope in to several lower ones. For this reason, the benches should be sufficiently wide. For the slope with cohesive and frictional strengths, the chief objective is to flatten the slope to control erosion and to establish vegetation.

F. Drainage Control

The presence of water in joints or in soil slope has a fundamental influence in slope stability. Hence the knowledge of the water pressure distribution constitutes a basic input data for stability analysis. Drainage improves slope stability in two important ways: (1) It reduces pore pressures within the soil, thereby increasing effective stress and shear strength; and (2) it reduces the driving forces of water pressures in cracks, thereby reducing the shear stress required for equilibrium .It clearly indicates the contribution of surface runoff and infiltration to aggravate the slope instability phenomenon. It has been inferred that efforts should be made to control the surface drainage and percolation of water in to problematic slopes for their stabilization.

The surface drainage control should be planned to involve: Reshaping the surface of the slope area to control flow and surface runoff, providing a flow line to divert undesirable surface flow into non-problem area and minimizing the removal of vegetation and establishing new vegetation growth. - Construction of toe drains to intercept the discharge and materials swept down by the flow.It has been noticed during the field studies and the stability analysis that most of the slope failures become active during rainy season on Agoro-shahgubi road.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1Conclusion

Cut slope instability is a major concern in the area of Agoro - Shahgubi where failures causes catastrophic destruction on the surrounding area. These destructions include failure of soil/rock masses, destruction of retaining walls and scouring of the road side ditches. The failures was triggered by Method of excavation, Increase in pore Water Pressure, over steepening of existing slope, Effects of deforestation and Erosion. These triggering factors were a cause for the imbalance of natural forces and brings internal changes. During the construction of the road was excavated by blasting and mechanical excavation and was a cause for the instability.

Applying Slope/W, it was easy to find out the position of the critical slip surface, safety map, and exact factor of safety. Cut Slope stability analysis was conducted for the moist conditions using 2D Limit equilibrium method using SLOPE/W. the safety factors calculated by slope/W utilizing the Morgenstern Price methods, Ordinary method, modified Bishop Method and Janbu method shows the slope is failed.

Results from this study indicates that the route Agoro-shahgubi are critical places from a slope stability point of view. Safety factors below the allowable limit have been obtained on these place. In order to minimize the above problems. Control works, Support Systems (Retaining walls), Cut and Fill solution, Removing Unstable or Potentially Unstable Materials, Benches incorporating in the Slope, and Drainage Control are remedial measures for the stated problem.

5.2Recommendation

In light of the above conclusions, following recommendations are made:-

- ✓ For the appropriate and economically justifiable solutions, relevant experts should be consulted to reduce time and catastrophic effects along the route.
- ✓ Benching of the slope should be provided in combination of drainage ditch in order to resist the sliding mass.
- ✓ Potentially Unstable and overhanging large boulders should be removed which is expected to fall and can cause slope instable, damage to the asphalt and passengers.
- ✓ Check dams should be incorporated to prevent erosion.
- ✓ Excavation should be provided properly.

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List of Appendices

Appendix A

Table A1.1 Atterberg limit test results for A-1-01

A-1-01						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	17	22	28	36		
Mass of Can (g)	17.90	17.60	17.80	19.10	10.80	10.80
Wet soil + Can (g)	66.70	65.80	69.10	67.80	13.40	14.20
Dry soil + Can (g)	58.00	57.60	60.70	60.10	13.18	13.88
Mass of Dry soil (g)	40.10	40.00	42.90	41.00	2.38	3.08
Mass of water (g)	8.70	8.20	8.40	7.70	0.22	0.32
Moisture content (%)	21.70	20.50	19.58	18.78	9.24	10.39
					10	

Liquid Limit, LL	21
Plastic Limit, PL	10
Plastic Index, PI	11

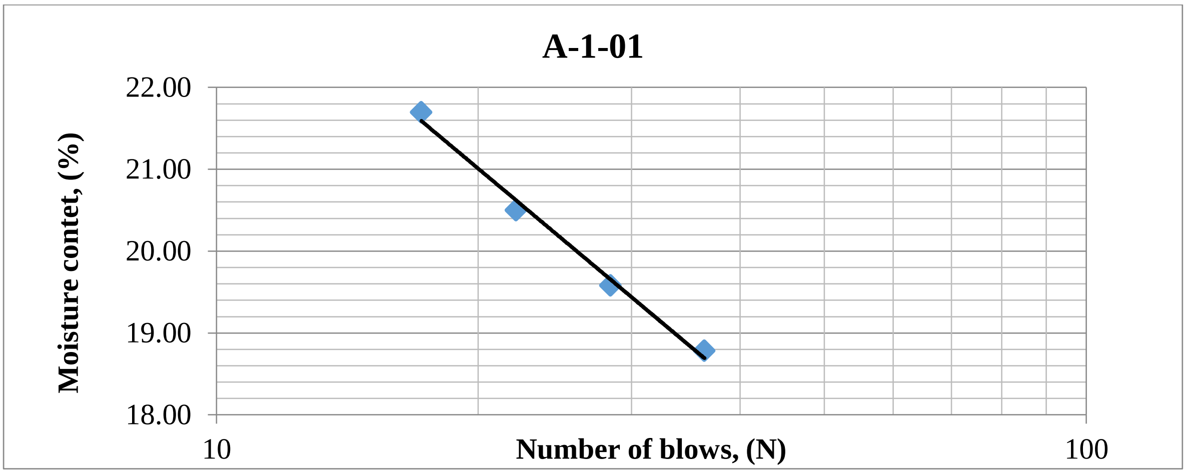


Figure 1.1 Number of bellows Vs moisture Content Graph

Table A1. 2 Atterberg limit test results for A-1-02

A-1-02						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	19	27	32	38		
Mass of Can (g)	16.90	16.60	16.80	18.10	10.80	10.80
Wet soil + Can (g)	67.70	66.80	70.10	68.80	13.40	14.20
Dry soil + Can (g)	57.40	56.70	59.70	59.40	13.12	13.92
Mass of Dry soil (g)	40.50	40.10	42.90	41.30	12.10	9.00
Mass of water (g)	10.30	10.10	10.40	9.40	0.28	0.28
Moisture content (%)	25.43	25.19	24.24	22.76	2.31	3.11
					2.71	

Liquid Limit	25.00
Plastic Limit	10
Plastic Index	15

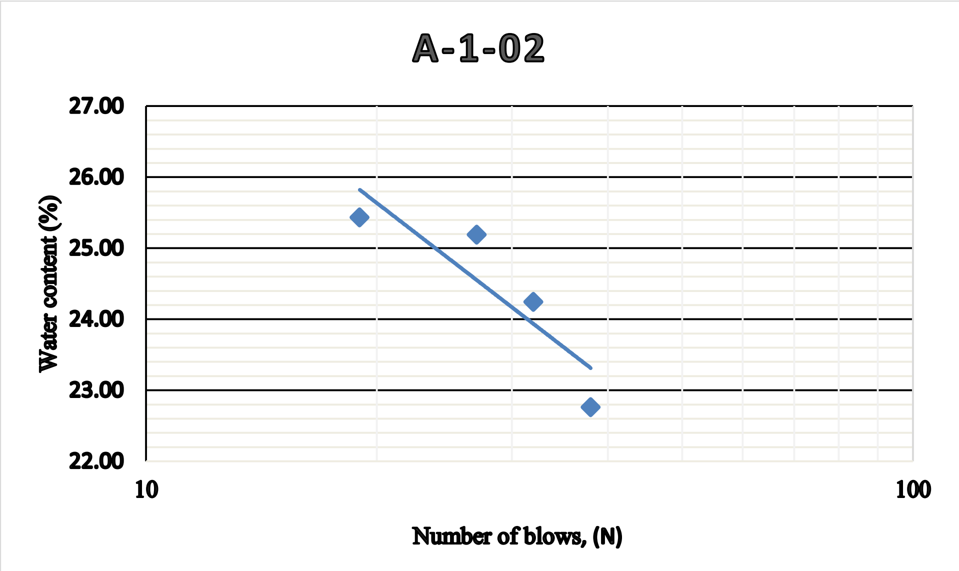


Figure A1.2 Number of bellows Vs. moisture Content Graph

Table A 1.3 Atterberg limit test results for K

K						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	38	32	27	19		
Mass of Can (g)	16.70	18.20	19.70	20.30	12.80	13.40
Wet soil + Can (g)	60.70	61.80	67.10	64.80	15.40	16.20
Dry soil + Can (g)	54.60	55.20	59.20	56.90	15.20	16.00
Mass of Dry soil (g)	37.90	37.00	39.50	36.60	11.10	9.60
Mass of water (g)	6.10	6.60	7.90	7.90	0.20	0.20
Moisture content (%)	16.09	17.84	20.00	21.58	1.80	2.08
					1.94	

Liquid Limit	16
Plastic Limit	16
Plastic Index	0.00

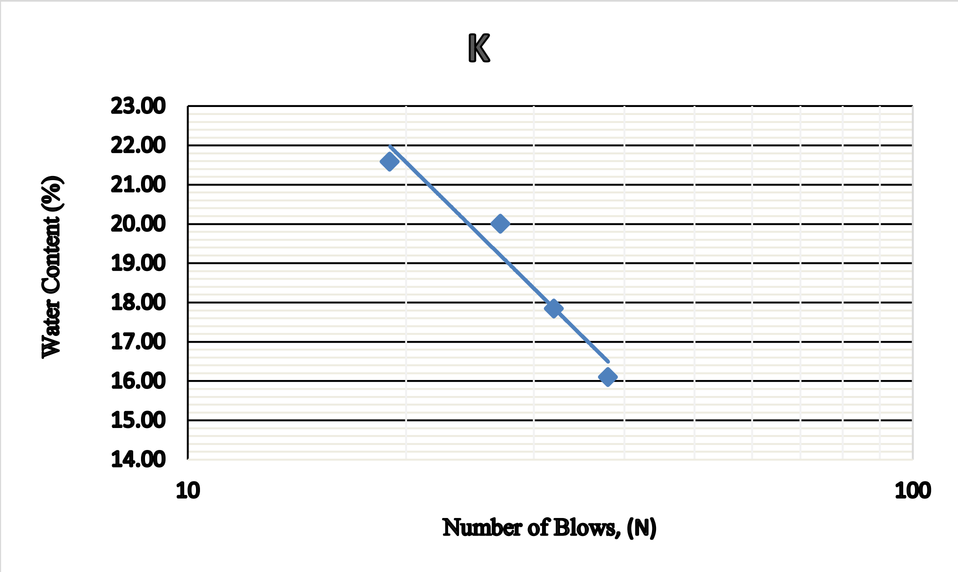


Figure A 1.3 Number of bellows vs. moisture Content Graph

Table A 1.4 Atterberg limit test results for AS-1-01

AS-1-01						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	19	27	32	38		
Mass of Can (g)	17.90	17.60	17.80	19.10	10.80	10.80
Wet soil + Can (g)	66.70	57.60	60.70	60.70	13.60	14.00
Dry soil + Can (g)	58.00	56.70	59.70	59.40	13.18	13.88
Mass of Dry soil (g)	40.10	39.10	24.30	40.30	17.60	3.90
Mass of water (g)	8.70	0.90	4.10	1.30	0.42	0.12
Moisture content (%)	21.70	20.50	19.58	18.78	2.39	3.08
					2.73	

Liquid limit	21
Plastic Limit	11
Plastic Index	10

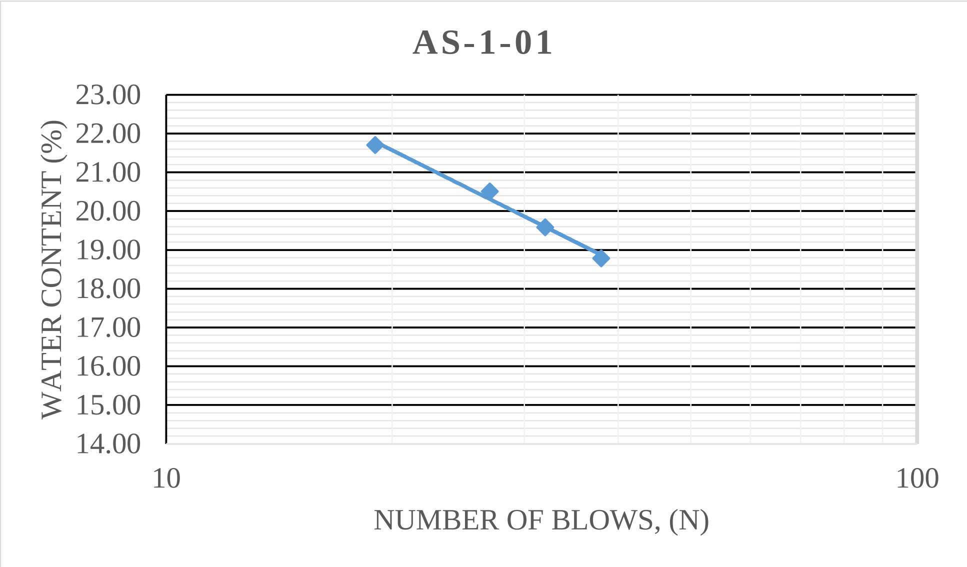


Figure A 1.4 Number of bellows vs. moisture Content Graph

Table A 1.5 Atterberg limit test results

As-1-02						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	19	27	32	38		
Mass of Can (g)	16.90	16.60	16.80	18.10	10.80	10.80
Wet soil + Can (g)	67.70	66.80	70.10	68.80	13.90	14.70
Dry soil + Can (g)	57.40	56.70	59.70	59.40	13.52	14.20
Mass of Dry soil (g)	40.50	40.10	42.90	41.30	14.00	14.70
Mass of water (g)	10.30	10.10	10.40	9.40	0.38	0.50
Moisture content (%)	25.43	25.19	24.24	22.76	2.71	3.40
					3.06	

Liquid Limit	25.00
Plastic Limit	14
Plastic Index	11

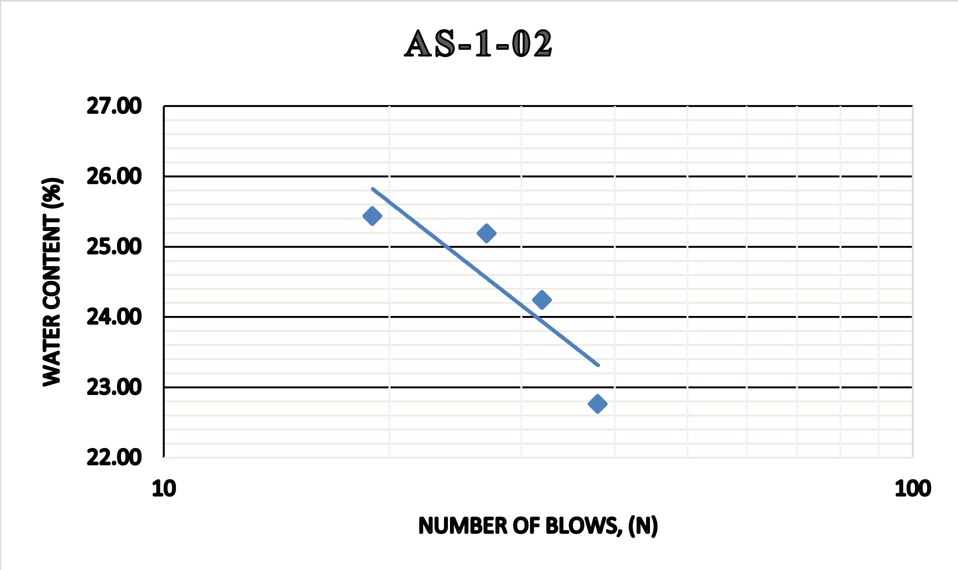


Figure A1.6 Number of bellows Vs. moisture Content Graph

Table A 1.5 Atterberg limit test results

AS-1-03						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M23	M32	B1	B2
No of blows	19	27	32	36		
Mass of Can (g)	16.70	18.20	19.70	20.30	12.50	13.40
Wet soil + Can (g)	60.70	61.80	67.10	64.80	13.40	14.20
Dry soil + Can (g)	52.40	53.70	58.70	57.40	13.31	14.13
Mass of Dry soil (g)	35.70	35.50	39.00	37.10	11.10	9.60
Mass of water (g)	8.30	8.10	8.40	7.40	0.09	0.07
Moisture content (%)	23.25	22.82	21.54	19.95	0.81	0.73
					0.77	

Liquid Limit	23
Plastic Limit	11
Plastic Index	12

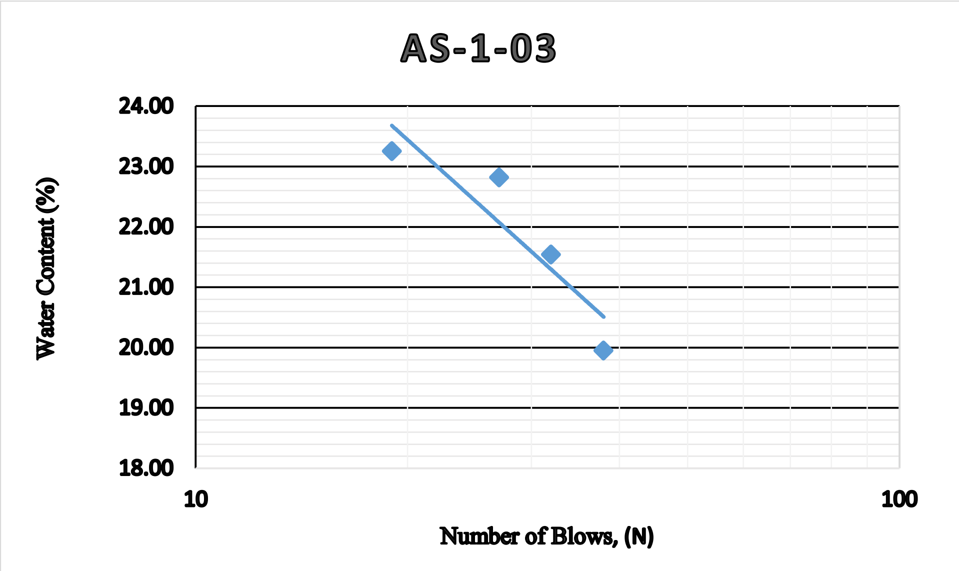


Figure A 1.7 Number of bellows vs. moisture Content Graph

Table A 1.6 Atterberg limit test results

AS-1-04						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	17	21	27	34		
Mass of Can (g)	16.70	19.80	17.40	22.50	12.80	13.40
Wet soil + Can (g)	56.40	65.80	45.80	55.60	15.40	16.20
Dry soil + Can (g)	50.00	58.60	41.70	51.10	15.20	16.00
Mass of Dry soil (g)	33.30	38.80	24.30	28.60	8.30	7.70
Mass of water (g)	6.40	7.20	4.10	4.50	0.20	0.20
Moisture content (%)	19.22	18.56	16.87	15.73	2.41	2.60
					2.50	

Liquid Limit	18
Plastic Limit	8.00
Plastic Index	10

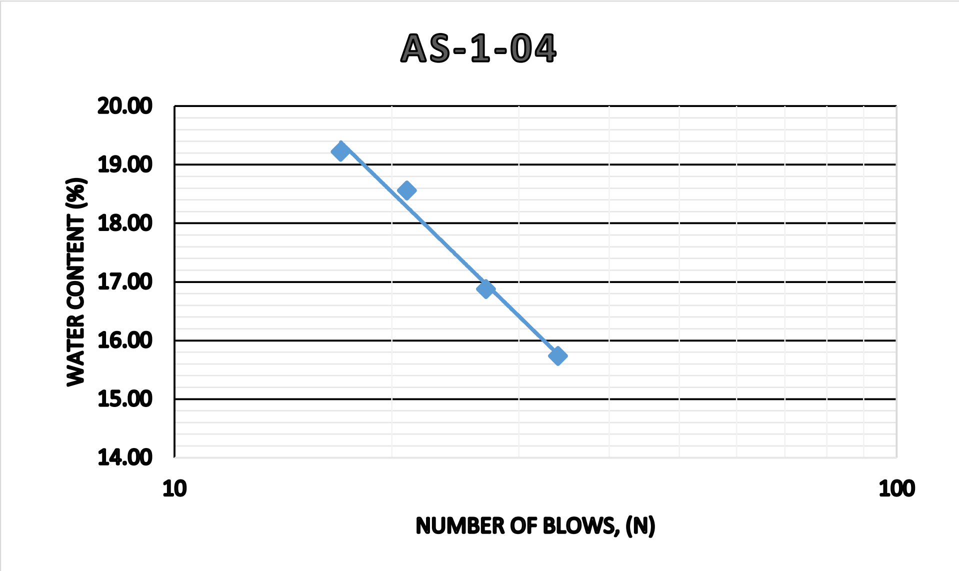


Figure A 1.8 Number of bellows Vs moisture Content Graph

Table A1.7 Atterberg limit test results

AF						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	17	21	27	34		
Mass of Can (g)	16.70	19.80	17.40	22.50	12.80	13.40
Wet soil + Can (g)	56.40	65.80	45.80	55.60	15.40	16.20
Dry soil + Can (g)	50.00	58.60	41.70	51.10	15.20	16.00
Mass of Dry soil (g)	33.30	38.80	24.30	28.60	8.30	7.70
Mass of water (g)	6.40	7.20	4.10	4.50	0.20	0.20
Moisture content (%)	19.22	18.56	16.87	15.73	2.41	2.60
					2.50	

Liquid Limit	18
Plastic limit	8.00
Plastic Index	10

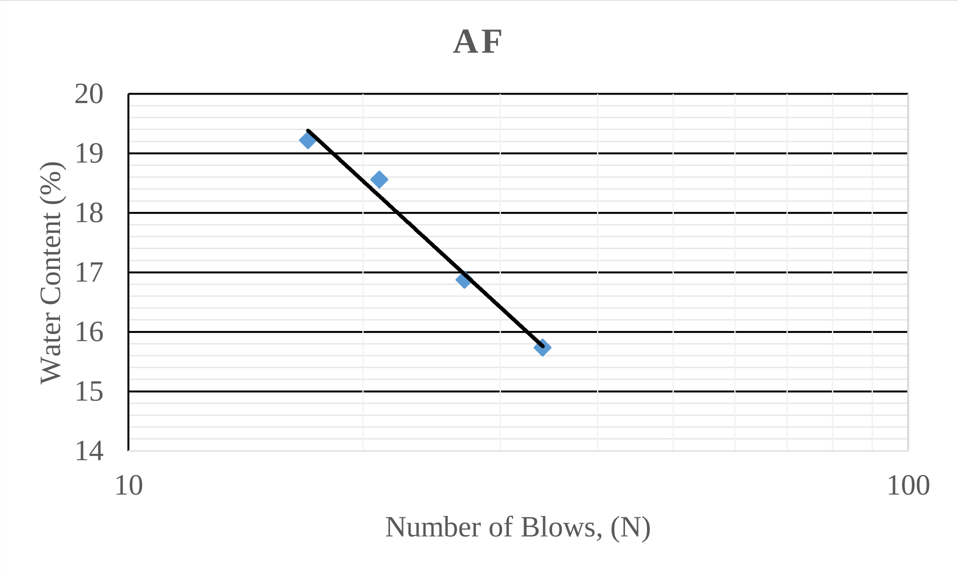


Figure A1.9 Number of bellows Vs moisture Content Graph

Table A1.8 Atterberg limit test results

SG						
	Liquid Limit (LL)				**Plastic Limit (PL)**	
Trial	1	2	3	4	1	2
Can No	M12	M11	M32	M21	B1	B2
No of blows	19	27	32	38		
Mass of Can (g)	17.00	18.60	19.80	19.50	10.80	10.80
Wet soil + Can (g)	56.70	55.00	58.10	57.20	13.40	14.20
Dry soil + Can (g)	48.50	47.90	50.90	50.40	13.00	13.75
Mass of Dry soil (g)	31.50	29.30	31.10	30.90	2.20	2.95
Mass of water (g)	8.20	7.10	7.20	6.80	0.40	0.45
Moisture content (%)	26.03	24.23	23.15	22.01	18.18	15.25
					16.72	

Liquid Limit	25
Plastic Limit	17
Plastic Index	8

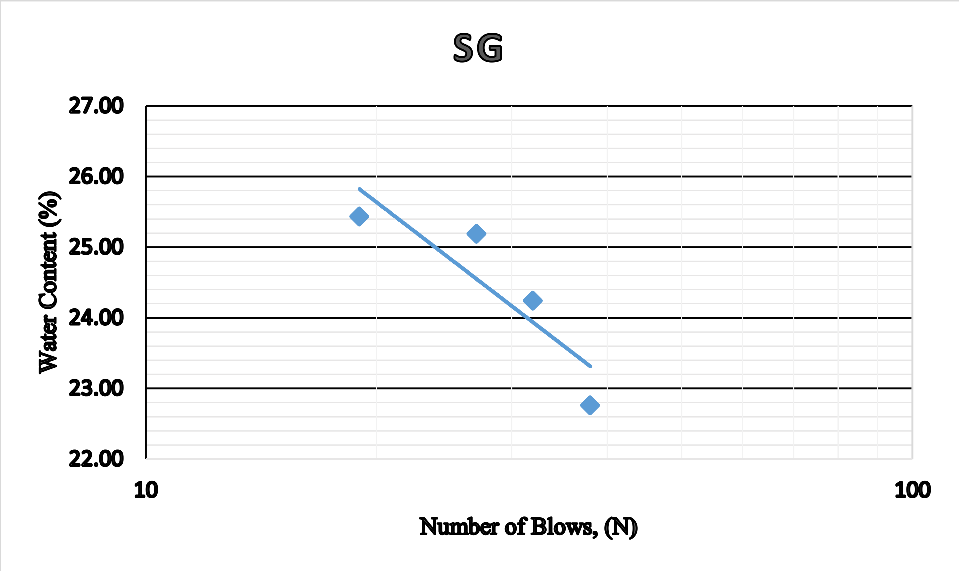


Figure A1.10 Number of bellows vs. moisture Content Graph

Table a 3.1 Specific Gravity for All Samples taken From the Failed Slope

No	Description	Sample code								
		A1-01	A1-02	K	AS-1-01	AS-1-02	AS-1-03	AS-1-04	AF	SG
1	Pycnometer no.	4	2	4	2	4	4	4	4	2
2	Wt. of dry soil (Ws) gm.	25	25	25	25	25	25	25	25	25
3	Wt. of Pycnometer + water + soil (W1).gm.	179	178	180	177	177	178	181	178	179
4	Wt. of pycnometer + water (W2) gm.	164	162	164	162	162	163	165	162	163
5	Temperature ($^{o}{}_{c}$)	24	24	24	24	24	24	24	24	24
6	Specific gravity of soil @Test To	2.551	2.78	2.8	2.6	2.5	2.6	2.68	2.7	2.7
7	specific gravity of soil @ 20 ^{o}c	2.6	2.7	2.8	2.6	2.5	2.6	2.7	2.7	2.7
8	average specific gravity	2.55	2.78	2.8	2.6	2.5	2.6	2.69	2.7	2.68

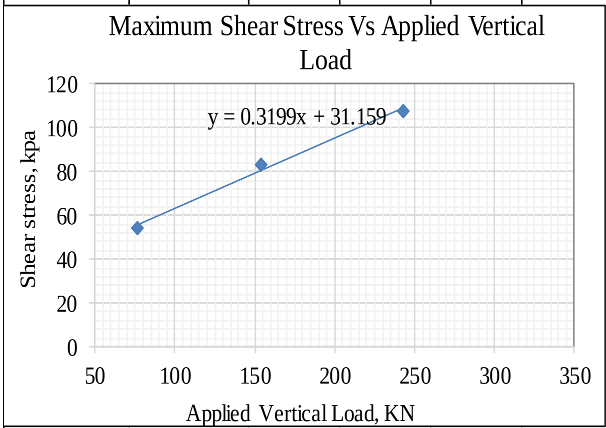
Table A.3.2 Summary of Laboratory Test Results

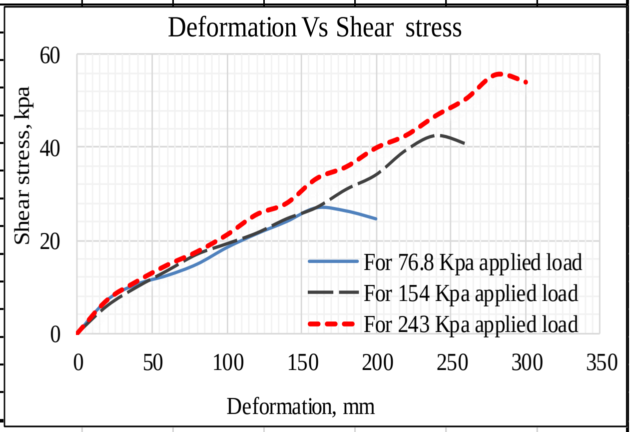
Soil parameters	Sample code								
	A-1-01	**A-1-02**	**K**	**AS-1-01**	**AS-1-02**	**AS-01-03**	**AS-1-04**	**SG**	**AF**
Moisture content (%)	20.14	24.4	17.6	20.14	24.4	21.9	17.6	23.86	18.9
Wet Unit weight (KN/m^3)	16.5	16.9	15	16.8	16.2	16.3	16.2	16.8	18.9
Dry Unit weight (KN/m^3)	15.22	15.5	14.9	15.34	14.5	14.9	14.9	15.16	14.2
Liquid limit (%)	20.3	25	16.2	20.3	25	22.6	17.5	17.5	16.2
Plastic limit (%)	9.8	10.6	16.2	10.8	14.4	10.4	8.0	9.5	6.7
Plasticity index (%)	10.5	14.4	0	9.5	10.6	12.2	9.5	8.0	9.5
Angle of internal friction (ϕ)	17.79	15.51	12.94	12.23	11.89	15.19	12.2	16.9	12.2
Cohesion(C) KN/m^2	31	43	13	32	37	17	1.8	38	31

Appendix B

Table B1.1 Direct shear results for A-1-01

Sample Code.		A-1-01			Sample Depth, m:		9.0m					
					Ring Calib. Factor:		17.66 N/div		Wet unit weight, kN/M^3:			16.50
Length of sample :		60 mm			Rate of strain :		0.8 mm/min		Dry Unit Weight, kN/M^3:			15.22
Width of sample:		35 mm			Moisture content, %		20.1		Sample Condition:		Molded	
				Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
				76.8 kPa			154 kPa			243 kPa		
Horizontal Displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	5.5	15.0144	7.19079	4.5	12.2845	5.88337	5.5	15.0144	7.19079
40	0.4	1.6	2076	8	21.9653	10.5806	7.5	20.5925	9.91931	8.5	23.3382	11.2419
60	0.6	2.4	2064	9.3	25.6831	12.4434	10	27.6163	13.38	11	30.3779	14.718
80	0.8	3.2	2052	11	30.5556	14.8906	12.5	34.7222	16.9212	13	36.1111	17.598
100	1	4	2040	13.5	37.7206	18.4905	14	39.1176	19.1753	15.5	43.3088	21.2298
120	1.2	4.8	2028	15.5	43.5651	21.4818	15.5	43.5651	21.4818	18.5	51.997	25.6396
140	1.4	5.6	2016	17.2	48.631	24.1225	17.5	49.4792	24.5432	20	56.5476	28.0494
160	1.6	6.4	2004	19.1	54.3263	27.109	19	54.0419	26.967	23.5	66.8413	33.354
180	1.8	7.2	1992	18.4	52.6506	26.431	21.5	61.5211	30.8841	25	71.5361	35.9117
200	2	8	1980	17	48.9394	24.7169	23.4	67.3636	34.022	27.5	79.1667	39.9832
220	2.2	8.8	1968				26.7	77.3323	39.2949	29	83.9939	42.6798
240	2.4	9.6	1956				28.5	83.0521	42.4602	31.5	91.7945	46.9297
260	2.6	10.4	1944				27	79.1667	40.7236	33.5	98.2253	50.5274
280	2.8	11.2	1932							36.5	107.686	55.7383
300	3	12	1920							35	103.906	54.1178
320	3.2	12.8								33		
340	3.4	13.6										
360	3.6	14.4										
380	3.8	15.2										
400	4	16										

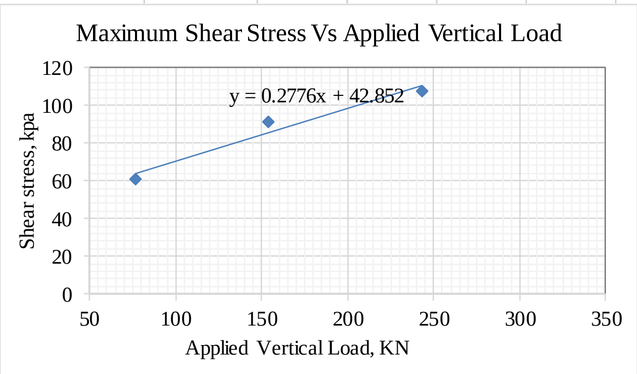


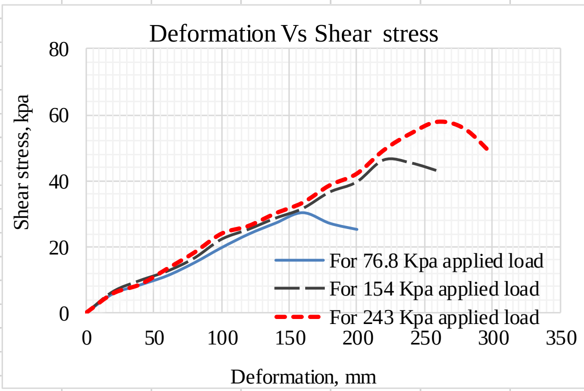


Angle of internal friction, ϕ =	17.79	Cohesion , C (kN/m^2) =	31.16

Table B1.1 Direct shear results for A-1-02

Sample Code.		**A-1-02**		Sample Depth, m:			**8.00**					
				Ring Calib. Factor:			**17.66 N/div**		Wet unit weight, kN/M^3:			**16.90**
Length of sample :			**60 mm**		Rate of strain :		**0.8 mm/min**		Dry Unit Weight, kN/M^3:			**15.50**
Width of sample:			**35 mm**		Moisture content, %		**24.4**		Sample Condition:			**Molded**
				Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
				76.8 kPa			**154**			**243 kPa**		
Horizontal Displacement [mm]	actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	4.5	12.2845	5.88337	5	13.6494	6.53708	4.5	12.2845	5.88337
40	0.4	1.6	2076	6.5	17.8468	8.59673	7.5	20.5925	9.91931	6.5	17.8468	8.59673
60	0.6	2.4	2064	8.5	23.4738	11.373	9.5	26.2355	12.711	10	27.6163	13.38
80	0.8	3.2	2052	11.3	31.3889	15.2967	12.3	34.1667	16.6504	13.5	37.5	18.2749
100	1	4	2040	14.5	40.5147	19.8601	16.3	45.5441	22.3255	17.5	48.8971	23.9691
120	1.2	4.8	2028	17.3	48.6243	23.9765	18.3	51.4349	25.3624	19	53.4024	26.3325
140	1.4	5.6	2016	19.5	55.1339	27.3482	20.5	57.9613	28.7506	21.5	60.7887	30.1531
160	1.6	6.4	2004	21.5	61.1527	30.5153	22.3	63.4281	31.6508	23.5	66.8413	33.354
180	1.8	7.2	1992	19	54.3675	27.2929	25.5	72.9669	36.63	26.9	76.9729	38.641
200	2	8	1980	17.5	50.3788	25.4438	27.3	78.5909	39.6924	29	83.4848	42.1641
220	2.2	8.8	1968				31.5	91.2348	46.3591	33.5	97.0274	49.3026
240	2.4	9.6	1956				30.5	88.8804	45.4399	36.5	106.365	54.3789
260	2.6	10.4	1944				28.5	83.5648	42.986	38.4	112.593	57.918
280	2.8	11.2	1932							36.5	107.686	55.7383
300	3	12	1920							31.1	92.3281	48.0876
320	3.2	12.8	1908									
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									

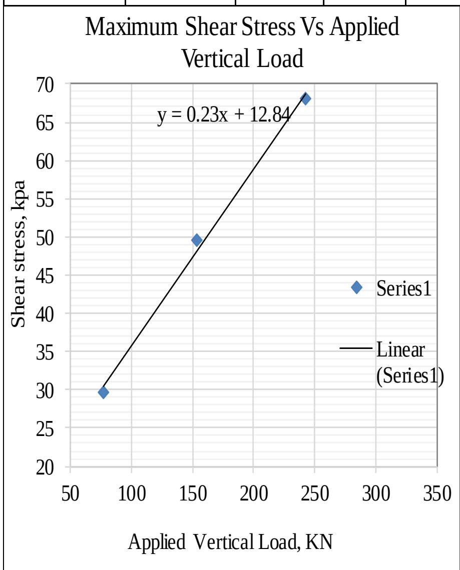


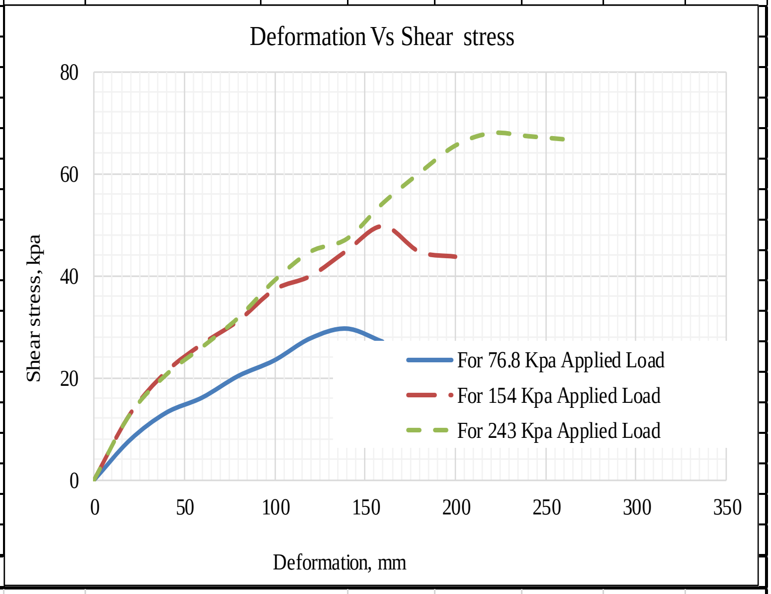


Angle of internal friction, ϕ =	**15.51**	**Cohesion , C (kN/m^2) =**	**42.85**

Table B1.1 Direct shear results for K

Sample Code.		K			Sample Depth, m:		6.40					
					Ring Calib. Factor:		17.66 N/div		Wet unit weight, kN/M^3:			16.50
Length of sample :		60 mm			Rate of strain :		0.8 mm/min		Dry Unit Weight, kN/M^3:			15.22
Width of sample:		35 mm			Moisture content, %		17.6		Sample Condition:		molded	
				Applied Vertical Stress 76.8 kPa			Applied Vertical Stress 154 kPa			Applied Vertical Stress 243 kPa		
Horizontal Displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	3	16.5	7.902298851	5	27	12.931034	5	27	12.93103
40	0.4	1.6	2076	5	27.5	13.24662813	8	44	21.194605	8	43	20.71291
60	0.6	2.4	2064	6	33.5	16.23062016	10	55	26.647287	10	54	26.16279
80	0.8	3.2	2052	7.5	42	20.46783626	11.5	64	31.189084	12	65.5	31.92008
100	1	4	2040	8.5	48	23.52941176	13.6	76	37.254902	14.5	80	39.21569
120	1.2	4.8	2028	10	56.5	27.85996055	14.5	81	39.940828	16.5	91	44.87179
140	1.4	5.6	2016	10.5	60	29.76190476	16	90.5	44.890873	17	95.5	47.37103
160	1.6	6.4	2004	9.5	54.49902	27.19512123	17.5	99.5	49.650699	19.5	109	54.39122
180	1.8	7.2	1992				15.5	89	44.678715	21.5	120	60.24096
200	2	8	1980				15	86.5	43.686869	23	130	65.65657
220	2.2	8.8	1968							23.5	134	68.08943
240	2.4	9.6	1956							23	132	67.48466
260	2.6	10.4	1944							22.5	130	66.87243
280	2.8	11.2	1932									
300	3	12	1920									
320	3.2	12.8	1908									
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									



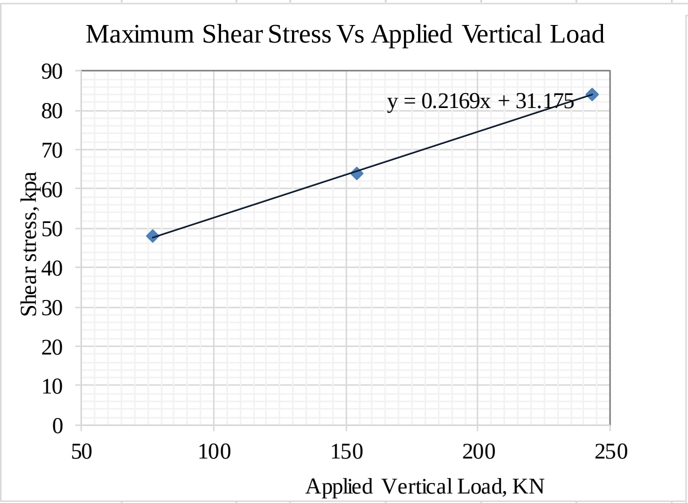


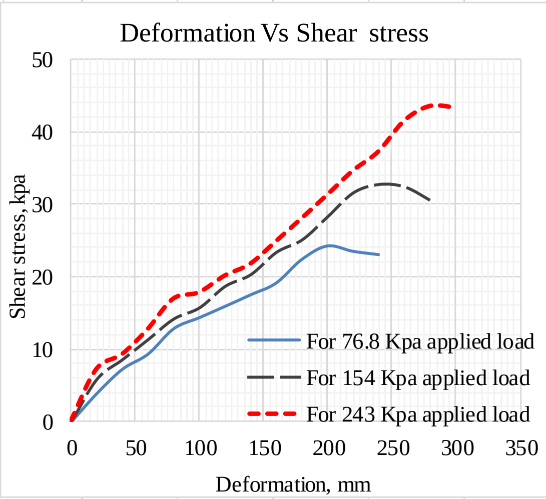
Angle of internal friction, =	ϕ	12.90	Cohesion , C (kN/m^2) =	13.06	

Table B1.1 Direct shear results for AF

Sample Code.	**AF**	Sample Depth, m:	**7.00**		
		Ring Calib. Factor:	**17.66 N/div**	Wet unit weight, kN/M^3:	**16.00**
Length of sample :	**60 mm**	Rate of strain :	**0.8 mm/min**	Dry Unit Weight, kN/M^3:	**14.20**
Width of sample:	**35 mm**	Moisture content, %	**18.9**	Sample Condition:	**Molded**

				Applied Vertical Stress **76.8 kPa**			Applied Vertical Stress **154 kPa**			Applied Vertical Stress **243 kPa**		
Horizontal Displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	2	8.18966	3.92225	4.5	12.2845	5.88337	5.5	15.0144	7.19079
40	0.4	1.6	2076	3	15.1012	7.27416	6.5	17.8468	8.59673	7	19.2197	9.25802
60	0.6	2.4	2064	3.5	19.3314	9.36599	8	23.4738	11.373	9	26.2355	12.711
80	0.8	3.2	2052	4	26.3889	12.8601	9.5	29.1667	14.2138	11.5	34.7222	16.9212
100	1	4	2040	4.5	29.3382	14.3815	11	32.1324	15.7512	13	36.3235	17.8057
120	1.2	4.8	2028	5	32.3225	15.9381	12.5	37.9438	18.71	15.5	40.7544	20.0959
140	1.4	5.6	2016	7.5	35.3423	17.5309	13	40.997	20.3358	17	43.8244	21.7383
160	1.6	6.4	2004	6.5	38.3982	19.1608	13.5	46.9311	23.4187	18.5	49.7754	24.838
180	1.8	7.2	1992	6	44.6386	22.4089	13	50.0753	25.1382	20.5	55.7982	28.0111
200	2	8	1980		48.0758	24.2807	12.5	56.1364	28.3517	21.5	61.8939	31.2596
220	2.2	8.8	1968		46.3415	23.5475		62.2713	31.6419	20.5	68.064	34.5854
240	2.4	9.6	1956		45.1687	23.0924		64.1104	32.7763	20	72.8528	37.2458
260	2.6	10	1944					63.0401	32.428		80.6327	41.4777
280	2.8	11	1932					59.0062	30.5415		84.0839	43.5217
300	3	12	1920								83.125	43.2943
320	3.2	13	1908									
340	3.4	14	1896									
360	3.6	14	1884									
380	3.8	15	1872									
400	4	16	1860									



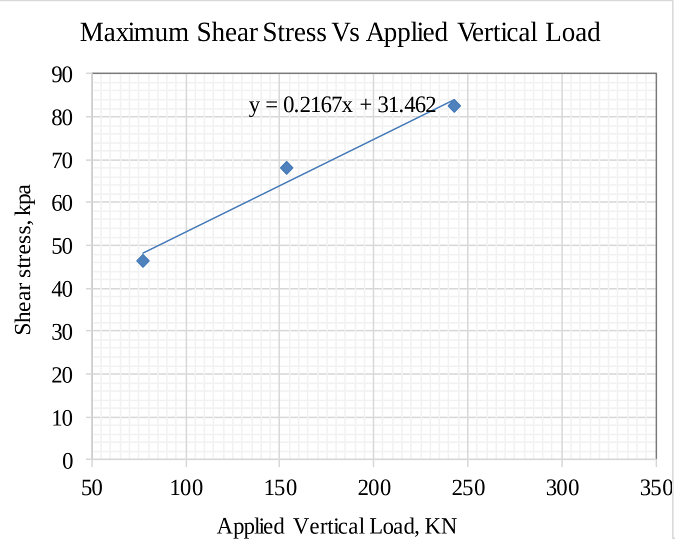


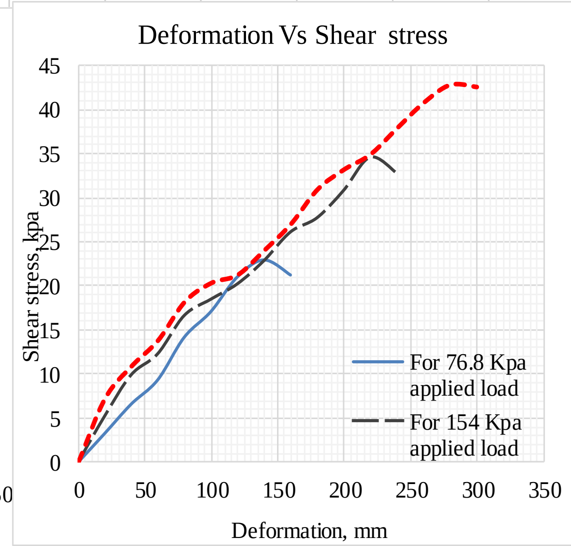
Angle of internal friction, ϕ =	**12.24**	**Cohesion , C (kN/m^2) =**	**31.18**

Table B1.1 Direct shear results for AS-1-01

Sample Code.	**AS-1-01**	Sample Depth, m:	**6.5m**		
		Ring Calib. Factor:	**17.66 N/div**	Wet unit weight, kN/M^3:	**16.80**
Length of sample :	**60 mm**	Rate of strain :	**0.8 mm/min**	Dry Unit Weight, kN/M^3:	**15.34**
Width of sample:	**35 mm**	Moisture content, %	**20.1**	Sample Condition:	**Molded**

				Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
				76.8 kPa			**154 kPa**			**243 kPa**		
Horizontal isplacemen [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	2.5	6.82471	3.26854	4	10.9195	5.22966	5.5	15.0144	7.19079
40	0.4	1.6	2076	5	13.7283	6.61287	7.5	20.5925	9.91931	8.2	22.5145	10.8451
60	0.6	2.4	2064	7	19.3314	9.36599	9.2	25.407	12.3096	10.3	28.4448	13.7814
80	0.8	3.2	2052	10.5	29.1667	14.2138	12.3	34.1667	16.6504	13.4	37.2222	18.1395
100	1	4	2040	12.5	34.9265	17.1208	13.5	37.7206	18.4905	14.8	41.3529	20.271
120	1.2	4.8	2028	15.2	42.7219	21.066	14.6	41.0355	20.2345	15.3	43.003	21.2046
140	1.4	5.6	2016	16.4	46.369	23.0005	16.3	46.0863	22.8603	17.1	48.3482	23.9822
160	1.6	6.4	2004	15	42.6647	21.2898	18.4	52.3353	26.1154	19	54.0419	26.967
180	1.8	7.2	1992				19.3	55.2259	27.7238	21.5	61.5211	30.8841
200	2	8	1980				21.2	61.0303	30.8234	22.8	65.6364	33.1497
220	2.2	8.8	1968				23.5	68.064	34.5854	23.7	68.6433	34.8797
240	2.4	9.6	1956				22	64.1104	32.7763	25.4	74.0184	37.8417
260	2.6	10.4	1944							27	79.1667	40.7236
280	2.8	11.2	1932							28	82.6087	42.7581
300	3	12	1920							27.5	81.6406	42.5212
320	3.2	12.8	1908							26		
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									

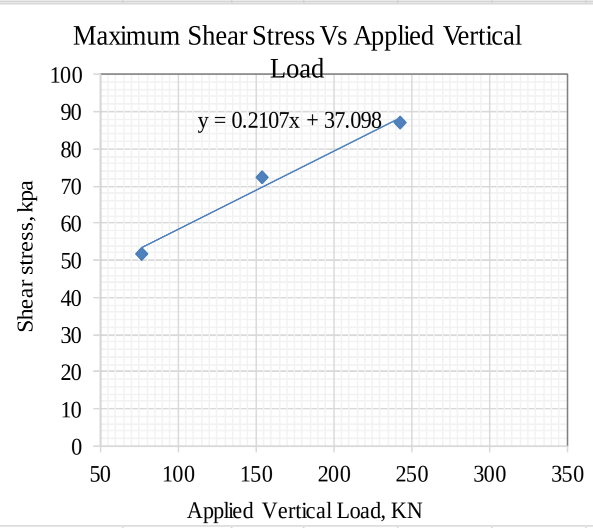


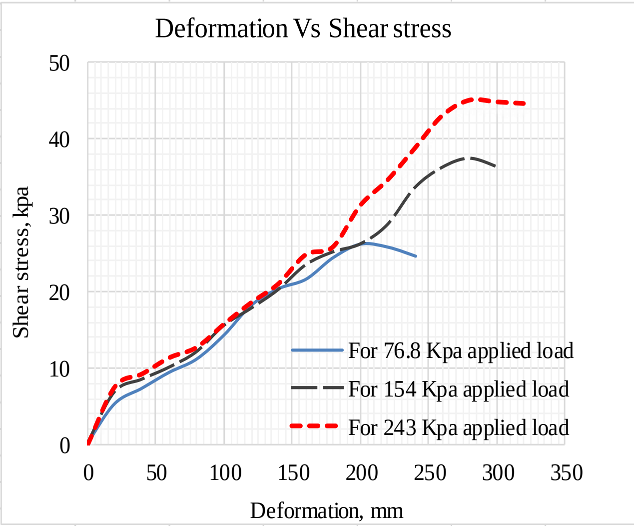


Angle of internal friction, ϕ = 12.23   Cohesion , C (kN/m^2) = 31.46

Table B1.1 Direct shear results for AS-1-02

Sample Code.		**AS-1-02**			Sample Depth, m:		**5.5m**					
					Ring Calib. Factor:		**17.66 N/div**		Wet unit weight, kN/M^3:			**16.20**
Length of sample :		**60 mm**			Rate of strain :		**0.8 mm/min**		Dry Unit Weight, kN/M^3:			**14.50**
Width of sample:		**35 mm**			Moisture content, %		**24.4**		Sample Condition:		**Molded**	
				Applied Vertical Stress **76.8 kPa**			Applied Vertical Stress **154 kPa**			Applied Vertical Stress **243 kPa**		
Horizontal Displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	4	10.9195	5.22966	5.2	14.1954	6.79856	5.8	15.8333	7.58301
40	0.4	1.6	2076	5.5	15.1012	7.27416	6.4	17.5723	8.46448	7	19.2197	9.25802
60	0.6	2.4	2064	7	19.3314	9.36599	7.5	20.7122	10.035	8.5	23.4738	11.373
80	0.8	3.2	2052	8.2	22.7778	11.1003	8.9	24.7222	12.0479	9.4	26.1111	12.7247
100	1	4	2040	10.4	29.0588	14.2445	11.3	31.5735	15.4772	11.5	32.1324	15.7512
120	1.2	4.8	2028	13.1	36.8195	18.1556	12.8	35.9763	17.7398	13.4	37.6627	18.5714
140	1.4	5.6	2016	14.5	40.997	20.3358	14.4	40.7143	20.1956	15	42.4107	21.0371
160	1.6	6.4	2004	15.2	43.2335	21.5736	16.5	46.9311	23.4187	17.5	49.7754	24.838
180	1.8	7.2	1992	17	48.6446	24.42	17.5	50.0753	25.1382	18	51.506	25.8564
200	2	8	1980	18	51.8182	26.1708	18	51.8182	26.1708	21.5	61.8939	31.2596
220	2.2	8.8	1968	17.5	50.686	25.7551	19.5	56.4787	28.6985	23.5	68.064	34.5854
240	2.4	9.6	1956	16.5	48.0828	24.5822	22.5	65.5675	33.5212	26	75.7669	38.7356
260	2.6	10.4	1944				24	70.3704	36.1988	28.5	83.5648	42.986
280	2.8	11.2	1932				24.5	72.2826	37.4134	29.5	87.0342	45.0487
300	3	12	1920				23.5	69.7656	36.3363	29	86.0938	44.8405
320	3.2	12.8	1908							28.5	85.1415	44.6234
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									

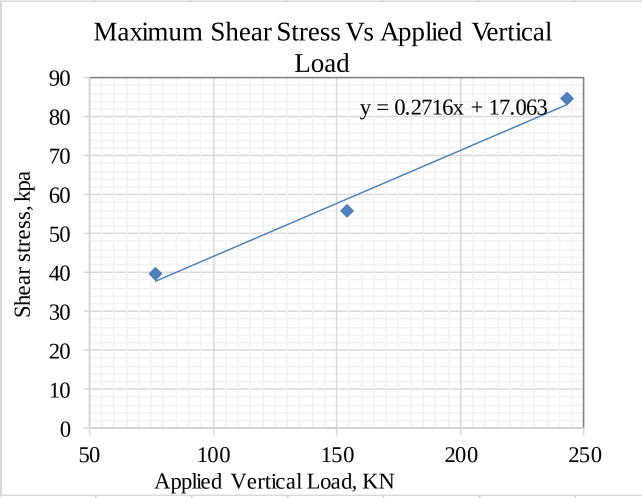


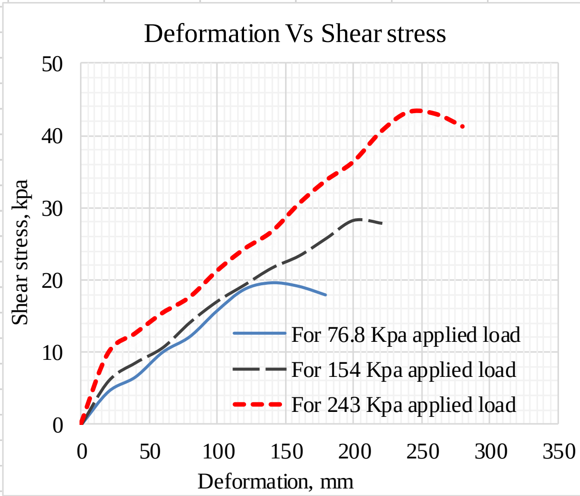


Angle of internal friction, ϕ =		11.89		Cohesion , C (kN/m^2) =		37.10

Table B1.1 Direct shear results for AS-1-03

Sample Code.		**AS-1-03**			Sample Depth, m:		**8.00**					
					Ring Calib. Factor:		**17.66 N/div**		Wet unit weight, kN/M^3:			**16.30**
Length of sample :		**60 mm**			Rate of strain :		**0.8 mm/min**		Dry Unit Weight, kN/M^3:			**14.90**
Width of sample:		**35 mm**			Moisture content, %		**21.9**		Sample Condition:		**Molded**	
				Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
				76.8 kPa			**154 kPa**			**243 kPa**		
Horizontal	Actual	strain	Corrected	Proving	Shear	Shear	Proving	Shear	Shear	Proving	Shear	Shear
isplacemer	leformatior	%	Area	Ring	Load	Stress	Ring	Load	Stress	Ring	Load	Stress
[mm]	cm		[mm2]	Reading	[KN]	[kPa]	Reading	[N]	[kPa]	Reading	[N]	[kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	3.5	9.5546	4.57596	6.5	12.5823	6.02601	7.5	20.4741	9.80562
40	0.4	1.6	2076	5	13.7283	6.61287	8	17.7443	8.54733	9.5	26.0838	12.5645
60	0.6	2.4	2064	7.5	20.7122	10.035	10.5	21.9653	10.6421	11.5	31.7587	15.387
80	0.8	3.2	2052	9	25	12.1832	12.5	28.9971	14.1311	13	36.1111	17.598
100	1	4	2040	11.5	32.1324	15.7512	14	34.7222	17.0207	15.5	43.3088	21.2298
120	1.2	4.8	2028	13.5	37.9438	18.71	15.5	39.1176	19.2888	17.5	49.1864	24.2536
140	1.4	5.6	2016	14	39.5833	19.6346	16.5	43.5651	21.6097	19	53.7202	26.6469
160	1.6	6.4	2004	13.5	38.3982	19.1608	18	46.6518	23.2793	21.5	61.1527	30.5153
180	1.8	7.2	1992	12.5	35.7681	17.9559	19.5	51.1976	25.7016	23.5	67.244	33.757
200	2	8	1980				19	55.7982	28.1809	25	71.9697	36.3483
220	2.2	8.8	1968					54.697	27.7932	27.5	79.6494	40.4723
240	2.4	9.6	1956							29	84.5092	43.2051
260	2.6	10.4	1944							28.5	83.5648	42.986
280	2.8	11.2	1932							27	79.6584	41.231
300	3	12	1920									
320	3.2	12.8	1908									
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									



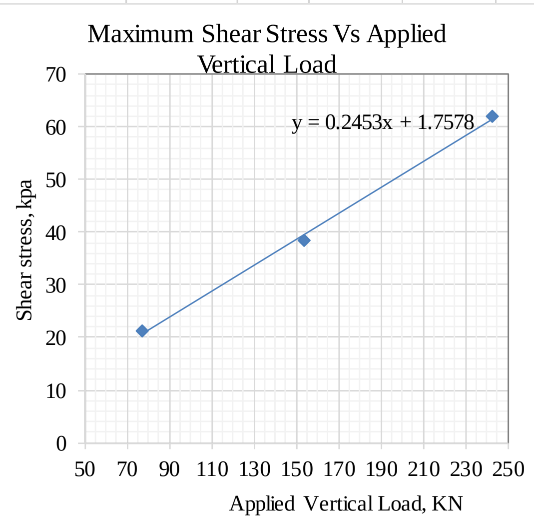


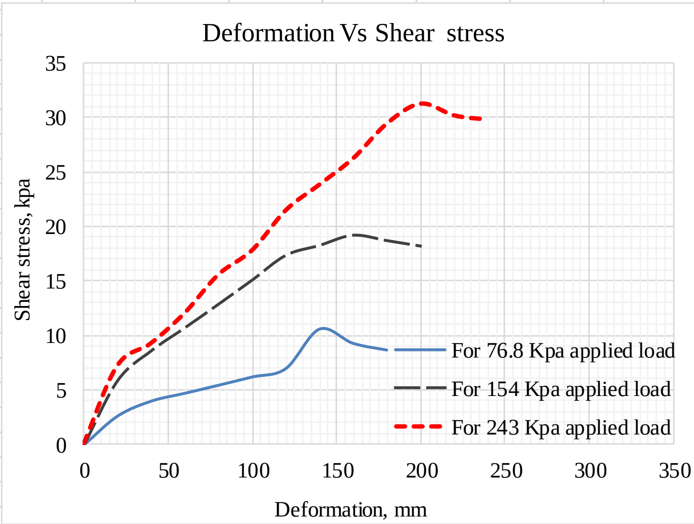
Angle of internal friction, ϕ =	**15.19**	**Cohesion , C (kN/m^2) =**	**17.06**

Table B1.1 Direct shear results for AS-1-04

Sample Code.	**AS-1-04**	Sample Depth, m:	**7.5m**		
		Ring Calib. Factor:	**17.66 N/div**	Wet unit weight, kN/M^3:	**16.20**
Length of sample :	**60 mm**	Rate of strain :	**0.8 mm/min**	Dry Unit Weight, kN/M^3:	**14.90**
Width of sample:	**35 mm**	Moisture content, %	**17.6**	Sample Condition:	**Molded**

				Applied Vertical Stress **76.8 kPa**			Applied Vertical Stress **154 kPa**			Applied Vertical Stress **243 kPa**		
Horizontal Displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	2	5.45977	2.61483	4.5	12.2845	5.88337	5.5	15.0144	7.19079
40	0.4	1.6	2076	3	8.23699	3.96772	6.5	17.8468	8.59673	7	19.2197	9.25802
60	0.6	2.4	2064	3.5	9.6657	4.68299	8	22.093	10.704	9	24.8547	12.042
80	0.8	3.2	2052	4	11.1111	5.41477	9.5	26.3889	12.8601	11.5	31.9444	15.5675
100	1	4	2040	4.5	12.5735	6.16349	11	30.7353	15.0663	13	36.3235	17.8057
120	1.2	4.8	2028	5	14.0533	6.92961	12.5	35.1331	17.324	15.5	43.5651	21.4818
140	1.4	5.6	2016	7.5	21.2054	10.5185	13	36.756	18.2321	17	48.0655	23.842
160	1.6	6.4	2004	6.5	18.488	9.22556	13.5	38.3982	19.1608	18.5	52.6198	26.2574
180	1.8	7.2	1992	6	17.1687	8.61881	13	37.1988	18.6741	20.5	58.6596	29.4476
200	2	8	1980				12.5	35.9848	18.1742	21.5	61.8939	31.2596
220	2.2	8.8	1968							20.5	59.375	30.1702
240	2.4	9.6	1956							20	58.2822	29.7966
260	2.6	10.4	1944									
280	2.8	11.2	1932									
300	3	12	1920									
320	3.2	12.8	1908									
340	3.4	13.6	1896									
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									

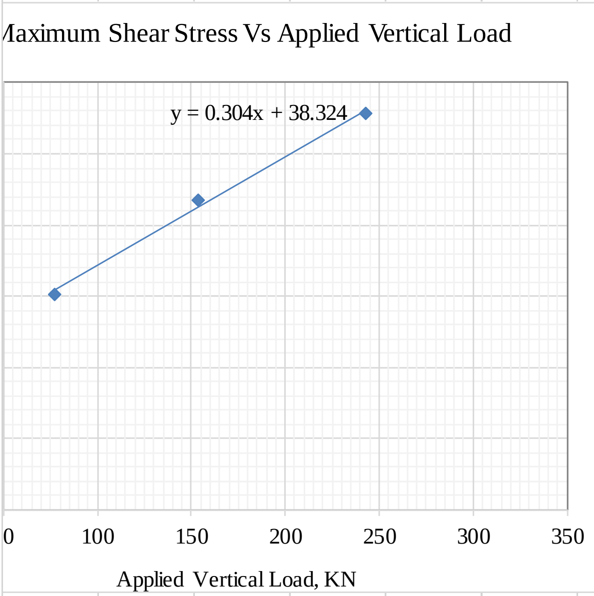


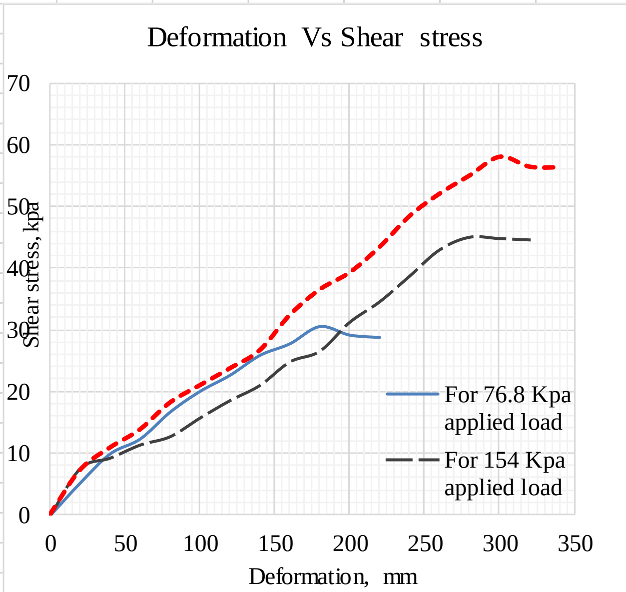


Angle of internal friction, ϕ =	13.78	**Cohesion , C (kN/m^2) =**	1.76

Table B1.1 Direct shear results for SG

Sample Code.		SG			Sample Depth, m:		6.50					
					Ring Calib. Factor:		**17.66 N/div**		Wet unit weight, kN/M^3:			**15.16**
Length of sample :		**60 mm**			Rate of strain :		**0.8 mm/min**		Dry Unit Weight, kN/M^3:			**16.80**
Width of sample:		**35 mm**			Moisture content, %		**23.9**		Sample Condition:		**Molded**	
				Applied Vertical Stress **76.8 kPa**			Applied Vertical Stress **154 kPa**			Applied Vertical Stress **243 kPa**		
Horizontal displacement [mm]	Actual deformation cm	strain %	Corrected Area [mm2]	Proving Ring Reading	Shear Load [KN]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0	0	2100	0	0	0	0	0	0	0	0	0
20	0.2	0.8	2088	4	10.91954	5.229665	5.8	15.83333	7.583014	5.5	15.01437	7.190789
40	0.4	1.6	2076	7.5	20.59249	9.919309	7	19.21965	9.258022	8.2	22.51445	10.84511
60	0.6	2.4	2064	9.2	25.40698	12.30958	8.5	23.47384	11.37298	10.3	28.44477	13.78138
80	0.8	3.2	2052	12.3	34.16667	16.65042	9.4	26.11111	12.72471	13.4	37.22222	18.13948
100	1	4	2040	14.6	40.79412	19.99712	11.5	32.13235	15.75115	15.3	42.75	20.95588
120	1.2	4.8	2028	16.3	45.81361	22.59054	13.4	37.66272	18.57136	17.1	48.06213	23.69928
140	1.4	5.6	2016	18.4	52.02381	25.80546	15	42.41071	21.03706	19	53.72024	26.64694
160	1.6	6.4	2004	19.5	55.46407	27.67668	17.5	49.77545	24.83805	22.8	64.8503	32.36043
180	1.8	7.2	1992	21.2	60.66265	30.45314	18.5	52.93675	26.57467	25.4	72.68072	36.48631
200	2	8	1980	20	57.57576	29.07867	21.5	61.89394	31.25957	27	77.72727	39.2562
220	2.2	8.8	1968	19.5	56.47866	28.69851	23.5	68.06402	34.58538	29.5	85.44207	43.41569
240	2.4	9.6	1956				26	75.76687	38.73562	32.5	94.70859	48.41952
260	2.6	10.4	1944				28.5	83.56481	42.98602	34.5	101.1574	52.0357
280	2.8	11.2	1932				29.5	87.03416	45.04874	36	106.2112	54.97473
300	3	12	1920				29	86.09375	44.84049	37.5	111.3281	57.9834
320	3.2	12.8	1908				28.5	85.14151	44.62343	36	107.5472	56.36644
340	3.4	13.6	1896							35.5	106.7247	56.28939
360	3.6	14.4	1884									
380	3.8	15.2	1872									
400	4	16	1860									





Angle of internal friction, ϕ =	**16.91**	**Cohesion , C (kN/m^2) =**	**38.32**

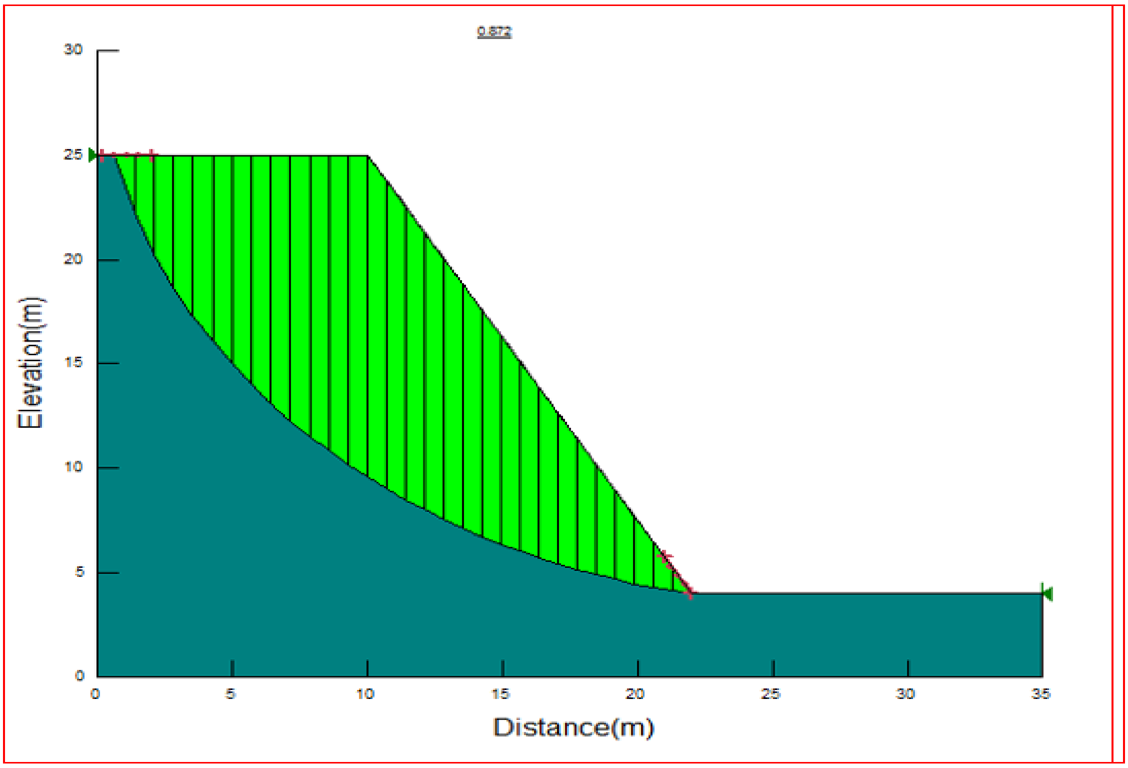


Figure B1.11 Most Critical Failure Circle for Homogenous failed slope **AS-1-01**

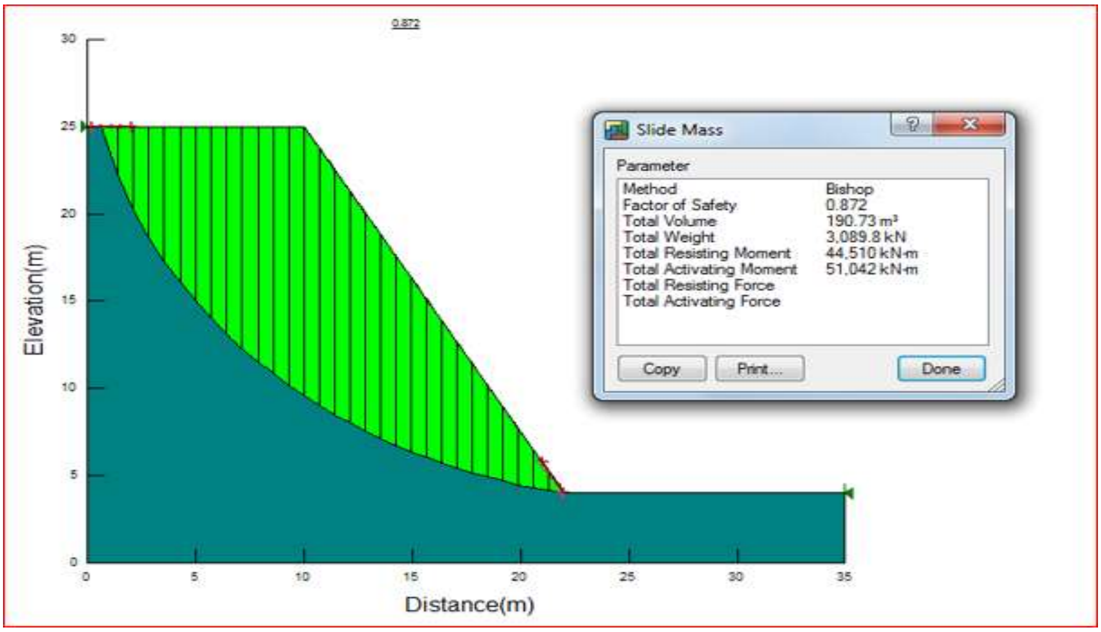


Figure B1.12 Most critical failure circle for Homogenous slide mass failed slope **AS-1-01**

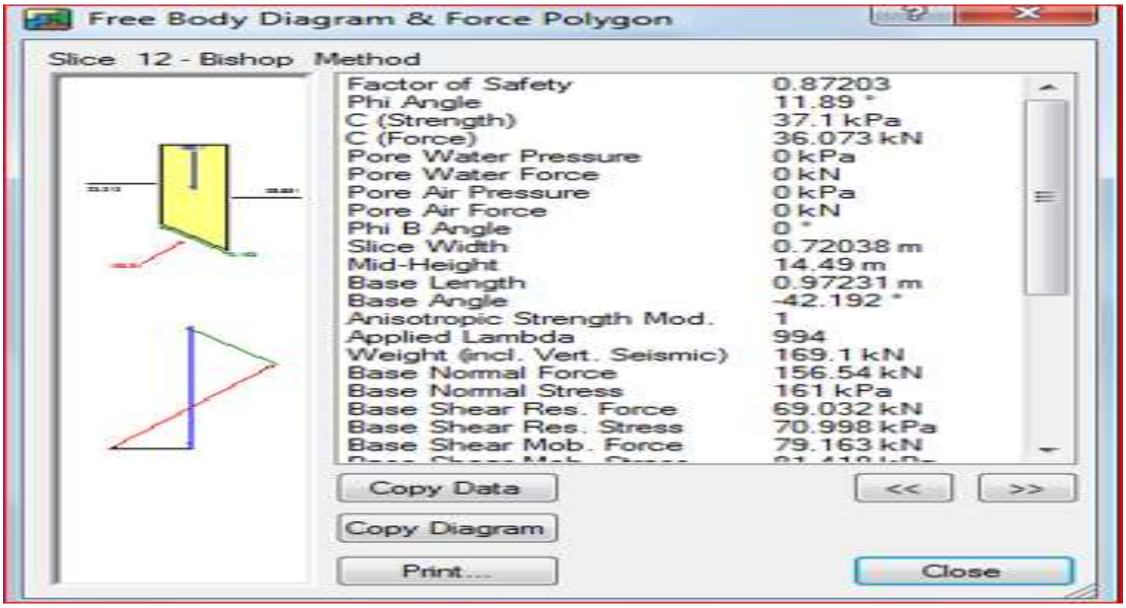


Figure B1.13 Stress acting on slice shaded for slope **AS-1-01**

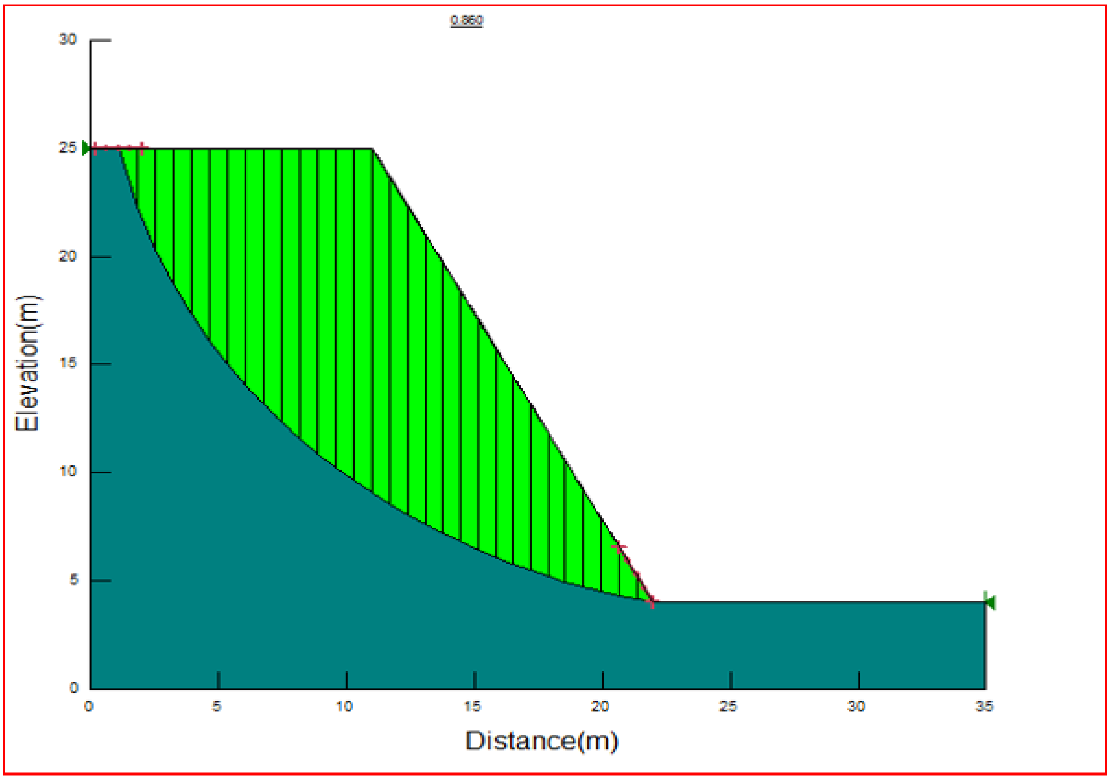


Figure B1.14 Stress acting on slice shaded for slope AS1-01

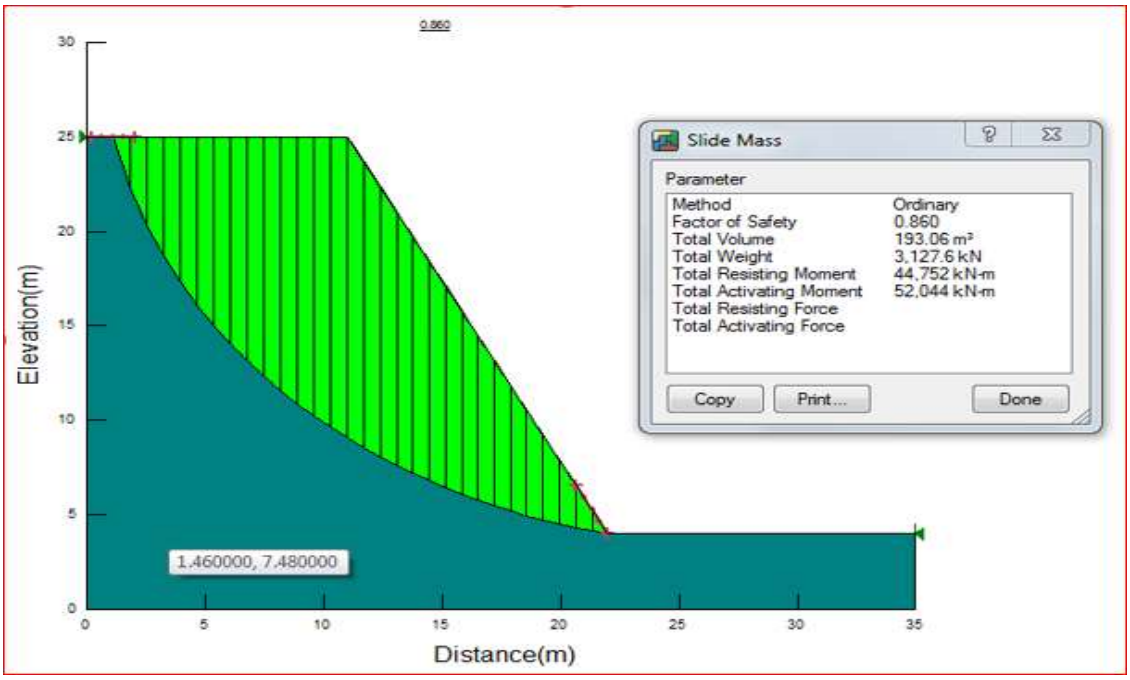
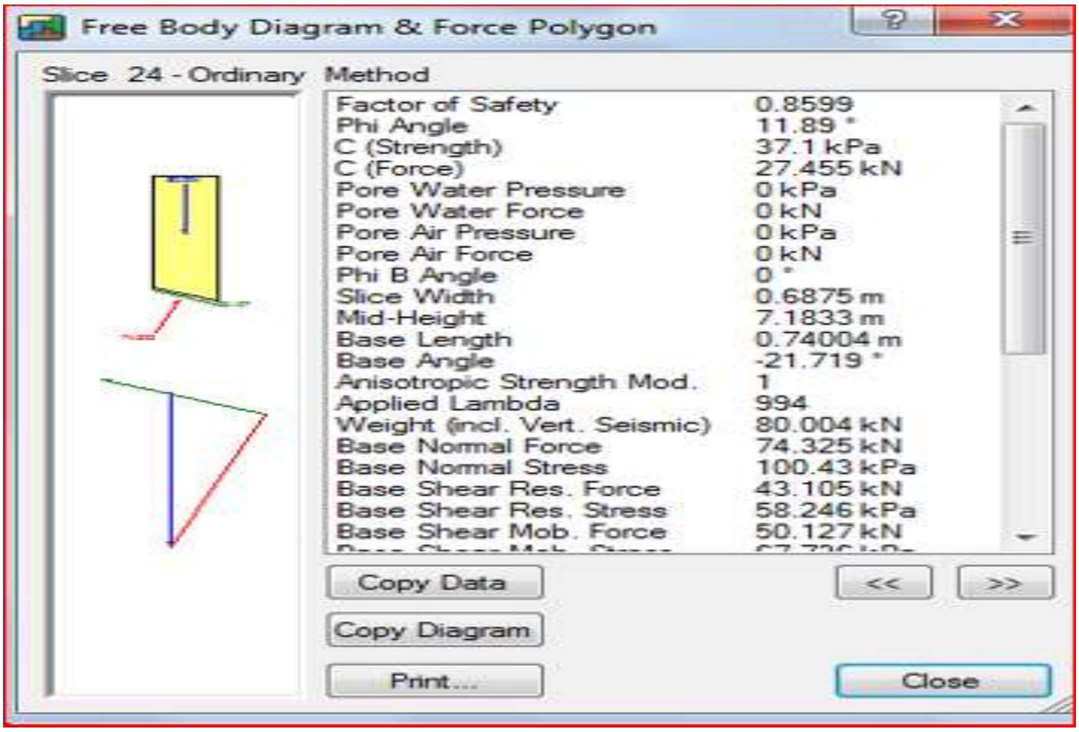
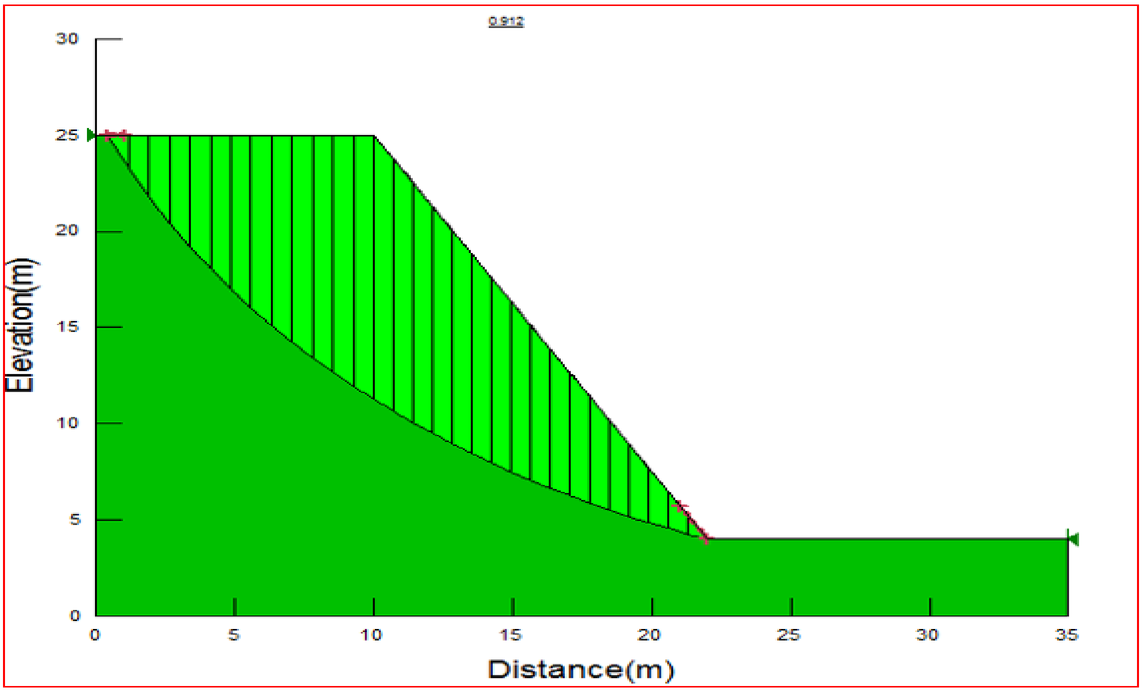


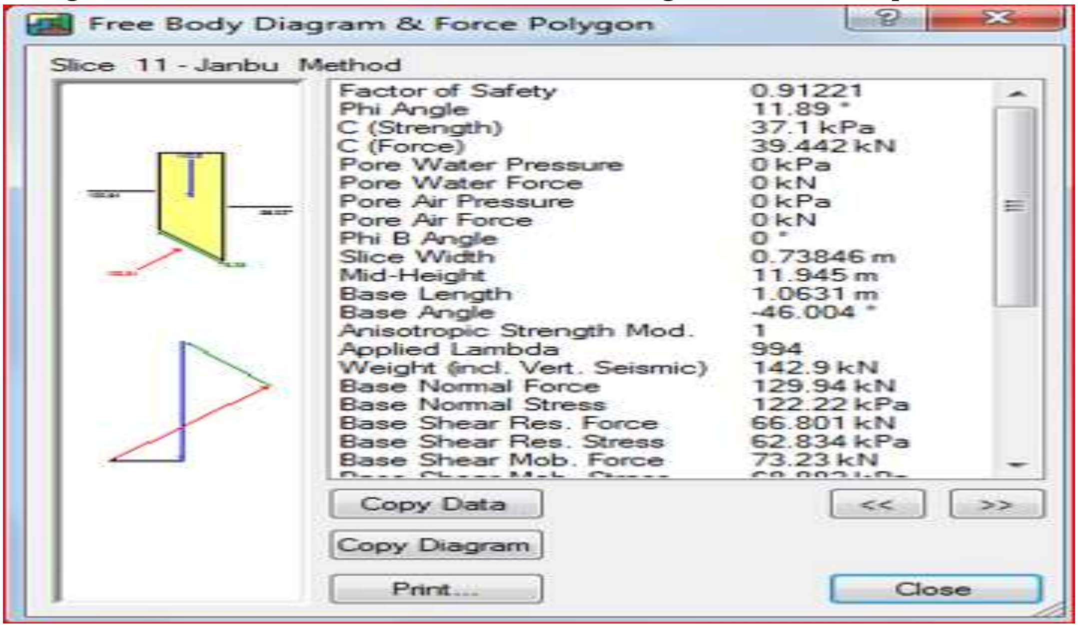
Figure B1.15 Most critical failure circle for Homogenous slide mass failed slope



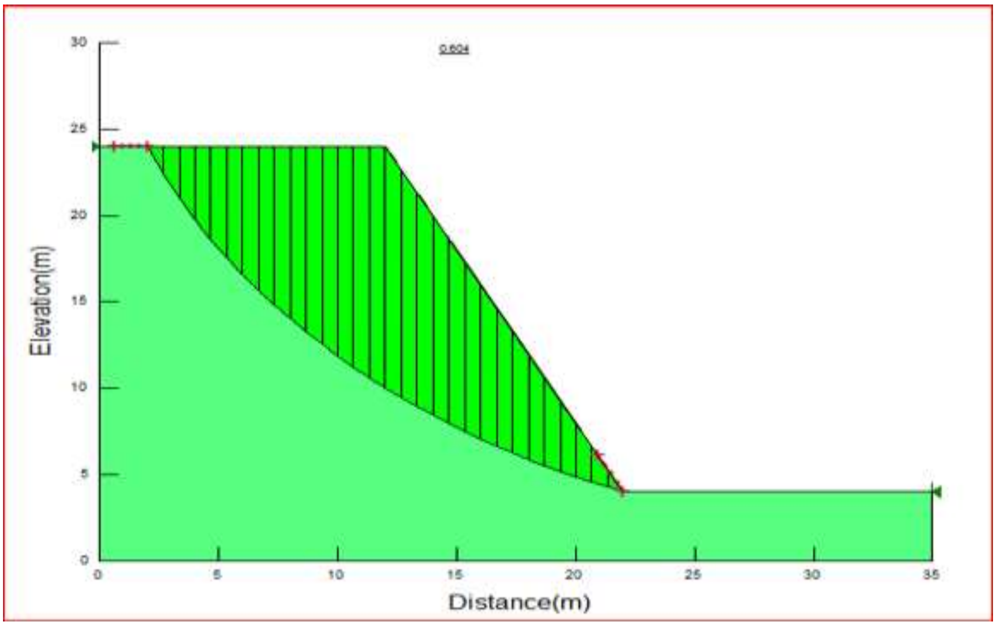
Figureb1.15 Most critical failure circle for homogeneous failed slope



Figureb1.16 Most critical failure circle for homogeneous failed slope AS-1-02



Figureb1.17 Most critical failure circle for homogeneous Failed slope AS-1-02.



Figureb1.18 Most critical failure circle for Homogeneous failed slope AS103

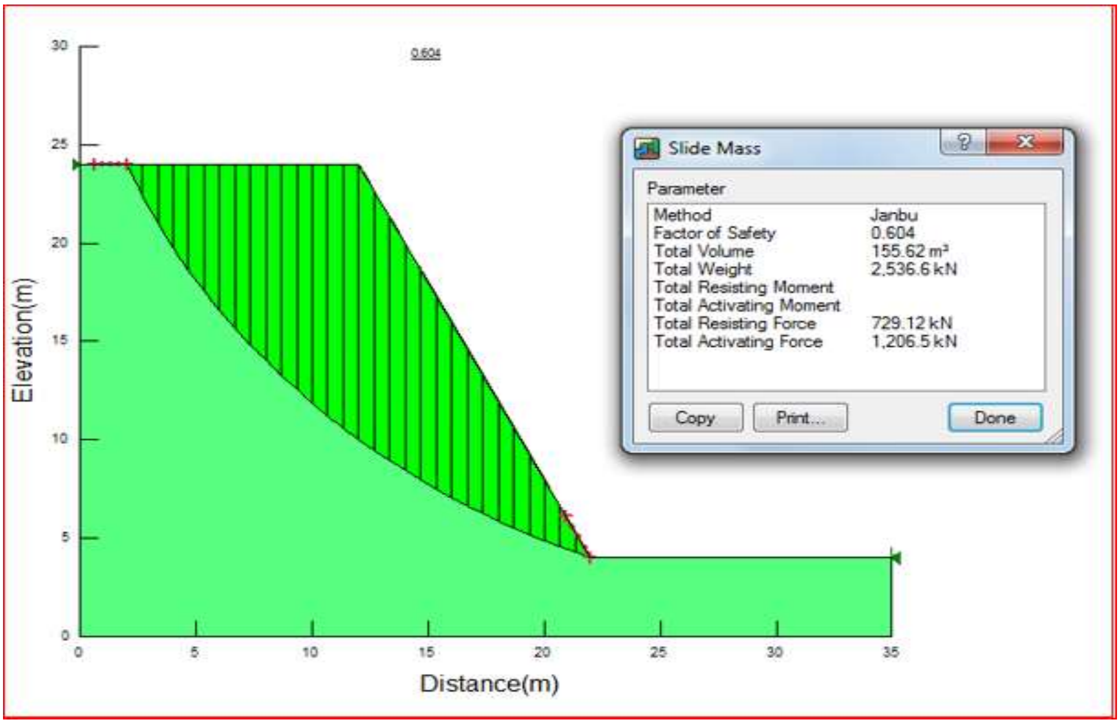
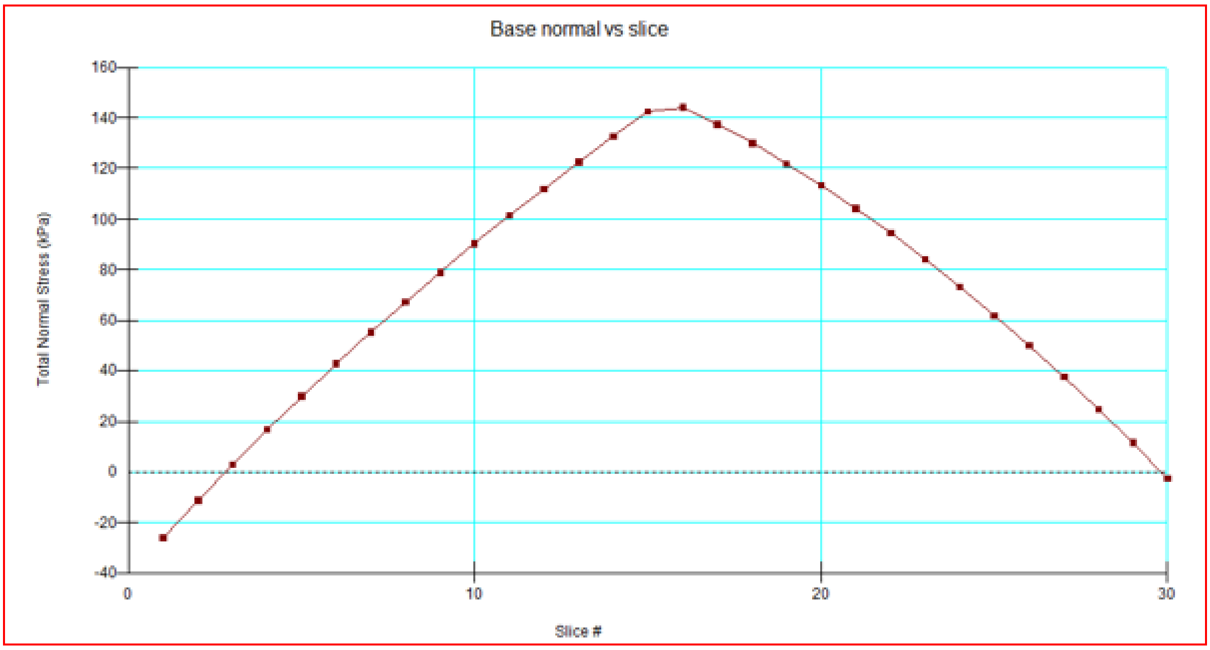
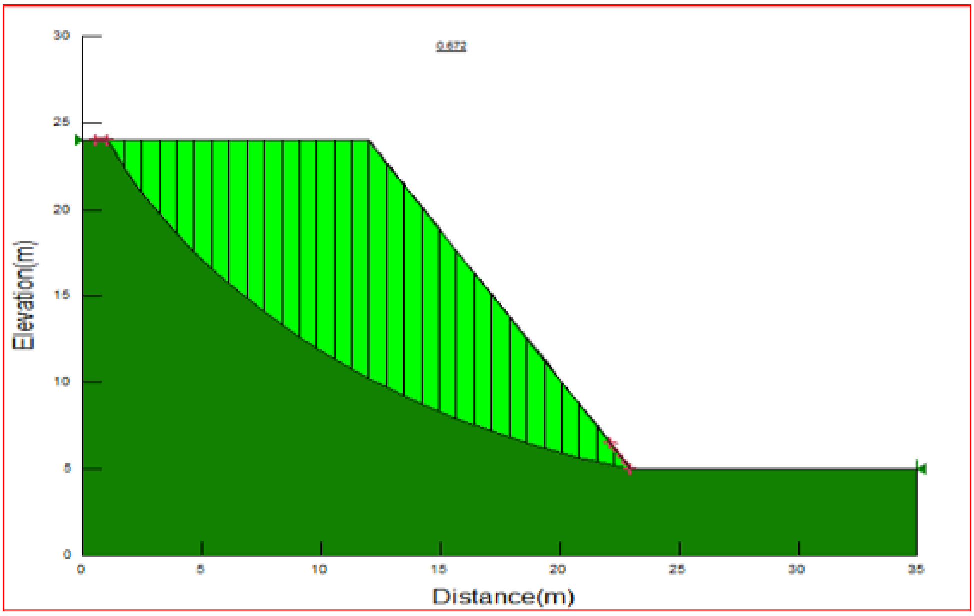


Figure B1.19 Most critical failure circle for homogeneous slide mass slope AS-1-03



B1.20 Figure slice vs. total normal stress



FigureB1.21 Most critical failure circle for homogeneous failed slope AS-1-03

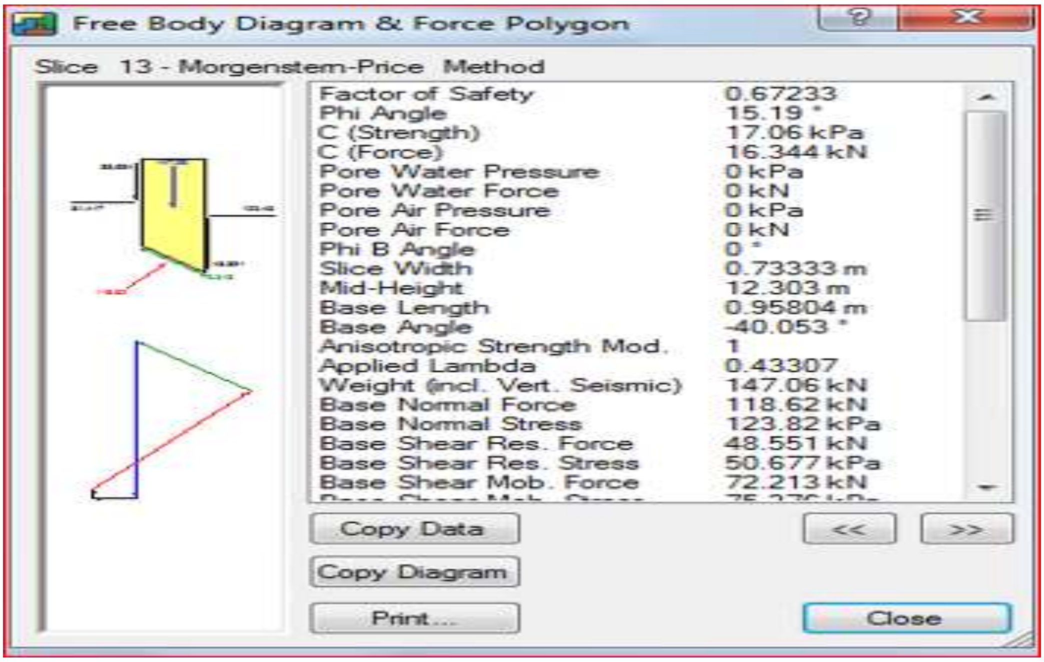


Figure B1.22Stress acting on slice shaded for slope AS-1-03

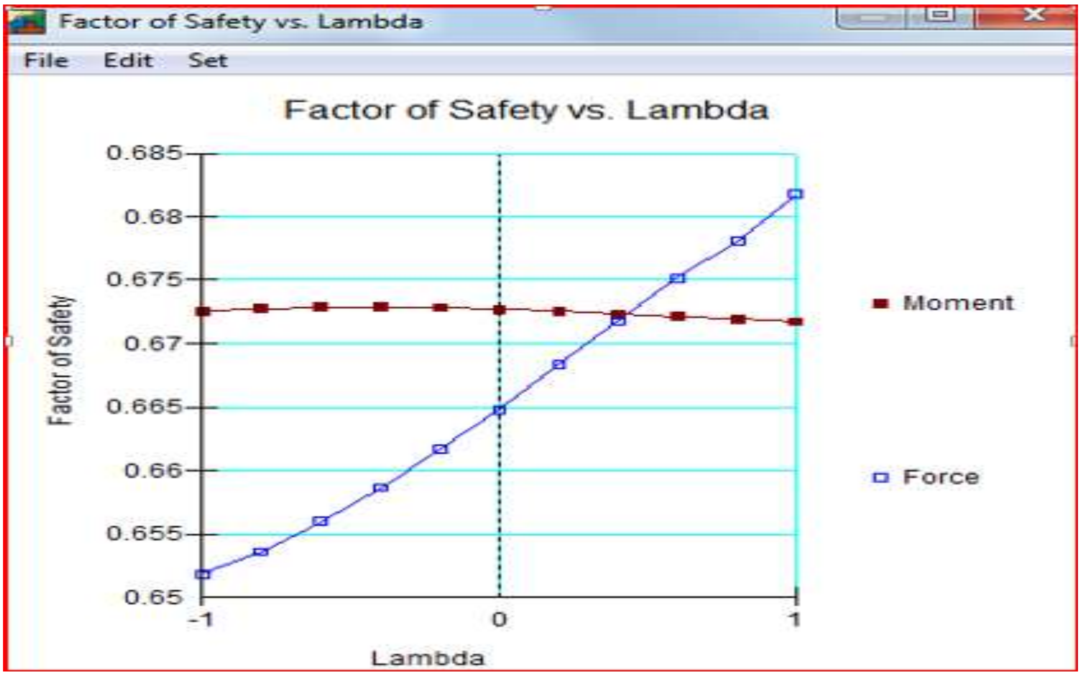


Figure B1.23 Figure lambda vs. factor of safety

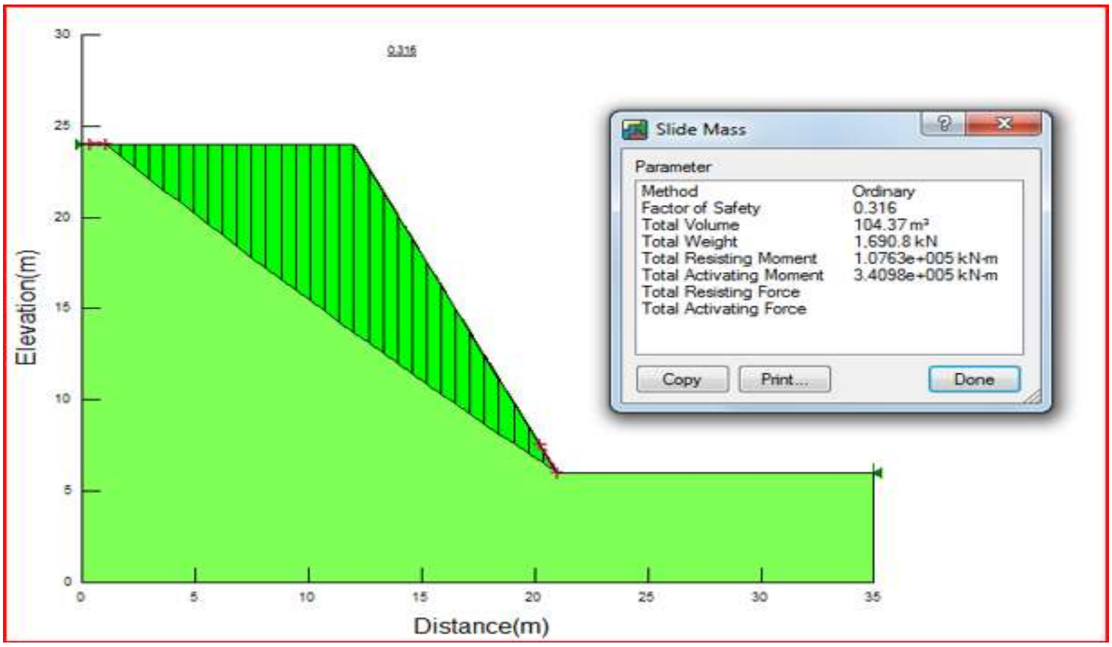


Figure B1.24Most critical failure Circle for Homogeneous slide mass for slope AS104

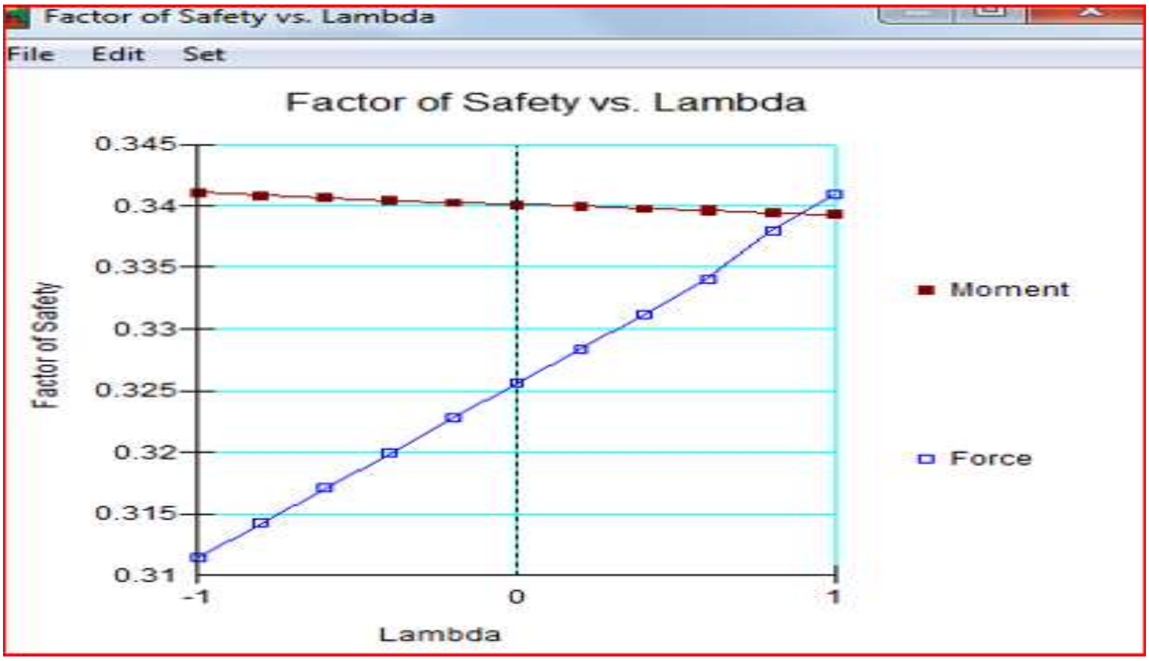


Figure B1.25 lambda vs factor of safety

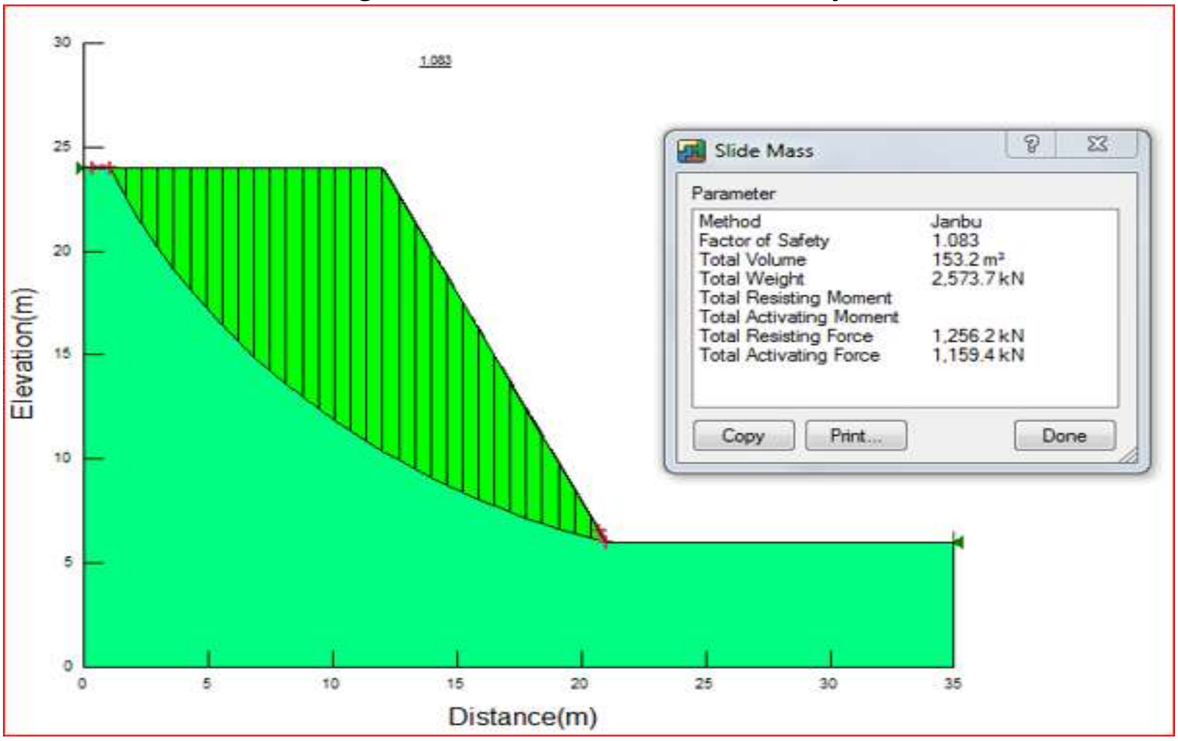
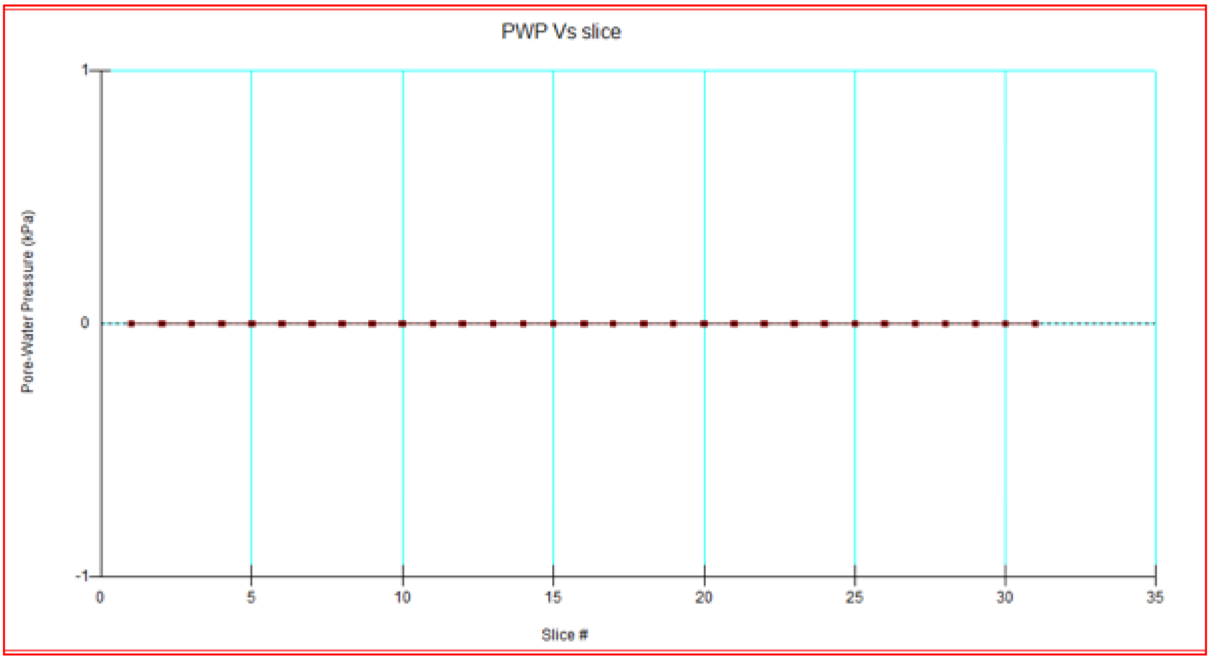


Figure B1.27 Most critical failure circle for Homogeneous slide mass for slope SG



FigureB1.27 slice vs pore water pressure for slope SG

Table B1.2 Total Resisting Force/Moment and Total Activating Force/Moment with Water Table below Failure Plane.

Failed slope Profile	Limit Equilibrium Methods	Total-Resisting-Force/Moment (KN/KN-m)	Total-Activating Force/Moment (KN/KN-m)
A-1-01	Morgenstern –price	1,149.7/2,7907	1,284.3/31,162
	Ordinary	/54.344	/47.608
	Bishop	/51.398	/45.472
	Janbu	1,912.6/	1,724.3/
A-1-02	Morgenstern –price	1023.8/	968.61/
	Ordinary	/66,600	/78,100
	Bishop	/71.956	/60.965
	Janbu	2,244.5/	1,833.9/
K	Morgenstern –price	396.03/	805.68/
	Ordinary	/16,197	/30,586
	Bishop	/27.804	/58.456
	Janbu	435.59/	888.8/
AS-1-01	Morgenstern –price	838.53/	895.59/
	Ordinary	/45.702	/64.564
	Bishop	33.056/	47.684/
	Janbu	871.69/	1,198.7/
AS-1-02	Morgenstern –price	1041.6/	1055.5/

	Ordinary	/44.752	/52.044
	Bishop	/44.510	/51.042
	Janbu	1,125.2/	1,233.5/
AS-1-03	Morgenstern –price	855/37.16	1271.8/855.7
	Ordinary	/52.86	/87.86
	Bishop	/37.542	/67.473
	Janbu	729.12/	1206.5/
AS-1-04	Morgenstern –price	371.11/15.433	1,093.4/45.471
	Ordinary	/1,076	/3,409
	Bishop	/88.926	/276.16
	Janbu	265.24/	840.28/
AF	Morgenstern –price	1,178.8/47.280	1248.7/50.08
	Ordinary	/36.613	/39.253
	Bishop	/37.318	/41.122
	Janbu	957.04/	1034.8/
SG	Morgenstern –price	1108.7/4.348	905.65/3.553
	Ordinary	/51,289	/49,593
	Bishop	/52.849	/49.606
	Janbu	1,256.2/	1,159.4/

Note that Morgenstern –price method gives best factor of safety since it considers both moment and force analysis.

Appendix C: Site and general information of the route

Figure C1.1 Blocked culverts

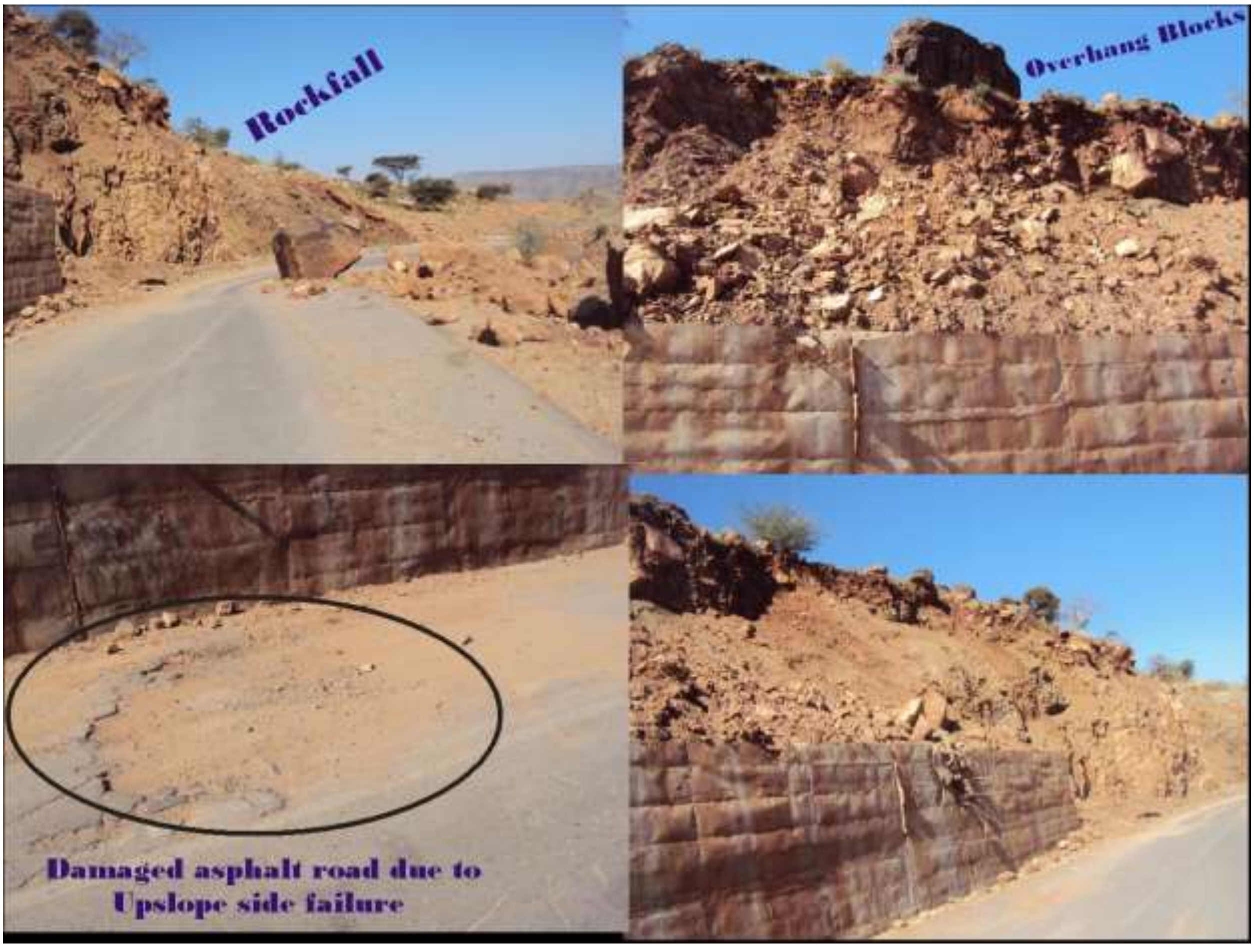
Figure C1.2 Road side ditches scouring problems

Figure C1.3: Damaged retaining wall and failed slopes

Figure D1.4 Cracked slope


Damaged retaining structures

Figure C1.4 Effects of rock fall on the road



Appendix D Additional Information of the Route

Table D1.1Land slide dimension along the route

	Location			Size (Dimension)			
Sr No	Easting	Northing	Elevation	Length	Width	Depth	**Lithology**
1	584544	1510350	2304	43	4	1.5	Shale
2	584814	1510998	2236	153	3.25	2	Shale
3	588421	1510432	1887	131	2	1.6	Shale
4	590764	1509835	1542	525	3	1	Sandstone (reddish)
5	590836	1509757	1529	227	2.5	0.8	Sandstone (reddish)
6	590935	1509047	1358	60m	4	4	Sandstone (reddish)
7	590944	1509838	1519	257	3	1.6	Sandstone (reddish)
8	590964	1509640	1501	75	4	40 cm	Sandstone (reddish)
9	591025	1509189	1373	30	3	4	Sandstone (reddish)
10	591353	1509278	1359	112	5	1.5	Sandstone (reddish)
11	591406	1509042	1286	20	4	2	Sandstone (reddish)
12	591452	1509126	1265	40	2.5	2	Sandstone (reddish)

13	592092	1509821	1206	52	1	0.6	Phyllite
14	594112	1510044	1161	65	5	2	Phyllite
15	594159	1510224	1155	25	1.5	1	Phyllite
16	596084	1513317	985	33	2	10	Quaternary sediment
17	596104	1513323	986	23	1.5	2	Quaternary sediment

Table D1.2 Damaged Road Side Shoulder Dimensions

	Road Shoulder							
	Location			Size (dimension)		Remark		
Sr No	Easting	Northing	Elevation	Length (m)	Width (m)	Upslope	Downslope	Status of road shoulder
1	591393	1509604	1458	17	1.75		Downslope	scouring problem
2	591384	1509498	1420	22	1.5		Downslope	scouring problem
3	593207	1509893	1179	1.4	0.6		Downslope	scouring problem
4	594775	1512281	1082	20	1		Downslope	scouring problem
5	596058	1513268	983	22	1.6		Downslope	scouring problem

Table D1.3 dimension of damaged road section

	Road Section					
	Location			**width (m)**	Length (m)	Possible Cause
Sr No	Easting	Northing	Elevation			
1	585046	1511478	2171	5.4	16	Limestone (Rockfall)
2	590921	1509742	1510	B1 = 1.2	L1 = 3.9	Adigrat Sandstone (Toppling Failure)
3	591278	1509302	1365	10	15	Adigrat Sandstone (Rockfall Failure)
4	591287	1509311	1367	3.4	2.8	Adigrat Sandstone (Rockfall Failure)
5	591301	1509315	1361	6	5.4	Adigrat Sandstone (Rockfall Failure)

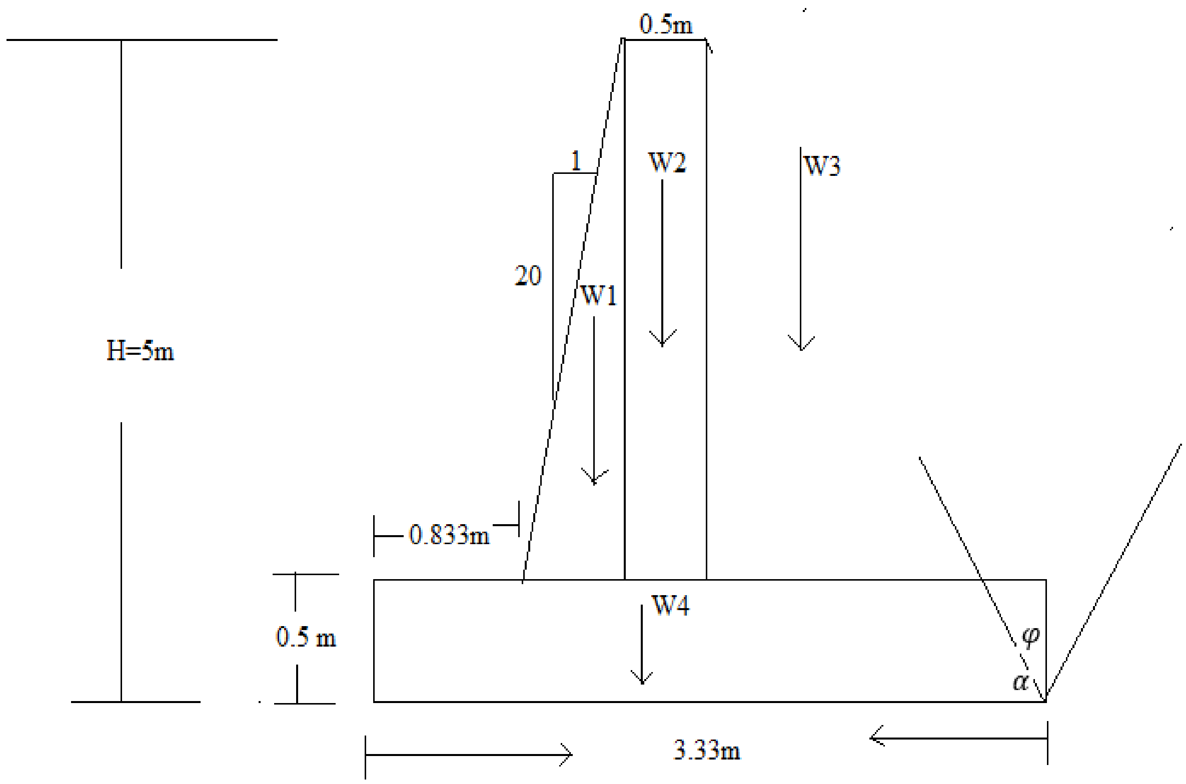
Table D1.4 Locations of Damaged Retaining Walls at the Route

	Retaining structures				
	Location				
Sr No	Easting	Northing	Elevation	Possible Cause	Remark
1	590782	1510081	1566	Adigrat Sandstone (Rockfall)	
2	591243	1509291	1370	Adigrat Sandstone (Rockfall)	
3	591285	1509311	1368	Adigrat Sandstone (Rockfall)	
4	591330	1509122	1305	Adigrat Sandstone (Rockfall)	
5	591350	1509149	1305	Adigrat Sandstone (Rockfall)	

Appendix E
Design of Gravity Retaining Wall

Design data for slope AS-1-04 profile Retaining wall

The soil to be retained is non-plastic and low cohesive silty soil with moist unit weight of 16.2 KN per cubic meter, internal angle of friction 12.24, and with cohesive value of 1.758 KN per square meter as represented by sample code number AS-1-04. From EBCS seven the bearing capacity of the pre-specified soil type is 280 Kpa. Taking C24 concert type for design, and assuming tentative dimensions of the gravity retaining wall as shown in figure below;



Stability analysis of the tentative dimension retaining wall

$K_a = \frac{1-sin\emptyset}{1+sin\emptyset} = K_a = \frac{1-sin12.24}{1+sin12.24}$=0.65

$P_a = \gamma * K_a * h - 2c\sqrt{K_a}.$

P_a=16.2*0.65h-2.835

p_a @ h=0m is 2.835Kpa and @ h=5m is 49.815 Kpa

The depth at which the p_a value equal to zero is $\frac{2c}{\gamma\sqrt{K_a}}$ i.e. $h_c = 0.69m$ from the top.

Slice No	Description	Force		Lever arm	Moment about toe	
		vertical	horizontal		CW	CCW
1	W1=0.5*0.025*24*4.5	1.35		0.849	1.14615	
2	W2=4.5*0.5*24	54		1.108	59.832	
3	W3=2.47*4.5*16.2	180.2		2.594	467.4388	
4	W4=0.5*3.33*24	40		1.7	68	
5	Horizontal activating		107.35	1.44		154.584
6	vertical resisting		0.921	4.77	4.39317	
sum		275.55	106.425		600.81	154.584

a) **Factor of safety against sliding**

$FoS = \frac{\varphi R_V}{R_H}$; Where

$\varphi = 45 - \phi/2$ = 45-12.24/2=38.83

Hence Fos =$\frac{tan38.83*275.55}{46.98}$=4.82……………………………………………………………….safe.

b) **Factor of safety against over turning**

$FoS = \frac{\sum M_R}{M_{activating}}$=$\frac{600.81}{154.584}$=3.88…………………………………………………………safe.

c) **Factor of safety against bearing capacity**

Xmean=$\frac{\sum M_R}{\sum V}$= $\frac{600.81-154.284}{275.55}$=1.62m

Eccentricity (e) =$\frac{B}{2} - Xmean$=$\frac{3.33}{2} - 1.62 = 0.045m$

As e <$\frac{B}{6}$…………………………………………………………………no tension crack.

Max. Pressure $=P_{max}=\frac{\sum V}{b}(1+\frac{6e}{b})=\frac{275.55}{3.33}(1+\frac{6*0.045}{3.33})$

P_{max}=89.45 Kpa

Min .pressure = $P_{min}=\frac{\sum V}{b}(1-\frac{6e}{b})=\frac{275.55}{3.33}(1-\frac{6*0.045}{3.33})$

P_{min}=76.03 Kpa

FoS $=\frac{soil\ bearing\ capacity}{P_{max}}=\frac{200}{89.45}=3.13>3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..$safe