ADDIS ABABA UNIVERSITY

School of Graduate Studies

College of Natural Science

ASSESSMENT OF SLOPE STABILITY USING COMBINED PROBABLISTIC AND DETERMINISTIC APPROACH FOR SELECTED SECTIONS ALONG GOHATSION DEJEN ROUTE, BLUE NILE GORGE, CENTRAL ETHIOPIA

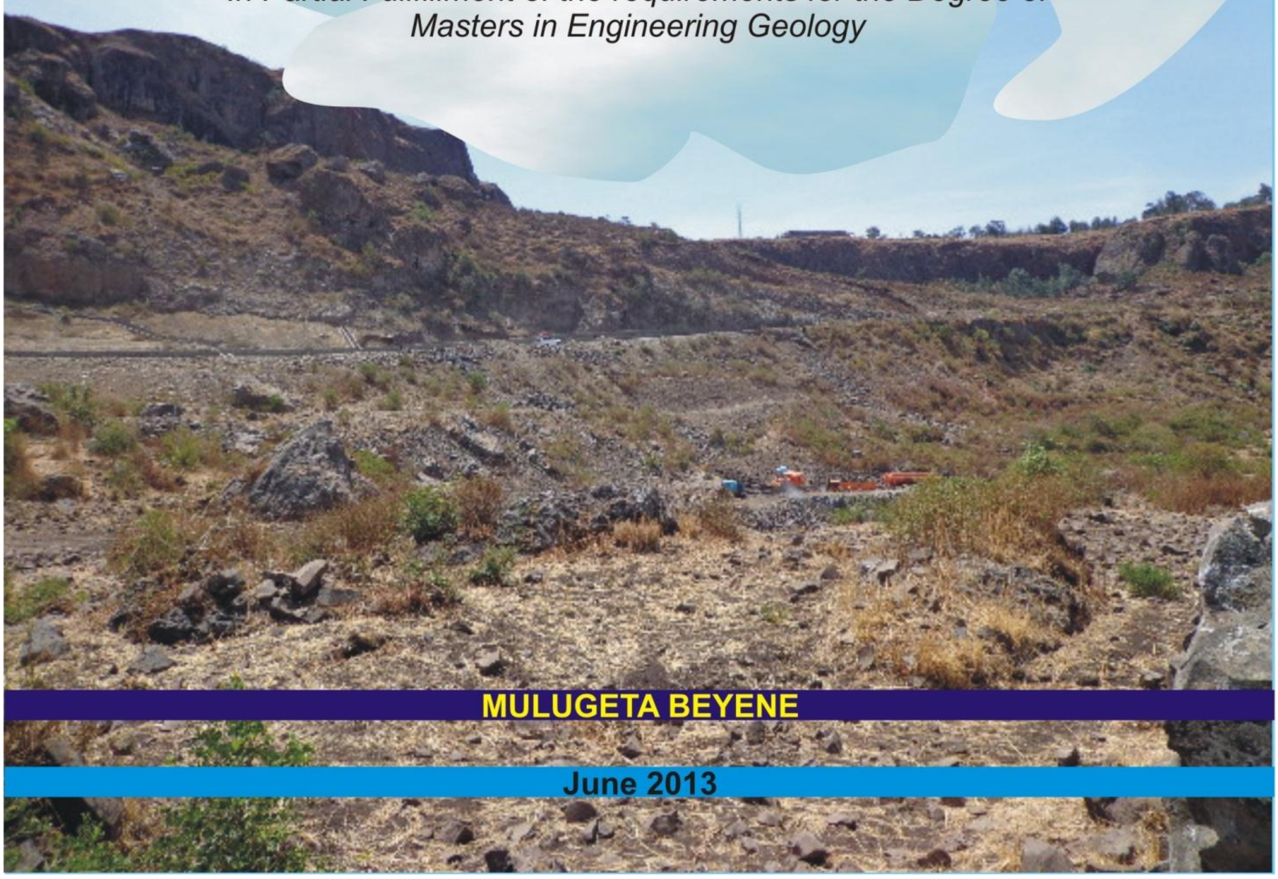
A Thesis

Submitted to

The School of Graduate Studies

Addis Ababa University

In Partial Fullfillment of the requirements for the Degree of Masters in Engineering Geology



MULUGETA BEYENE

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Advisor

Dr. Tarun K.Raghuvanshi

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By
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DECLARATION

I hereby declare that the thesis entitled "ASSESSMENT OF SLOPE STABILITY USING COMBINED PROBABLISTIC AND DETERMINISTIC APPROACH FOR SELECTED SECTIONS ALONG GOHATSION DEJEN ROUTE, BLUE NILE GORGE, CENTRAL ETHIOPIA" has been carried out by me under the supervision of Dr. Tarun Kumar Raghuvanshi, School of Earth Sciences, Addis Ababa University during the year 2013 as part of Master of Science Program in Engineering Geology. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

MULUGETA BEYENE

Signature______________________________

Place and date of submission: School of Graduate Studies, Addis Ababa University

June 2013

Abstract

The present research study was conducted in the study area defined by Gohatsion and Dejen towns, in Abay Gorge, Central Ethiopia. The study area is about 185Kms north of Addis Ababa on the main road that connects Addis Ababa to Bahir Dar town. The study area witnesses severe problem of landslides during rainy seasons. Such slope failures in the area frequently hampered the safe functioning of the road, which is the important link between the Addis Ababa and the northern part of the country.

The main objective of the present research study was to evaluate the stability condition of selected critical slope sections by combined deterministic and probabilistic methods for existing and anticipated worst conditions to which the slopes would be subjected.

The general methodology followed includes; literature Review, collection of primary and secondary data, analysis of data by different graphical, empirical and analytical approaches through standard software, interpretation of results and finally based on the results recommendations were evolved. Based on the field manifestations of actual and potential instability, a total of four critical slope sections; two from failed colluvium section and two from rock slopes were identified for further analysis. Later, relevant data pertaining to various aspects related to geology, geomorphology, hydrogeology, climate etc. from both primary and secondary sources was collected. The stability analysis of colluvium slopes were made for the existing and anticipated worst conditions by utilizing SLIDE software that supports both deterministic and probabilistic approaches. Similarly, for rock slope having planar mode of failure ROCPLANE software that operate based on the principles of Modified Analytical Technique proposed by Sharma et al., (1995) and support both deterministic and probabilistic methods were also utilized for analysis.

The results from the present study in general indicates that the critical colluvium slope sections, investigated in the present study, would be unstable during anticipated adverse conditions and possibility of damage is unavoidable during the event of failure. The general characteristics of these slides with respect to the mass involved in large quantities, deep seated failure plane, groundwater conditions and progressive failure mode made it difficult to suggest and implement any techno-economic feasible remedial measures. Therefore, avoidance of these slope sections for any developmental activities would be more economic and safer option. The stability analysis carried out for rock slope section by deterministic and probabilistic approaches has revealed that the slope is stable and would be instable under anticipated adverse conditions. However, manifestation of instability of this slope section was observed during the field trip. Thus, stability analysis was made at slide block level. The deterministic and probabilistic analysis results suggest possibility of rock block failures during anticipated worst conditions. Thus, based on the general findings of the present research suitable recommendations have been forwarded.

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Chapter 1

Introduction

1.1 Preamble

Slope stability is one of the major problems in geotechnical engineering where disasters involve loss to life and related infrastructure (Pan et al., 2008; Kanungo et al., 2006; Dai et al., 2002). The surface of the earth is continually being modified by mass movements operating in response to gravitational forces. One effect of the mass movements termed landslides can be to reduce the gradient of hill slopes to stable angles. Although there are numerous natural drivers of landslides, in many cases landslides result directly from disturbance of hillsides by road construction or other human activity (Gorsevski et al., 2006). The effect increases with Population growth as a function of which the need to find stable environment for agricultural land, infrastructural development and settlement aggravated the problem especially in developing countries and caused an increased interest in slope stability problems. This implies that it is very important to assess the stability of slopes or to predict the likelihood of failures of a body of soil or rock with sloping surface for safe functioning of hilly environments.

Slope failures in the form of landslide and rock fall are responsible for considerably greater economic losses and human casualties than is generally recognized all over the world (www.icsu-asia-pacific.org/resource_centre/Sassa-paper.pdf; www.emdat.be; Schuster and Fleming, 1986; Keefer, 2000; Mario and Jibson, 2000; Dai et al, 2002; Kanungo et al., 2006; Pan et al., 2008). Accounting for both the small but frequent and for the large and rarer catastrophic landslides, it has been calculated that during one average year, landslides kill about 5–7 people in Norway, 18 in Italy, 25–50 in USA, 186 in Nepal, 170 in Japan, and 140–150 in China (Sidle and Ochiai, 2006). However, the negative consequences of landslides are not limited to loss of life, but include the destruction of houses and infrastructures loss of productivity in the area affected (Turner and Schuster, 1996 as cited in Pan et al., 2008), unpredictable changes in the local watercourse, and reduction of arable or habitable land. Estimated costs of landslide damage (including both the direct costs caused by destruction, and indirect costs due to long-time effects in the local economy) are about 4 billion USD in Japan, 70 million USD in Canada, 2.6–5 million USD in Italy (Sidle and Ochiai, 2006).

Ethiopia is topographically a unique country where landslides are most likely to occur. Blue Nile gorge is one of the active landslide areas of the country. Although there were no recorded casualties of ancient time, integrated studies conducted since 1990's due to severity of the problem revealed there were loss of life and property in Blue Nile gorge.

Landslides in the region include deep-seated rotational slumps, massive translational slides, progressive creep movements, and debris- and mudflows. Rock falls on the other hand exist largely as discernible block topples and wedge failures all along mountains, valley walls, and road cuts (Lulseged Ayalew and Yamagishi, 2003).

The evolution of slope stability analysis in geotechnical engineering has followed closely the development in soil and rock mechanics as a whole. Assessment of stability of slope is so necessary from point of view of identifying potentially unstable areas in existing slopes and for the design of new slopes by considering the integrity and future performance of slopes associated with planned engineering construction. The engineering solutions to slope instability problems require good understanding of analytical methods, investigative tools and stabilization measures (Abramson et al., 2002).

Developmental activities may face great challenges due to unstable grounds. Similarly, the slope failure may interrupt the established imperative services like traffic movement, water supply, power production and similar infrastructures. Therefore, stability analyses are necessary to save human lives, reduce property damages and provide continuous services of existing structures. However, the chosen analysis method should be able to identify the existing safety conditions and suggest for technically feasible and economically viable solutions. As such combined methods will be used in this study for reliability and engineering solutions.

1.2 Problem Statement

Slope instability is one of the most important concerns among researchers and professionals in rock slope engineering. Slope failure is the second most destructive natural hazard after earthquake (Li et al., 1999); however it is the most frequent geo-hazard. Loss of lives of people living near to mountainous area and falling of blocks to the roads have enhanced the necessity of using and developing much reliable methods to analyze the stability of those vulnerable structures besides traditional slope stability analysis model (Gheibie, 2012).

Visual inspection from the performance of existing structure in the vicinity of the site and previous researches in the Blue Nile gorge revealed that there are active landslides in the area. Besides, historical records of slope failures in the northern section of the basin indicated that in 1960, a bigger landslide had destroyed the Gembechi village, and as a result 45 people were killed (Lulseged Ayalew, 1999). Most recently road failure earlier than its life time and various rock falls are also observed. In spite of previous research in this critical area on slope instability with different methods most of which were GIS and traditional analysis, none of them have been done without limitation that arise from parameters that were considered, scale on which the study were conducted, financial constraints that existed and the limitation on the models themselves. Thus, none of the previous researches has forwarded the real solution to the instability problems and the area is still facing slope instability problems. Further, landslides in the area have affected severely the road that connects Gohatsion with Dejen town. In general, landslide zones are also increasing more than anticipated by previous researches.

Many of the traditional slope stability assessment approaches are limited to a single valued parameter to characterize slope failure in the form of safety factor. However, the inherent variability of the geologic material and geomorphology of the area dictates a comparative use of the two geotechnical models (Deterministic and probabilistic models) for the safe performance of existing engineering structures and possible adverse effect the area may experience in future.

1.3 Significance of the Study

The present study was attempted with a very important scientific significance and practical worth. The scientific significance: slope stability analysis using combined models is a recent development in slope engineering unlike earlier single conventional method of analysis. Further, it is an interdisciplinary topic which synthesizes engineering geology, geotechnical engineering, mathematic statistics, and computer. The achievement of the present study can later be promoted for the development of these disciplines for further study in future.

As practical worth: combined deterministic and probabilistic slope analysis methods reflect the property of slope engineering more realistically, by overcoming the restrictions of many existing conventional slope stability analyses methods that depend on single value to represent performance of entire slope, and evaluate the failure probability or reliability of the slope more correctly and efficiently. Applying this study will enable people to consider the effects uncertainties and complex boundary condition of slope stability and based on the results of these combined analysis methods, engineers can carry out risk analyses, risk management, hazard forecasting and reinforcement design of slope failure in more economical and reliable way for subsequent works in this particular area as well as in different vulnerable areas elsewhere.

1.4 The Study Area

1.4.1 Location and Accessibility

The study area is located in the Blue Nile gorge (Locally known as Abay gorge) of Ethiopia, situated in between Amhara and Oromia regional states of the country. The Area is accessible by asphalted highway that connect Addis Ababa with Northern cities of the country especially to Debre markos and Bahir Dar covering distance of 185 Km (Gohatsion, the town to the Northern side of Abay river section) to 229 Km (Dejen, the town to the Southern Abay river section) of the main road from Addis Ababa. The study area is bounded geographically by 408000 m and 42400 m Easting, 1108000 m and 1126000 m Northing in UTM projection.

As the area is topographically rugged, accesses to the specific selected slope sections were challenging. Some of the colluvium sections had small local trails but for those rock slopes there was no route to climb the cliffs. It was accessed by searching for gentle slopes around.

1.4.2 Climate

i) Rain fall: Rain fall data collected from three stations (Gohatsion (Fig 1.2), Filiklik (Fig 1.3) and Dejen (Fig.1.4)) in the study area shows Rainfall varies considerably from year to year, with pronounced wetter and drier decadal cycles. The area receives maximum rain fall in the months between end of May to September with its peak value in July and/or August. From December to February the area is relatively dry.

The Abay Basin in general receives on average 1,535 mm of rainfall, ranging from 800 mm to 2,220 mm and generally increasing with altitude. The record of monthly rain fall at Gohatsion station from 1996 to 2012 indicates a big difference in between the years. Taking randomly five years gap in between them the mean annual rain fall in 1996 was 126 mm and this value reduced to 110 mm in the year 2001. However, in 2006 the value raised to 127 mm. Recent data in this station indicates there is low rain fall potential in 2012 with mean annual value of 44.2 mm and annual rain fall of 1606.1 mm. By taking total precipitations of the other months as negligible, total rain fall from March to November for Gohatsion station was analyzed for the most recent year, 2012.

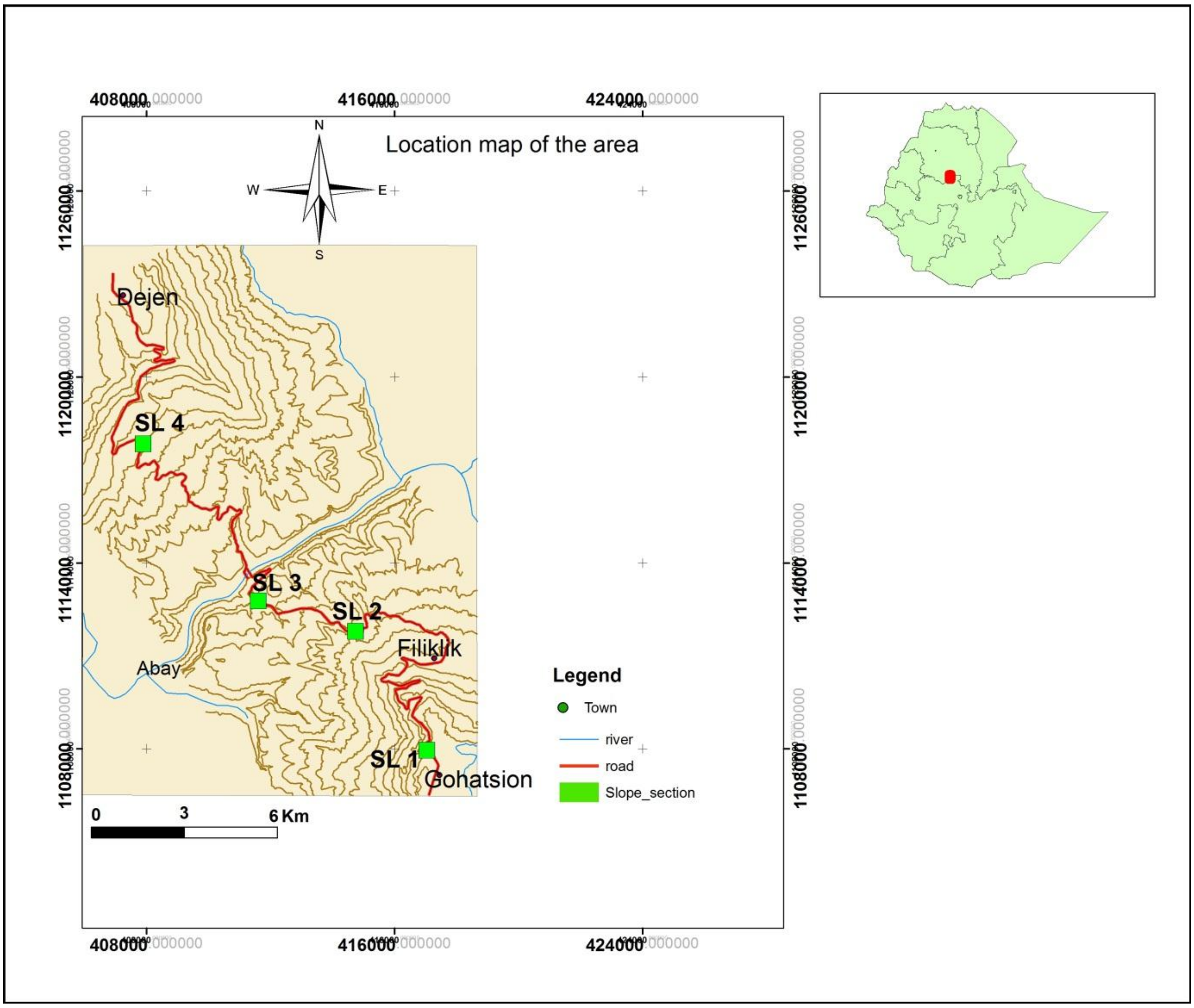


Fig 1.1 Location map of the study area

In March, 2012 total rain fall of 47.8 mm was recorded at Gohatsion station. In April this value was raised to 55.5 mm and in the next month, May the value dramatically dropped by half to 25.5 mm. This type of fluctuation effect the ground water table which in turn influence the strength of material through which it is moving, especially for weak material. The grain to grain contact will be loosened and material cohesion will be affected. In June, 2012 the value again raised to 63.7 mm followed by abrupt increase to 457.2 mm in July in weak total rain fall experienced its peak value of the year. This sudden increase by many folds is serious from slope instability point of view. In a particular study area the failed slope sections are mostly colluvium in nature with boulder sized fragments. This material is highly permeable with relatively loose connection between its grains from strength point of view. Hence, an abrupt change in rain fall intensity will cause increase in ground water. One series slide down this station has been studied presently. Down on the way to Dejen, another station is found at Filiklik on Southern Abay gorge. Topographically it is found at lower level in the gorge relative to Gohatsion station which is found at a distance away from flank of the gorge.

This area receives lower annual rain fall relative to upper Gohatsion station. The annual rain fall of the area varies between 573.3 mm to 2154.3 mm in the years considered from 1996 to 2012. These extreme values has been recorded in the year 2007 (maximum) and 2003 (minimum). Reconnaissance survey made to identify critical section revealed that there is a big slide in the section.

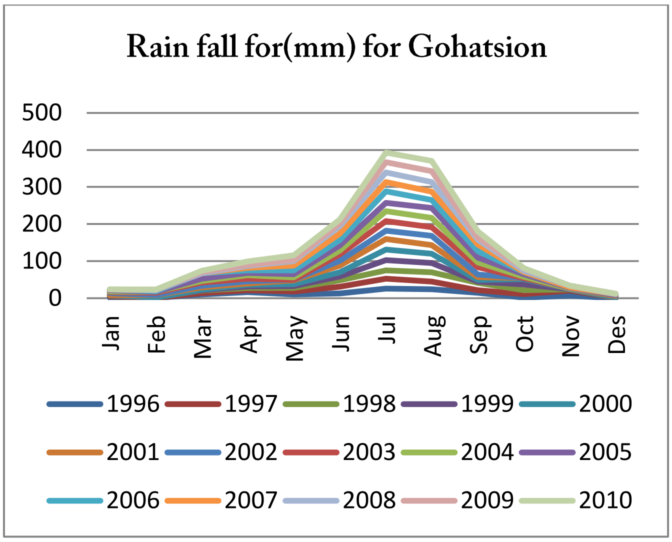


Fig. 1.2 Graph showing monthly rainfall of Gohatsion station.

The strange thing happened in 2007 in Filiklik that the area received annual rain fall difference of 916.2 mm more than at Gohatsion in the same year. May be this situation have exacerbated the problem in the area more than ever. However, it wasn't considered in the present study. Recorded rainfall data at this section also shows that there are variations in value from one month to another.

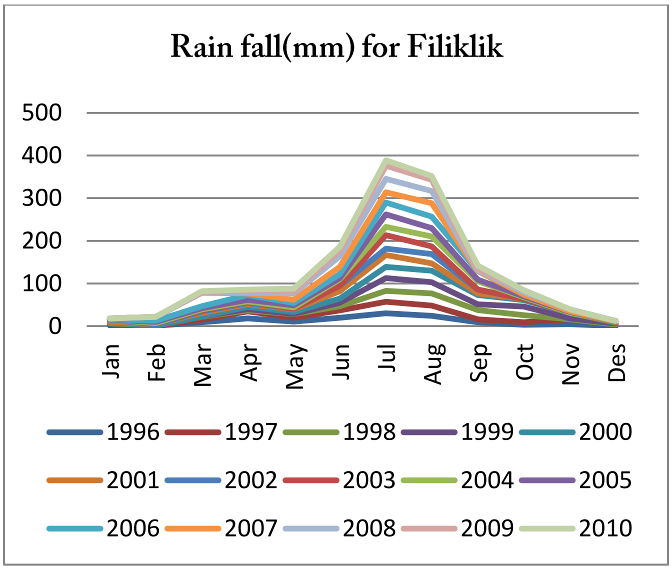


Fig 1.3 Graph showing monthly rainfall of Filiklik station.

Considering months from June to October as the highest rain fall receiving months and neglecting others, most recent (2012) record of rain fall data in the area was compared to observe peak rain fall receiving months. Accordingly, in July, 2012 the peak rain fall record of the station was 235.6 mm. In August, of the same year the value slightly dropped to 213.3 mm and in September abruptly dropped to 156.6 mm. The record shows there was no precipitation in October at the station but only this few months contributed to the total annual rain fall of 1631.0 mm in the area.

The third station considered in this study was on the way that is found to the Northern Abay gorge at Dejen. In this section one big slide was selected for further analysis among other critical sections. Dejen Received annual rainfall varying from 955.4 mm to 1985.1 mm in the years 2012 and 2000, respectively. Rain fall was so uncertain that it is varying from year to year and even from time to time by virtue of the areas location within the climatic zone.

The material on which the mass is failing happened to be colluvium which is relatively more permeable and thought to contribute to raise ground water table of the site. Even when we consider the most recent rain fall record (2012) of the site to characterize the area, it received the maximum amount in July with 322.8 mm value.

It started dropping even in the month that is expected to receive peak value in August to 267.1 mm. Continuously dropping it reached a value of almost 0 mm in October which is after two months the area received its peak value. In general the area is characterize by uncertain rainfall intensity event and amount which is also affecting the area by virtue of its material characteristics it is made of (colluvium soil of different source rock in this case).

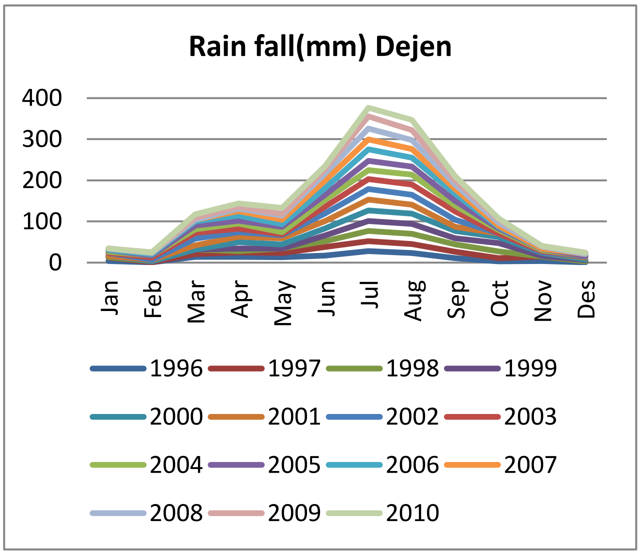


Fig. 1.4 Graph showing monthly rainfall of Dejen.

In the present study the contribution to and sensitivity of rainfall toward slope instability of slopes under considerations has been analyzed for remedial design of the area.

ii) Temperature: The area is characterized by hot and dry climatic condition out of its summer seasons. The mean annual temperature ranges from 5 to 30 ◦C depending on altitude (Yilma and Awulachew, 2009).

1.4.3 Physiography

The geomorphology of the study area is influenced by the geology of the area. Geologically the area is a succession of Mesozoic sedimentary rock underlain by basement rocks and overlain by tertiary basalt. The Mesozoic succession is different from one unit to another in which weak rock material is overlain by competent rock which has probably contributed to the present land siding and that has influenced in shaping the morphology of the gorge.

Several investigators believe that a series of tectonic disturbances during the geological past, together with drainage-aided surface erosion, led the geomorphology of most parts of the northwest Ethiopian highlands including the Blue Nile basin into the form that is observed today (Lulseged Ayalew and Yamagishi, 2003). This conclusion has been made from different geodynamics observed in the gorge but still there is lack of information in providing the principal phases of basin development and/or landscape evolution in the region.

The slope gradient is most commonly greater than 45^{o} and the bedrock is weakly to moderately weathered with weakly developed colluvium soils along the road cut. The ridges are generally convex and the side slopes are straight. Gentle hills consist of gently to moderately sloping hills with colluvium toe slopes on the upper regions of the gorge along the road cut.

On upper most of the two sides of gorge, agricultural practice is undertaken and the area is generally sparsely vegetated.

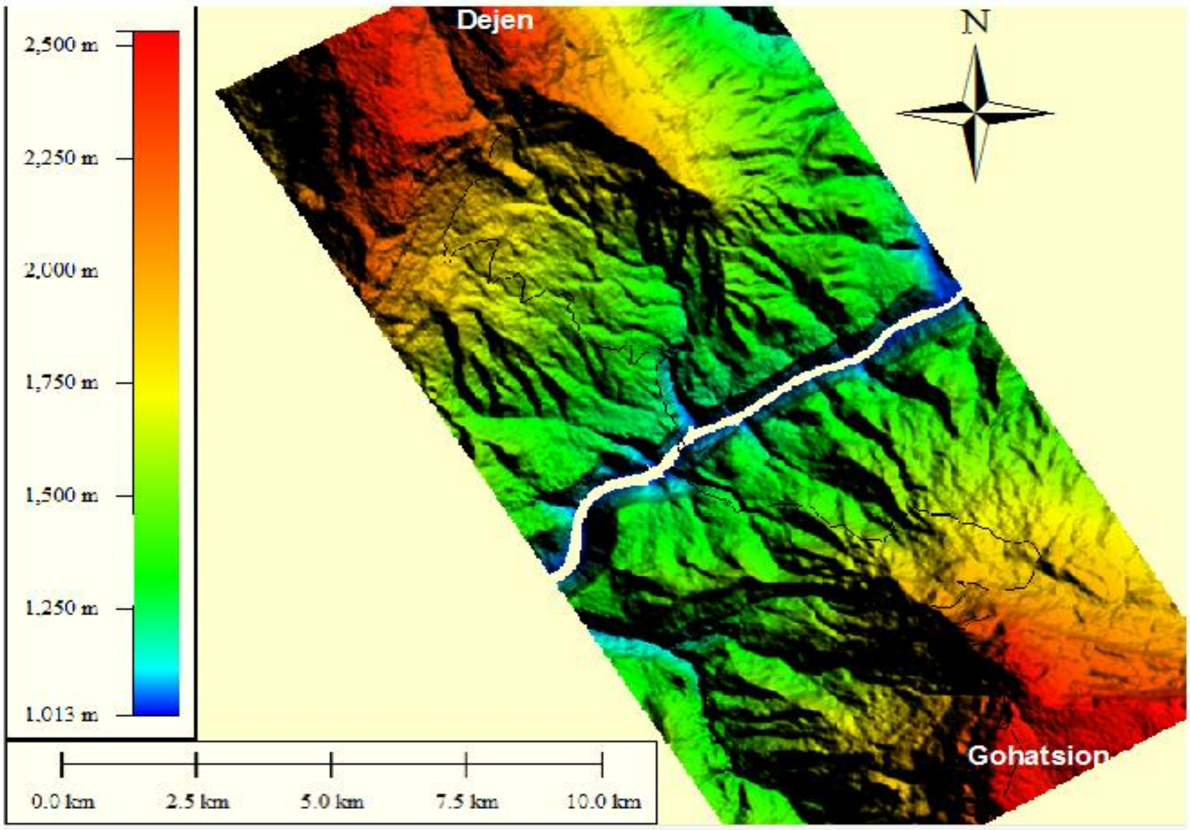
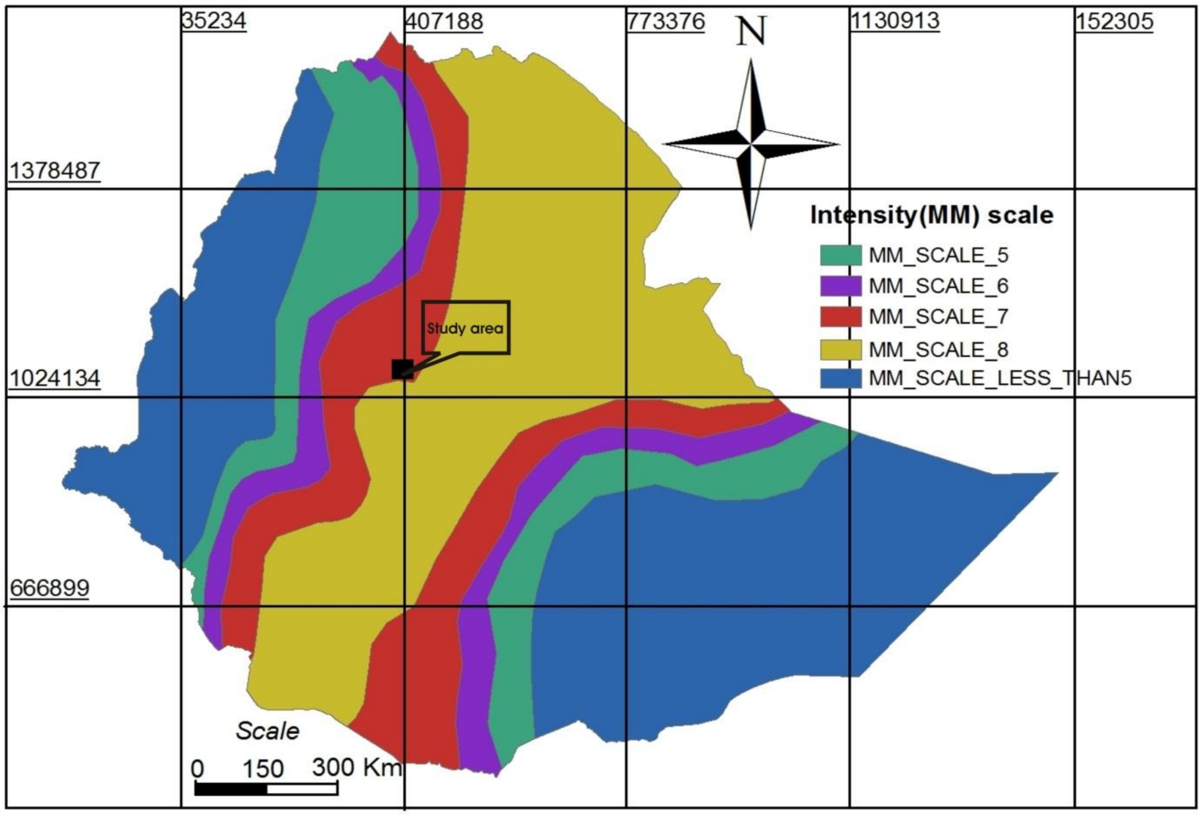


Fig 1.5 Physiographic map of the area

1.4.4 Seismicity of the Study Area

In Ethiopia, seismic activities follow narrow zones associated with structures of the Afar depression and the main Ethiopian rift. The Seismic zone of Ethiopia have been delineated by Gouin (1979) and later updated by (Laike Mariam Asfaw, 1986 Shiferaw Ayale, 2009). The later work of Laike Mariam Asfaw (1986) included the previously omitted earthquake parameters measurements of Southern Ethiopia. In addition, strain release and seismic risk maps has been produced for earthquake from year 1400 to 1985 and 1900 to 1985; the probable return period of destructive earthquakes has been considered and a discussion of some unique features of earthquake hazard in the Afar Depression has been presented (Laike Mariam Asfaw, 1986 as cited in Shiferaw Ayale, 2009).

The ‘Seismic Risk Map' produced by Laike Mariam for a hundred year return period and 0.99 probability (Fig 1.6) shows that the study area falls within 7 M.M scale. Based on the MM scale intensity the estimated horizontal earthquake acceleration comes out to be 0.08g, as determined from the MM intensity graph (Johnson et al., 1988). Hence, the same value has been utilized for the stability analysis of the slopes in the study area for the present study.



(*Source: After Laike Mariam Asfaw, 1986*)

Fig.1.6 Seismic risk map of Ethiopia, 100 year return period, 0.99 probabilities

1.5 Objectives of the Study

A) General Objective:

Considering the previous studies done in reliability engineering related to slope stability on the study area, this thesis was proposed to combine deterministic and probabilistic approaches for stability assessment of slopes. The need of using these combined methods in the study area was for the reason that in deterministic approaches the input variables are assumed to have certain values, however, rock mass parameters are always contain uncertainty and probabilistic approaches offer a systematic way of treating uncertainties and quantifying the reliability of a design (Kirsten, 1983) which were not attempted before. Therefore, both general and specific objectives were framed to come up with the better result by these methods.

The general objective of this research is to develop combined deterministic and probabilistic model for the assessment of active landslides by taking into consideration uncertain geologic parameters resulting from variability and heterogeneity of stratigraphy, material properties and conventional models. Uncertainty in geologic parameters are then justified by the most detailed results of deterministic approach, expressing the hazard in absolute values in the form of safety factors and accordingly delineating the current active site as well as possible future landslide probability occurrence in the areas with their proper remedial measures.

B) Specific objectives:

For the accomplishment of general objective the following specific objectives were proposed.

(i) Delineating critical slope sections

(ii) Collection of relevant data to obtain sufficient information regarding the geological structures and discontinuity patterns and the effect of their orientation on modes of failure (planer and wedge failure analysis or circular if required) for further analysis.

(iii) Determination of geometry of rock cut in different areas to apply the rock fall simulation analysis for the impact of falling rocks on the highway and inclination of incompetent slopes for prediction of future impact.

(iv) To assess the stability of slopes under short term and long term conditions.

(v) Determination of the slopes sensitivity to different triggering mechanisms by combined deterministic and probabilistic approaches.

(vi) To assess the possibility of landslide involving natural or existing engineered slopes in future by using uncertain geologic parameters and zoning the site.

(vii) Develop a mitigation strategy for each sections of the highway and dwellers of the area those settled near to the verge of failure and development of alternative solutions to minimize future problems

1.6 Analytical Tools and Materials Used

To meet out the objectives of the study and for the successful completion of the present research work following analytical tools and materials were used:

- Satellite image was used to mark scarp faces of critical sections from which slope geometry was produced for analysis.
- During data collection for discontinuities on rock mass rating (RMR), Schmidt Hammer was used to estimate the rock strength.
- Brunton compass was used for determinations of slope and discontinuity orientations.
- Slope sections were located with the help of GPS. Also, GPS was used to estimate the slope height on critical sections where the sections were inaccessible.
- For preparations of maps, diagrams and graphics Arc GIS 10, Corel Draw 10, Picasa 3, Micro Soft Excel and Global Mapper 11 were used.
- Eventually, the analysis was carried out by Dip Analyst for kinematic check and by ROCKSCIENCE package software's, Rock PLANE and SLIDE 5 which supports both deterministic and probabilistic approaches for stability analysis.

1.7 Limitations of the Study

The investigation and studies carried out in the present research were in the area defined by Gohatsion and Dejen towns in Abay Gorge of Ethiopia. Therefore, the present research work has to be looked within the geological context of this area. Geologically, the present area can be described as successive Mesozoic sedimentary rocks underlain by basement complex and overlain by tertiary basalts. Further, the present study was conducted under the constrains of time, resource and financial limitations. All these constraints made the work very challenging to study all the critical sections with varying geological, topographical and hydro –geological setup. Therefore, in the present study based on the field manifestations of actual and potential instability only 4 most critical slope sections were selected for combined deterministic and probabilistic stability analysis.

1.8 Scope for Future Studies and Extensions of the Research Work

In spite of the limitations, all efforts were made to produce reliable results with a comprehensive methodology. Due to constrain of time, resources and financial limitations only 4 most critical slope sections were selected and studied during the present study. Thus, there is a need to extend this methodology to cover remaining slopes also in future studies so that entire area can be thoroughly studied for all kinds of slopes and based on the anticipated slope stability conditions appropriate engineering design for slopes can be worked out.

Although the two approaches; deterministic and probabilistic methods are complementary most of the studies done before were based on individual approaches only. Even with in these two individual approaches different methods do exist. It also sounds great if the analysis will be attempted by all available methods within the individual approach and in combination with the other approach to increase reliability of the approaches for detailed and site specific studies.

1.9 Research Work Presentation

Chapter 1 of the thesis introduces some background on slope failures followed by problem statement, objectives, general methodology, limitations, scope for future studies and extension of research work and the general outline of the study area.

Chapter 2 deals with literature review. Previous works related to the current study has been reviewed from which conceptual frame work was developed. General slope instability contributing factors are also presented.

Chapter 3 focuses on geological set up of the study area. Regional geology has been summarized, followed by discussion on local geology.

Chapter 4 covers the methodology development for the present study. All the analysis procedures and in what way, what manner the combined approaches are used is discussed.

Chapter 5 is about results and discussion. It is the core of the present study and covers the description on input data, analysis procedures, results and interpretations of the present research. A detailed description on results and interpretation with respect to the objectives of the study is also presented.

Chapter 6 deals with possible remedial measures and preventive options for the critical slope sections in the study area.

Chapter 7 of the thesis is about conclusion and recommendations. Conclusions were made from the overall work accomplished during the present study. Finally, based on the findings general recommendations for implementation is forwarded and presented in this chapter.

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Chapter 2

Literature Review

2.1 Preamble

In hilly terrains slope instability is a major problem all over the world (www.icsu-asia-pacific. org/resource_centre/Sassa-paper.pdf; www.emdat.be; Schuster and Fleming, 1986; Keefer, 2000; Mario and Jibson, 2000; Dai et al, 2002; Kanungo et al., 2006; Pan et al., 2008). The same is true in Ethiopia particularly in the highlands of the country (Engdawork Mulatu et al., 2009; Ayalew and Yamagishi, 2003; Henok Woldegiorgis, 2008). The Blue Nile Basin in Ethiopia witnesses a wave of landslides both small and large scale during the rainy season. Such landslides and rock fall have been witnessed starting from some 150 km downstream of Lake Tana up to the Sudan border (Ayalew and Yamagishi, 2003).

Slope instability is a geo-dynamic process that naturally shapes up the geo-morphology of the earth. It became a major concern with the increased demand for development and expansion of human settlements in those unstable slopes and as these areas posed an effect on the safety of people and property (Weerasinghe et al., 1999).

In the present study the work of different researchers from different parts of the world and highlands of Ethiopia on individual deterministic and probabilistic approach as well as the output they have produced through comparative studies considering various instability contributing factors and uncertainties in earth materials is reviewed.

2.2 Landslide and Related Slope Instability Assessment Approaches

The assessment of an existing unstable or potentially unstable slope, or of a slope to be cut, provides the basis for the selection of slope treatments. Treatment selection requires forecasting the form of failure, the volume of material involved, and the degree of the hazard and risk. Assessment can be based on quantitative analysis in certain situations, but in many cases must be based on qualitative evaluation of the slope characteristics and environmental factors including weather and seismic activity (Hunt, 2005).

As per Banakia et al., (2013) there are two types of stability analysis against failure; that is ‘Deterministic’ and ‘Probabilistic’ analysis.

2.2.1 Deterministic Approaches

Deterministic analysis are traditional approaches based on calculation of stabilizing and driving forces and a resulting factor of safety, has long been most common in stability analysis. This analysis approach includes;

(i) Limit Equilibrium Methods (LEM)

Most deterministic analytical methods applied to evaluate slope stability are based on limiting equilibrium, i.e., on equating the driving or shearing forces due to water and gravity to the resisting forces due to cohesion and friction (Hunt, 2005). Further, Cheng and Lau (2008) classified the limit equilibrium method into two main categories: ‘simplified’ methods and ‘rigorous’ methods. For the simplified methods, either force or moment equilibrium can be satisfied but not both at the same time. For the rigorous methods, both force and moment equilibrium can be satisfied, but usually the analysis is more tedious and may sometimes experience non-convergence problems. Besides, (Whitman and Bailey, 1967 as cited Abramson et al., 2002) presented a very interesting and classical review of the limit equilibrium analysis methods, which can be grouped as; Method of slices and wedge method (less commonly used for slopes). Hence, in the method of slices the unstable soil mass is divided into a series of vertical slices and the slip surface can be circular or polygonal.

Methods of analysis which employ circular slip surfaces include; Fellenius (1936); Taylor (1949); and Bishop (1955). Methods of analysis which employ non-circular slip surfaces include: Janbu (1973); Morgenstern and Price (1965); Spencer (1967); and Sarma (1973) as cite in Hunt, (2005).

Table 2.1 Static equilibrium conditions satisfied by limit equilibrium methods

Methods	Force equilibrium		Moment equilibrium
	X	Y	
Ordinary method of slice(OMS)	No	No	Yes
Bishop’s simplified	Yes	No	Yes
Janbu’s simplified	Yes	Yes	No
Corps of engineers	Yes	Yes	No
Spencer’s	Yes	Yes	Yes
Bishop’s rigorous	Yes	Yes	Yes
Janbu’s generalized	Yes	Yes	No
Sarma’s	Yes	Yes	Yes
Morgenstern-price	Yes	Yes	Yes

i) Finite Element Method (FEM)

In the classical limit equilibrium and limit analysis methods, the progressive failure phenomenon cannot be estimated except for the method by Pan. Hence, finite element method has been proposed to overcome some of the basic limitations in the traditional methods of analysis. At present, there are two major applications of the finite element in slope stability analysis.

The first approach is to perform an elastic (or elasto-plastic) stress analysis by applying the body force (weight) due to soil to the slope system. Once the stresses are determined, the local factors of safety can be determined easily from the stresses and the Mohr–Coulomb criterion. The global factor of safety can also be defined in a similar way by determining the ultimate shear force and the actual driving force along the failure surface.

The second finite element slope stability approach is the strength reduction method (SRM). The main advantages of the SRM are as follows: (i) the critical failure surface is found automatically from the localized shear strain arising from the application of gravity loads and the reduction of shear strength; (ii) it requires no assumption on the inter-slice shear force distribution; (iii) it is applicable to many complex conditions and can give information such as stresses, movements and pore pressures which are not possible with the LEM (Cheng and Lau, 2008).

ii) Limit Analysis Method

The limit analysis adopts the concept of an idealized stress–strain relation, that is, the soil is assumed as a rigid, perfectly plastic material with an associated flow rule. Without carrying out the step-by-step elasto-plastic analysis, the limit analysis can provide solutions to many problems.

2.2.2 Probabilistic Approaches

As a direct integration of the adopted performance functions for the ‘Factor of Safety’ (FOS) is not feasible, the expected values and variance of the factor of safety may be approximated probabilistically (Sharma, 2002) by;

i) Monte Carlo Simulation Method

This method effectively simulates the response of the factor of safety performance function to randomly selected, discrete values of the component variables. The process is repeated many, many times to obtain an approximate, discrete, ‘Probability Density Functions’ (PDF) of the resulting FOS values, F. The component random variables for each calculation are selected from a sample of random values that are based on the selected PDF of the random variables.

Although these probability density functions can take on any shape, the normal, log normal, triangular, beta, and uniform distributions are used for analysis (Abramson et al., 2002).

ii) Taylor Series Method

This method is based on the Taylor series expansion of the performance function about the expected values of the random variables (Hahn and Shapiro, 1967 as cited in Abramson et al., 2002). The main advantage of the Taylor series method is that as the terms are summed for the variance, one can readily see the relative contribution of the uncertainty projected by each component random variable. Although this method will give an exact solution for a linear function, the first order approximations introduces errors for nonlinear performance functions, such as; the ones typically used for the FOS (Abramson et al., 2002).

iii) Rosenblueth's Point Estimate Method

It is originally proposed by Rosenblueth (1975) where the PDFs of the random variables are simulated by "point" masses located at plus or minus one standard deviation from the mean values. This method is direct and gives reasonably accurate results quickly (McGuffey et al., 1981; Harr, 1987; wolf, 1996 as cited in Abramson et al., 2002).

iv) Fourier Analysis

If the complete joint PDF of the FOS is required rather than a Monte Carlo histogram of possible FOS values, then one can use the Fourier analysis method, which is based on the evolution of independent PDFs (Feller, 1966 as cited in Abramson et al., 2002). The main difficulty with this Fourier method is the requirement of a linear performance function. However, if a linear performance function can be identified, then an accurate, unique PDF of the will be obtained, without any restriction on the type of input PDF that may be used to define the component random variables. This allows the use of highly skewed PDF if necessary (Abramson et al., 2002).

2.3 Studies conducted by Deterministic and Probabilistic Approaches – A Review

Parka and West (2000) have utilized probabilistic approach for rock wedge failure in South Korea. Water pressure, joint orientation, joint dimension, and joint shear strength were considered as probabilistic input parameters to evaluate probability of slope failure in the study. In order to compare the probability of slope failure with the traditional deterministic method, the Factor of Safety (FS) for each case was calculated.

A total of six discontinuity sets were identified and for stability analysis, all possible discontinuity combinations were delineated and checked for stability. Based on the deterministic analysis results, it was concluded that the slope is stable and there is no possibility of instability however; through probabilistic approach a low volume wedge failure probability was anticipated.

MatRadhi et al (2008) have developed Probabilistic Approach for Rock Slope Stability Analysis Using Monte Carlo Simulation and tested it by using data from Pos Selim Highway, Malaysia. The probabilistic analysis was carried out using kinematic and kinetic analysis. Kinematic analysis is based on stereographic projection analysis and kinetic analysis is based on the deterministic analysis. From the total of six slope data collected, three of them had Factor of safety less than unity which shows the slope is un-stable with possible planar failure mode. Whilst for wedge failure analysis, all the slopes show FS greater than 1.00 indicating stable condition. Further, Probabilistic analysis was developed for rock slope stability using Monte Carlo Simulation. Monte Carlo simulation calculate the probability of failure for planar and wedge type of failure. Finally, it was concluded that as factor of safety increases the probability of failure decrease in variable ranges depending on dry or wet slope conditions.

Gheibie (2012) utilized Probabilistic-numerical modeling for stability of a rock slope in Amasya Turkey. He considered Joint Roughness Coefficient (JRC), Joint Compressive strength (JCS), cohesion and angle of internal friction of the filled material. The analysis indicated that the increase of cohesion decreases the displacements and failure probability of the structure. Also, reduction of basic friction angle and joint wall compressive strength increases the probability of failure and increases the displacement.

Ying (2002) studied slope stability by classical limit equilibrium methods through analysis by considering various equations, unknowns and assumptions. The study also covered the suitability of various methods of analysis, factor of safety calculated by considering different examples from Hong Kong. The influence of tension crack, external loading, ground water fluctuation on slope stability was also considered.

2.4 Landslide studies in Ethiopia – A Review

Landslide and related slope instability problems are common in highlands of Ethiopia. Several studies has been conducted by various organization, instaurations and individuals in past by following various approaches. A review of all such works was made during the present study and is summarized in the following paragraphs;

Gebretsadik Eshete (1982) conducted landslide studies in Dessie area. According to him two of the common causes which seem to play a considerable role in causing instabilities in the Dessie area are the water content and weathering. The water content at the time of rain attributed to excess hydrostatic pressure which reduces the shearing resistance. The formation of clay as a result of weathering in joints and faults can reduce the resistance and causes sliding as a consequence.

Gezahegn and Dessie (1994) classify slope instabilities in the Abay Gorge and its tributaries in to four types. These are (i) Continuously moving granular deposits from the slopes of basalt escarpments, (ii) Rotational failure of colluvial soil, (iii) Gully erosion and (iv) Rock fall and toppling. Based on their investigation they proposed a road realignment which relatively avoided the landslide hazard zone.

Lulseged Ayalew and Vernier, (1999), has studied the causes and mechanisms of slope instability in Dessie town. In this work, they examined the relationship between slope instability and seasonal rainfall in the area. By taking the variation in moisture content of soil, the rate of daily precipitation, the amount of cumulative precipitation, and mean annual rainfall as variable, they derived a simple equation that is useful to determine the likelihood of land sliding.

Getachew Lemmesa et al (2000) conducted mass movement hazard assessment in Betto, Goffa district, North Omo Zone, Southern Ethiopia. The study identified that the main cause of landslide was the existence of old landslides on steep slopes that was covered by deeply weathered, closely jointed or sheared basaltic rocks.

Berhanu Temesigen et al (1999) conducted a research on landslide in the Wondo-Genet area. In this research evaluation of the occurrences of landslides and their relationships with various event controlling parameters was made using GIS and Remote Sensing techniques.

Lulseged Ayalew and Yamagishi (2003) described slope failures in the Abay Gorge from the point of view of landscape evolution. In their study they related topographical characteristics with the process of landslide and rock fall. They concluded that slope instability was part of the mega-forces that shaped the entire Abay river basin and that it also contributed to general landscape evolution.

Tenalem Ayenew and Barbieri (2004) have conducted an inventory landslides and susceptibility mapping in the Dessie area. During this study, four broad landslide susceptibility zones and 22 specific active landslide sites were identified. According to this research, the most important landslide types were complex earth and debris slides and flows in silty clay soils associated with alluvial and colluvial deposits overlying highly weathered basalts.

As cited by Jemal Saed (2005), the Transport Construction Design Share Company (TCDSCo) carried out detailed geotechnical investigation project along the Gohatsion-Dejen-Debre Markos road in 2003. This study revealed that thick unconsolidated colluvial soil mass was responsible for the damage of the northern parts of the road.

Yodit Teferi (2005) conducted a research on Evaluation of Land Degradation and Landslide Using Integrated GIS and Remote Sensing Approach around Sodo-Shone Area, Southern Ethiopia. During her research she observed two types of mass movements; flow and rock fall in the area. Mud flow was induced by intensive rainfall and observed in gently sloping areas whereas the slope mainly ranges from 200-300 m whereas the rock fall was common in areas where the slope angle exceeds 40^0.

Jemal Saed (2005) has conducted slope stability studies on the road section starting from Gohatsion to Dejen towns. He made the quantitative analysis of critical slope sections following limit equilibrium method.

Kifle Woldearegay (2008) carried out geological, geotechnical and geo-hydrological investigations to understand the causes and triggering mechanisms of the large-scale landslide in Tarmaber area, central highlands of Ethiopia. According to his research paper, the localities “Yizaba Wein” and “Shotel Amba” areas, with an estimated total area of 35 square kilometer, were completely affected by a single major deep-seated landslide which took place in September 12, 2006. More than 3000 people were displaced; 1250 dwelling houses were demolished; and 4 Churches, 4 Mills, and one elementary school were destroyed. The landslide also devastated about 1500 hectare of agricultural land and caused damage to the natural environment.

Henok Woldegeorgis (2008) had made landslide hazard zonation mapping in southern part of Blue Nile Gorge using a Land Hazard Evaluation Factor (LHEF) to characterize the Landslide Hazard potential in the study area. By utilizing the limit equilibrium method he further made the quantitative analysis for critical slopes.

Shiferaw Ayele (2009) utilized remote sensing and GIS approach to delineate Landslide Hazard zones in Abay Gorge (Gohatsion-Dejen), Central Ethiopia. The various causative factors considered for this study were; geology, groundwater condition, drainage, slope, structures, aspect and landuse/ land cover. In this study comparison of the landslide hazard map was made with actual landslide events of the study area and found that 67% landslides lie within the maximum hazard zone delineated by the study.

Fikre Girma (2010) conducted study in Ada Berga Woreda, Western Showa Zone, Oromiya Region, Ethiopia by utilizing A Multi Method Approach to Study Landslide Hazard. For this study various GIS and statistical tools were utilized to produce the landslide hazard zonation map.

Jemal Ibrahim (2009) conducted study in Mersa and Wurgessa Area, North Wollo, Ethiopia and carried out Landslide assessment and hazard zonation. For landslide hazard evaluation he utilizes slope susceptibility evaluation parameter scheme.

Samuel Molla, (2011) conducted Slope stability analysis on a selected slope section along the road Gohatsion – Dejen. He utilized limit equilibrium method for the quantitative assessment of slope stability condition for various anticipated adverse conditions.

Yonathan Tsegaye (2011) conducted study in Tarmaber Area in Northern Ethiopia and produced Landslide Hazard Zonation Map by utilizing Landslide Hazard Evaluation factor scheme.

Lensa Negassa (2012) conducted Landslide Hazard Zonation Using Remote Sensing and GIS Approach in Meta Robi Wereda, West Showa Zone, Oromiya, Ethiopia. The study utilized Grid overlay analysis and GIS modeling tool. The various causative and triggering factors were quantitatively assessed and evaluated to produce landslide hazard zonation map.

Similar landslide or slope stability studies were carried out by Lulseged Ayalew, (1999), Kefeyalew Terefe, (2001), Gebretsidik Eshete, (1982), Mesfin Wubshet et al., (1994) etc.

2.5 Contributing factors for slope instability

It is difficult to determine the type and likelihood of a slope failure event because numerous factors interact in complex and often subtle ways to destabilize slopes. Despite the challenges, many factors has been proved to be responsible to the instability of slopes which are attributed to processes that increase shear stress or decrease shear strength of the soil or rock mass (Abramson et al., 2002). In slope stability analysis, it is of fundamental importance to identify causative factors for slope failure occurrences in a region, which often is difficult. It is also usually hard to establish the relationships between various causative factors. Indeed, the great variety of slope movements reflects the diversity of factors that may disturb the slope stability (Arora and Anbalagan, 2010).

However, it may be possible to demarcate landslide susceptible areas by identifying and analyzing the factors that have caused landslides in the past and under similar conditions in the future. It is of primary importance to understand the conditions, under which mass movements are caused and the factors that trigger the movements to recognize the extent of danger and to propose adequate remedial measures. The major driving force that affects the shearing stresses in a slope, gravity, is highly correlated to slope gradient. When the slope gradient increases, the gravity and shearing stresses increase as well. A number of natural and anthropogenic factors can significantly reduce stability of slopes and contribute for slope failure (Aleotti and Chowdhury, 1999).

The major factors are discussed as below;

2.5.1 Geological factors

It is an important geological parameter as it is related to the basic characters of the slope forming materials. There are two fundamental types of slope forming materials – loose, unconsolidated materials and in-situ rocks. The unconsolidated materials, except older fluvial materials, in general, have least shear strength and are more prone to failure. Particularly, if they are charged with water, they show high potential to failure. The rocks are in general more stable as compared to unconsolidated material.

The disposition of the structural discontinuities of primary and secondary origin in the rocks such as; bedding, joints, foliations, faults and thrusts in relation to slope inclination and direction has a great influence on the stability of slopes (Arora and Anbalagan, 2010). Many slides occur in a geologic setting that places permeable sands and gravels above impermeable layers of silt and clay, or bedrock. Water seeps downward through the upper materials and accumulates on the top of the underlying units, forming a zone of weakness.

2.5.2 Geomorphic factors

Geomorphic factors include slope shape and aspect. The distribution of slope categories is dependent on the geomorphological history of the area. The angle of slope of each unit is a reflection of a series of localized processes and controls, which have been imposed on the slope (Long, 2008).

Slope shape has a strong influence on slope stability in steep terrain by concentrating or dispersing surface and primarily subsurface water in the landscape. There are three basic hydro geomorphic slope units: (i) divergent, (ii) planar or straight, and (iii) convergent. The divergent or convex landform is most stable in steep terrain, followed by planar hill slope segment and convergent or concave hills slope (least stable). The main reason is related to landform structure affecting largely the concentration or dispersion of surface and sub-surface water. Convergent hill slope tend to concentrate sub-surface water into small areas of the slope, thereby generating rapid pore water pressure increase during storms or periods of rainfall. If pore pressures develop in the hollow, the soil shear strength reduces to a critical level and a landslide can occur. Hence, hollows are susceptible sites for initiation of debris slide and debris flows (Hack and Goodlett, 1960; Dietrich and Dunne, 1978; Benda, 1990 as cited in Long, 2008).

Slope aspect (direction) strongly affects hydrologic processes via evapotranspiration and thus affects weathering processes and vegetation and root development, especially in drier environments (Sidle and Ochiai, 2006). Altitude or elevation is usually associated with landslides by virtue of other factors such as; slope gradient, lithology, weathering, precipitation, ground motion, soil thickness and land use. The relative relief (gradient) refers to the local height of the slope between the ridge top and valley floor in a slope facet. It has also an important role in forming the size of the unstable wedges. The slope having higher relative relief may form unstable rock wedges of big size with more probability of failures (Long, 2008).

2.5.3 Hydrologic factors

Hydrologic processes in the form of precipitation (spatial and temporal distribution of rainfall), water recharge into soil (and the potential for overland flow), lateral and vertical movement within the regolith, evapotranspiration and interception contribute to slope instability (Long, 2008). Water is commonly the primary factor triggering a landslide. Slides often occur following intense rainfall, when storm water runoff saturates soils on steep slopes or when infiltration causes a rapid rise in groundwater levels. Groundwater may rise as a result of heavy rains or a prolonged wet spell. As water tables rise, some slopes become unstable due to increase in slope saturation building up positive pore water pressure. This causes a decrease in the effective normal stress acting along the potential failure plane, which in turn diminishes the available shear strength to a point where equilibrium can no longer be sustained in the slope (Orense, 2004).

Hill slope hydrology is one of the most important factors in landslide triggering. Two processes mainly control changes in slope hydrology that may lead to failure: infiltration of water from the surface and rising of groundwater levels by impoundment of water in a reservoir. However, in case of sub-surface water, it is only the shallow water close to the surface that is important from the point of landslides as they can sufficiently reduce strength of surface materials. The sub-surface water flowing at deeper levels which are not day-lighted on surface is not important to be considered in slope stability analysis (UNIMIB, 2012).

Another important influence of water is to reduce angle of repose. The angle of repose is the steepest angle at which a pile of unconsolidated grains remains stable, and is controlled by the frictional contact between the grains. Hence, when the material becomes saturated with water, the angle of repose is reduced to very small values and the material tends to flow like a fluid. This is because the water gets between the grains and eliminates grain to grain frictional contacts. In general, for dry materials the angle of repose increases with increasing grain size, but usually lies between about 30 and 37^{o} (Upreti & Dhital, 1996).

2.5.4 Land use and land cover

Land cover is an indirect indication of the stability of hill slopes. The thickly vegetated forest areas are less prone to erosion and are generally more stable. However, the barren and sparsely vegetated lands are more prone to erosion and instability (Anbalagan, 1992). Forest cover smoothers the action of climatic agents on the slope and protects them from weathering and erosion. A well spread-out root system increases the shearing resistance of the slope materials. However, the growth of plants and other vegetation in the pre-existing plane of joints of the rocks may also cause excess stress on joint walls due to increase in size of roots. This phenomenon may push the slope materials out and cause landslides (Anbalagan, 1992).

2.5.5 Earthquakes

Seismic activities have always been a main cause of landslides throughout the world. Any time plate tectonics move the soil that covers and moves with it. When earthquakes occur on areas with steep slopes, many times the soil slips causing landslides. Earthquakes in steep landslide-prone areas greatly increase the likelihood that landslides will occur, due to ground shaking alone, liquefaction of susceptible sediments, or shaking-caused dilation of soil materials, which allows rapid infiltration of water. Rock falls and rock topples can also be caused by loosening of rocks or rocky formations as a result of earthquake ground shaking (Highland and Bobrowsky, 2008). Furthermore, ash and debris flows caused by earthquakes can also trigger mass movement of soil.

There is one type of landslide that is typical for earthquakes, i.e. liquefaction failure which can cause fissuring or subsidence of the ground. Liquefaction involves the temporary loss of strength of sands and silts which behave as viscous fluids rather than as soils. This can have devastating effects during large earthquakes (Long, 2008).

2.5.6 Human Intervention

Human activity can modify by remediating the influence of natural processes such as; rainfall or toe erosion on slope stability but on the contrary, it can also substitute the natural processes with an artificial equivalent, typically accelerated by several orders of magnitude compared to natural timescales, as in the case, for example, of major earthworks or other construction activities on slope (Gorsevski et al., 2006).

Landslides may result directly or indirectly from the activities of people. Among them are improper or uncontrolled discharge from sanitation or drainage works and water pipes, typically associated with human settlements and roads, especially in rural areas, which increase water infiltration in the slope, and possibly soil erosion at points of concentrated surface discharge. Excavation at the base of or on slopes and/or placement of fill at the top or on slopes, altering the geometry of and the stress distribution in the slope is also another influence. Quarrying and mining activities, altering the geometry of and the stress distribution in the slope is an important factor. Vibration from blasting and heavy construction equipment; besides causing transient stress changes which in extreme cases can trigger landslides by themselves, vibrations can disturb the microstructure of soils or rocks, induce fracturing in rock (UNIMIB, 2012)

Farming is generally considered as having a low impact on landslides and is often the only activity allowed in landslide prone areas. Even farming and collateral activities, however, may trigger slope movements, directly or indirectly. When land is first turned to farming, this often involves extensive deforestation which can lead to soil erosion and land sliding in hilly or mountainous terrain (Glade 1998). However, probably the main cause of landslide triggering due to farming is related to irrigation and drainage, which can alter the natural groundwater conditions (UNIMIB, 2012).

Removal of vegetation for change of land use to agriculture can also facilitate slope instability, reducing evapotranspiration, allowing increased infiltration and increased surface erosion through loss of the shallow reinforcement provided by the root system (UNIMIB, 2012).

2.2.6 Weathering

Factors influencing the rate of weathering are among others climate (temperature and rainfall), time and type of source rock (Lambe and Whitman, 1969). Weathering is the natural process of breaking down rocks into soil through freeze-thaw, Wet-dry, and the slow action of physical and chemical deterioration. Weathering has a large effect on the soil physical properties that control land sliding such as texture, porosity and shrink-swell properties. Weathering processes those that decompose the rock act as internal control that decreases rock’s shearing strength and contribute to a low and/or reduced strength (Varnes, 1984).

Rocks exposed to difference in weathering changes from unweathered rock to soil. The depth of this weathering is shallow for rocks like basalts that are fine grained. This condition leads to surface sliding generally triggered by heavy rain fall (Cornforth, 2005). Cut slopes made through sedimentary rocks may pass through sedimentary rock, partly to fully cemented conglomerate, indurated sand, clay shale, etc. Over a years differential weathering may erode the less resistant rock layer and under mining the slope above. Seepage occurring between two layers of differential permeability can accelerate the weathering and erosion at the face.

2.6 Methodology Evolved for the Present Study

The thorough literature review made in the present study was to have a general background and to evolve conceptual framework about landslide types, factors responsible for instability and to know about various techniques generally used for analysis of slope stability.

Thus, through this literature survey a general conceptual framework was developed and a comprehensive methodology was evolved. The methodology followed in the present study was based on the actual field data collection, procurement of data from secondary sources, exhaustive analysis through standard graphical, analytical and empirical approaches, logical interpretations of results and actual result based evaluation of feasible recommendations. For analysis purpose deterministic and Limit equilibrium based probabilistic Monte Carlo simulation approach was followed during the present study. Eventually, these combined methods were applied to the identified critical slope sections in the study area and based on the results suitable remedial measures were suggested.

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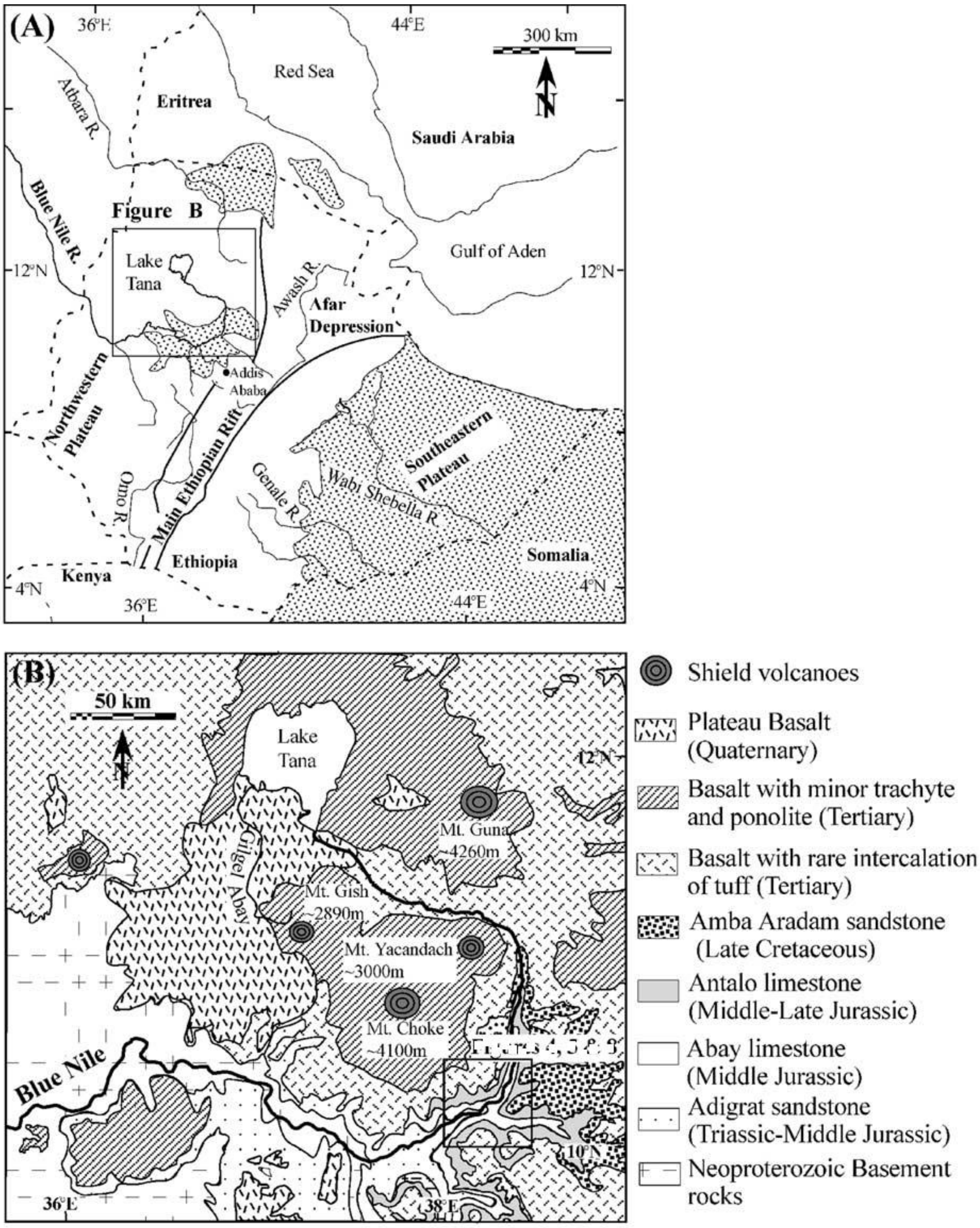
Chapter 3

Geological setup of the Study Area

3.1 Regional Geology

The Blue Nile Gorge of Ethiopia is located in the central part of North-western plateau of Ethiopian physiographic region. The evolution of the basin was associated with Triassic–Cretaceous time NE–SW trending lithospheric extension that affected northern and central Africa (Fairhead, 1988). This extension formed NW-trending Mesozoic rift basins including the Muglad, the Melut, the Blue Nile and the Anza rift basins (Mc Hargue et al., 1992; Binks and Fairhead, 1992 as cited in Gani et al., 2008). Although the architecture of Blue Nile basin is poorly known, many studies indicate as it was formed during Mesozoic break-up of Gondwana (Bosellini, 1989; Russo et al., 1994). Regardless of important studies on Blue Nile, the stratigraphic and structural evolution of the Basin is not fully understood since much of the basin's geological record (Mesozoic and Precambrian rocks) is buried beneath the extensive 500–2000 m thick Cenozoic volcanic rocks (Hofmann et al., 1997; Coulie et al., 2003; Kieffer et al., 2004) and no sub-surface data are available. However, the ~ 1600 m deep Gorge of the Nile (Gani and Abdelsalam, 2006; Gani et al., 2008) formed by the never ceasing forces of erosion and transportation Blue Nile River on the North-western Ethiopian Plateau provides good surface exposures suitable for focused stratigraphic and structural studies that can be used for regional reconstruction of the geological history of the Blue Nile Basin.

Geologically, the gorge contains the complete stratigraphy of Ethiopian geology starting from Precambrian basement to Mesozoic sediment overlain by Tertiary basalt (Mogessie et al., 2002). The age of the basement rocks are considered to be Neoproterozoic, ranging from 850 to 550 Ma as documented from U-Pb and Rb-Sr geo-chronologic studies (Tenalem Ayalew et al., 1990). These rocks underlie the Mesozoic sedimentary rocks. The Mesozoic sedimentation phase started following continuous subsiding period of the land and migration of the sea from east, in the Ogaden towards the west and north covering the central part and northern areas of the country. Blue Nile River basin is among the places in the country where Mesozoic sediments are exposed. It is characterized by ~1400 m thick horizontal to sub-horizontal successions of fluvial/ alluvial silica clastic and marine carbonate rocks, ranging in age from Triassic to Cretaceous and showing evidence for different phases of marine transgression and regression (Gani et al., 2008). The oldest recorded deposits above the basement are Paleozoic (Ordovician) continental sediments (Jepsen and Athearn, 1961). Fig. 3.1 shows the regional geological setup of the area.



(A) Distribution of Mesozoic sedimentary rocks in the Horn of Africa.
(B) Generalized geological map of the Gorge of the Nile (after Gani et al., 2008).
The rectangular box shows Gohatsion-Dejen part of Blue Nile gorge.

Fig. 3.1 Regional Geological Map

The Mesozoic sedimentary rocks exposed along the Gorge of the Nile are overlain by Mid-Oligocene to Early Miocene volcanic flows known as the Trap Series, the thickness of which reaches in some places ~2000 m (Coulie et al., 2003) as cited in Gani et al., 2008.

The depositional evolution of the Gorge of the Nile including Mesozoic sedimentary rocks has been modified from Russo et al., (1994) by Gani et al., (2006). Accordingly, it started with; (i) Peneplanation of the Neoproterozoic basement outcrops; (ii) Post-rift stage (related to Gondwana rifting), in which fluvial sedimentary rocks of the Triassic-Middle Jurassic Lower Sandstone covered the Neoproterozoic basement rocks; (iii) First marine transgression indicated by the glauconite bearing sandy mudstone beds deposited on top of the Lower Sandstone; (iv) Early flooding stage and deposition of the Middle Jurassic Lower Limestone which includes gypsum layers indicating drying of the basin; (v) Second phase of marine transgression when the Middle-Late Jurassic Upper Limestone has been deposited; (vi) Regression of the sea leading to the deposition of the Late Cretaceous Upper Sandstone in a fluvial condition; (vii) Basaltic lava extrusion during the Middle Oligocene to Early Miocene related to deep mantle plume activity (Marty et al., 1996; Ritsema and Heijst, 2000) and regional tectonic activity associated with the East African Rift System; and (viii) Second volcanic event leading to emplacement of the Quaternary volcanic rocks, which is one of the important Quaternary tectonic events in the Blue Nile region.

Getaneh Assefa (1979, 1991) has established informally five formations for Mesozoic sediments of Abay Gorge. These include;

i) Adigrat Sandstone

Also known as the Lower Sandstone unit, un- conformably overlies the basement and in some places Paleozoic continental sediments. The thickness ranges from about 100 m to 700 m; and particularly ~ 300m in the Abay River Basin. It is a resistant unit forming nearly vertical cliff on both sides of the river section. This unit consists mostly of sandstone intercalated with layers of silt stone, mudstone and rare beds of conglomerate and shale (Russo et al., 1994).

ii) Shale and Gypsum unit: (Strata of Abay of Krenkel 1926, or Gohatsion formation of Geteneh Assefa, 1980).

This unit has ~ 450 m thickness consisting variable geological materials in it. The lower part of the unit consists of inter-bedded silt stone and shale. The middle part consists of bio-turbated mudstones, coquinoid siltstones which are very rich in peleycypods, and chicken wire and laminated gypsum. The upper part includes barren greenish, reddish clays and silt stone (Russo et al., 1994).

iii) Limestone unit

This unit consists of ~ 600 m thick fossiliferous carbonate inter bedded with marl, shale, and mudstone. The top most part of the unit which is oolitic, massive and cliff forming limestone was named as Lagajimma Limestone by Aubry, (1886) as cited in Getaneh Assefa, 1991. The maximum thickness of this unit part is 170 m. This hard limestone unit was formed in shallow water, which is documented by the occurrence of oolitic bars, coral off shore patches and more protected inshore facies. Boundaries on both sides of this unit are transitional. The rest of the formation is dominated by marl and is thus informally known as marly limestone. Both boundaries of these units are translational environmentally (Getaneh Assefa, 1991).

iv) Muddy Sandstone unit or Mugher Mudstone Unit

It is named after ‘River Mugher’ which is one of the tributaries of Abay River. The formation rests conformably on lower Kimmeridgian laga Jimma limestone. It is either conformably overlain by Debrelibanos sandstone or dis conformably overlain by the Tertiary basalts. The unit is un-fosiliferous except the occurrence of wood fragments at few localities.

The mudstone is varicolored, including yellow, green, red, tan, white, and mottled as well as purple, brownish, red, and various shades of gray. It is also characterized by desiccation cracks, calcareous concretions, and numerous micaceous surfaces, subcuboidal blocky structure, and Mottling. The muddy sandstone unit or Mugher Mudstone is not present in some section but it is well exposed west of the confluence of the Bale and Mugher rivers (Getaneh Assefa, 1980).

v) Debrelibanos Sandstone Unit: (Also known as Upper Sandstone Unit)

This unit has variable thickness from place to place ranging from 0 m near Gohatsion to 280 m near Lemi (Getaneh Assefa, 1991). It is composed of fine to medium grained sandstones with local lenses of clay stone and conglomerates. Chronologically, since no bio-stratigraphic or radiometric age data are available, this unit is determined to be of Late Jurassic–Early Cretaceous age based on its stratigraphic relationship with overlying and underlying units (Getaneh Assefa 1991; Russo et al., 1994).

The Upper Sandstone shows dune-scale trough cross-bedding and horizontal stratifications. Distinct pebbles horizons are locally present and small channels with lateral accretion surfaces are rarely observed. The overall depositional environment of this unit is interpreted to be continental alluvial to fluvial. Therefore, the unconformity (disconformity) at the base of this unit marks a regional regression when rocks of Early Cretaceous are absent (Russo et al., 1994).

3.2 Regional Tectonic Setup and Structures

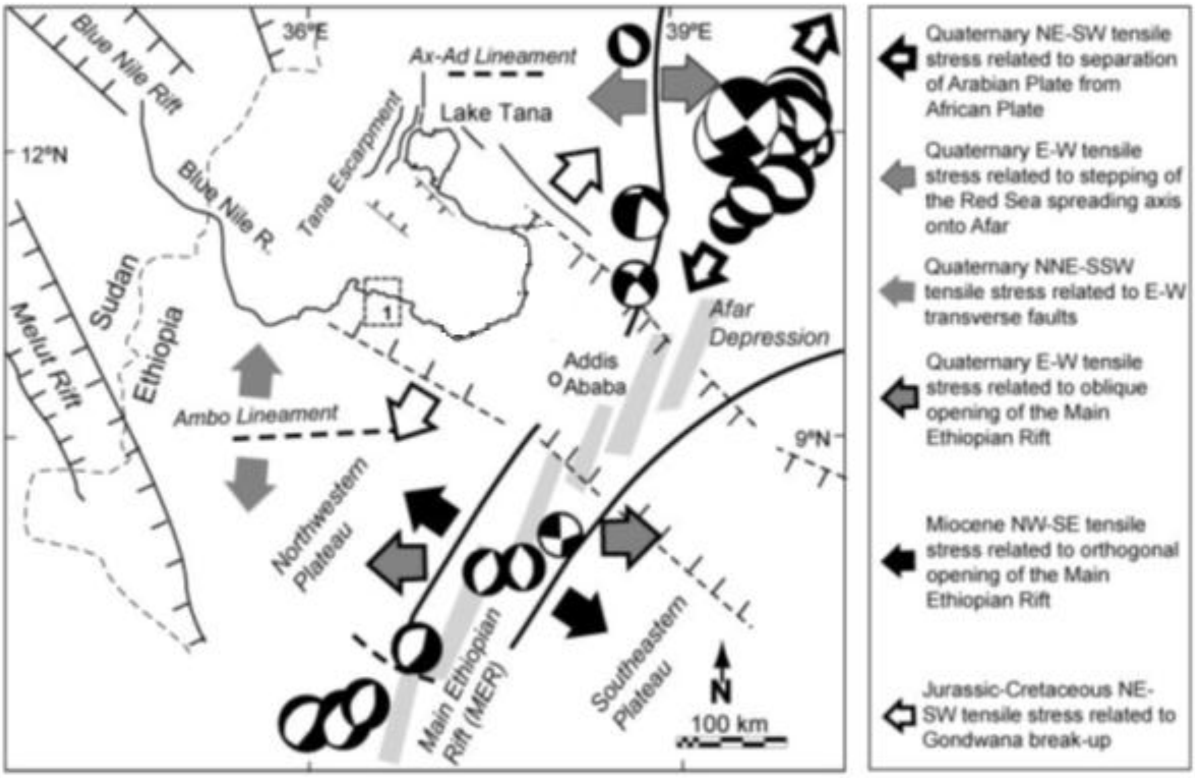
The Blue Nile Basin is bounded to the E and SE by the tectonic escarpment of the uplifted western flank of the Main Ethiopian Rift and to the N and S by the Axum–Adigrat and Ambo lineaments, respectively (Gani et al., 2008). During the Triassic–Cretaceous time, northern and central Africa was affected by lithospheric extension associated with NE–SW extension (Fairhead 1988). This formed NW-trending Mesozoic rift basins including the Muglad, the Melut, the Blue Nile and the Anza rift basins (McHargue et al., 1992; Binks and Fairhead, 1992). Bosellini (1989, 1992) and Russo et al. (1994) interpreted these structures as NW-trending aulacogen-like rift basins extending northwestward from the NE-trending Karoo rift which was formed in Late Palaeozoic–Jurassic times during Gondwana break-up. These rift basins terminate sharply in the northwest against the NE-trending Central African Shear Zone, which is considered to be a major dextral strike-slip shear zone (McHargue et al. 1992; Binks and Fairhead, 1992). However, there might be some lithospheric extension to the north of the shear zone, especially in the vicinity of the Blue Nile Basin (Millegan, 1990; McHargue et al., 1992).

The southeastern continuation of the Mesozoic rift basins, especially in the highlands of Ethiopia, is poorly understood. There, these basins are covered by 500–2000m thick pile of Early–Late Oligocene volcanic rocks, and locally followed by ~300m thick sequence of Quaternary volcanic rocks. These volcanic rocks are associated with the Afar Mantle Plume and subsequent opening of the Afar Depression and the Main Ethiopian Rift (Hofmann et al. 1997; Abebe, T. et al., 2005).

The Blue Nile Basin in Ethiopia lies between 98N and 13850’N, and 34850’E and 39850’E where the Blue Nile is incised into the ~2500m high (average) Northwestern Ethiopian Plateau. The linear exposures in the Gorge of the Nile make it difficult to trace the trend of extensional structures related to the Blue Nile Basin. Nevertheless, Mesozoic sedimentary sections and a few observed NW-trending faults have led some authors to suggest that the Blue Nile Basin is related to Mesozoic rift basins of eastern and central Africa (Bosellini, 1989, 1992). The presence of NW-trending sub-basins underneath Lake Tana has been taken as evidence to support this notion (Hautot et al., 2006).

Furthermore, it has been suggested that the Blue Nile Basin in Sudan continues south-eastward through Ethiopia, across the NE-trending Main Ethiopian Rift to join the Ogaden Basin in southeastern Ethiopia (Bosellini 1989; Russo et al. 1994).

The exposures of the Blue Nile Basin within the Northwestern Ethiopian Plateau are bordered by the uplifted tectonic escarpments on the western flanks of the Afar Depression and the Main Ethiopian Rift in the east and southeast, respectively, and in the west by the erosional Tana escarpment. The Quaternary-aged E-trending Axum–Adigrat and Ambo lineaments (Abebe, T. et al., 1998) bordered this region in the north and south, respectively, as cited in Gani et al., 2008



The rectangular box assigned number 1 is Blue Nile gorge of study area *(After; Gani et al., 2008)*

Fig 3.2 Regional tectonic stress regimes within and around the Blue Nile Basin

3.3 Local geology of the study area

Gohatsion–Dejen section of Abay Gorge is one of the areas of the Gorge where the complete geology of Ethiopia is exposed, starting from Precambrian basement to Mesozoic sediment over lain by Tertiary basal. Fig. 3.4 presents the geology of the study area. However, the basement rock is thin layer and practically not considered in this study. The sedimentation and post sedimentation phase of the Gorge which were the main concern of the study are discussed as follow in ascending order.

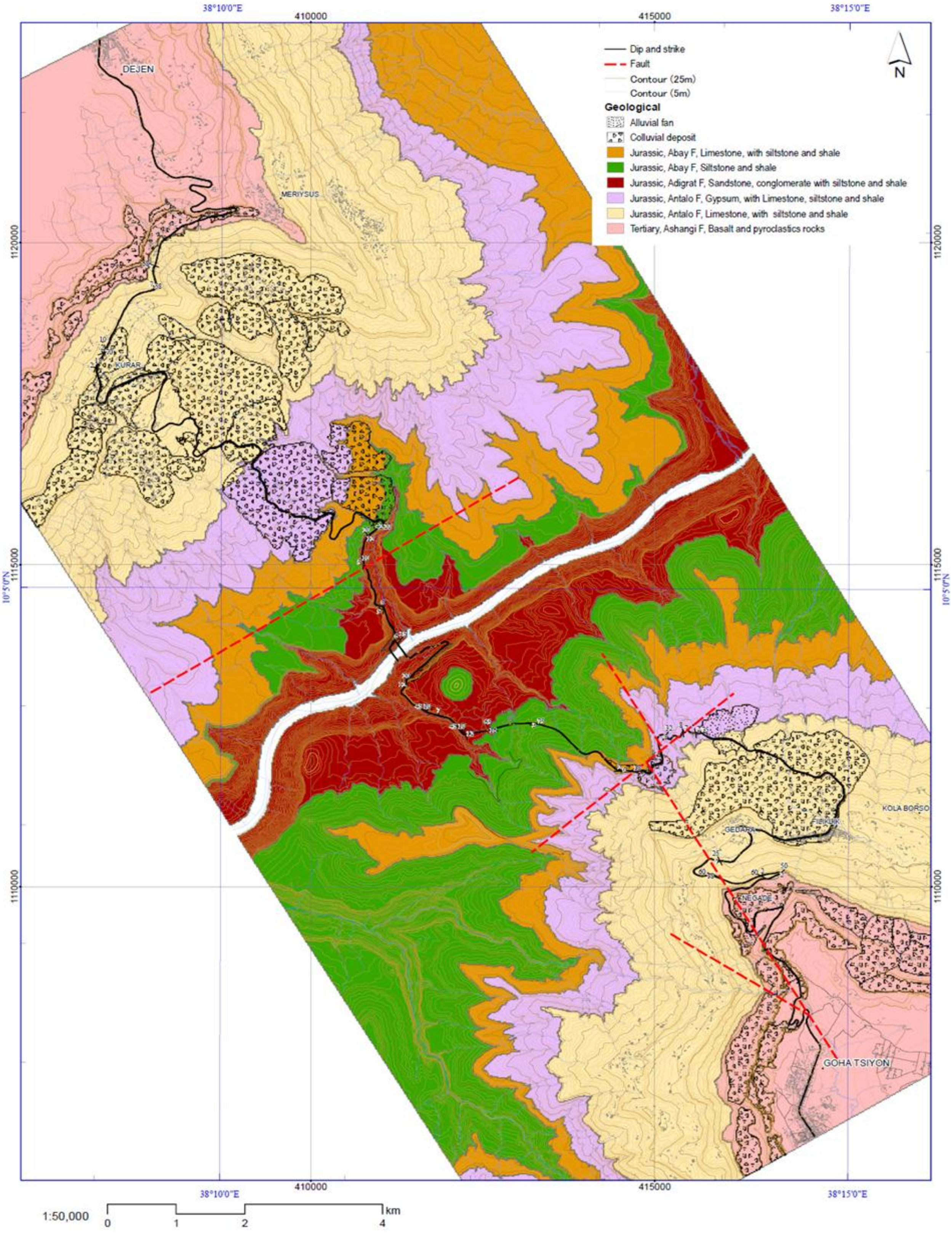
Lower Sandstone: The deposition of Lower sandstone in Blue Nile basin is partly the results of the Jurassic transgression related to the commencement of rifting between East and west Gondwana (Coffin and Rabinowtiz, 1988) and to the global rise in sea level (HAQ et al., 1987 as cited in Getaneh Assefa, 1991). The unit is found un-conformably overlying Neoproterozoic basement rocks and, in turn, is overlain by Early to mid-Jurassic siltstone and shale (sandy mudstone unit) which is found at the lower base of Lower limestone. The unit has a thickness of 280 m to 300 m and the grain size range from fine to coarse grained. The upper part of the unit range from fine to coarse grained sandstone with conglomerate horizons. The middle part on the other hand is formed from fine grained sandstone with mudstone intercalations. The lower part is made of fine to medium grained sandstone with basal conglomerate (Getaneh Assefa, 1991). The unit is mostly sandstone intercalate with layers of siltstone, mudstone, occasional beds of conglomerate and shale.

Structurally the unit has three well developed joint sets, the bedding plane which is horizontal and other two sets trending NW and NE. It forms a vertical cliff resulting rock fall due the intersection of the three joint sets (Almaz and Tadesse, 1994). On the middle part of this unit along the road cut the orientation of these joint sets were also resulting into plane failure that was one of the selected section for further analysis in the present study.

Fig 3.3 Lower sandstone unit

Silt stone Unit: It is typically thinly bedded, gray in color, and commonly contains cross laminations and convolute bedding. Dolomite is present in places as or as a secondary cementing medium is common in this unit. Siltstones can be massive or laminated, the individual laminae being picked out by mica and/or carbonaceous material. Micro-cross-bedding is frequently developed and the laminations may be convoluted in some siltstones.

Siltstones have high quartz content with predominantly siliceous cement. Frequently, silt stones are inter bedded with shales or fine-grained sandstones, siltstones occurs as thin ribs.



(Source: JICA, 2011)

Fig 3.4 Geologic map of the area

From strength point of view, grain interlocking is responsible for the principal increase in the angle of shearing resistance and increased with increasing density (Bell, 2000).

Clay stone Unit: Typically it is variegated, friable to semi indurated facies containing minerals such as; illite, kaolinite, calcite, dolomite, chert, quartz, chalcedony, and hematite. It exhibits various types of sedimentary structures such as thin bedding, and thick to thin laminations. Massive beds are very common. 'Clay ball' conglomerates within the clay stone layers consist of sub rounded to angular fragments of clay stone, 1-2cm in length (Getaneh Assefa, 1980).

From engineering point of view, the volume of clay soils is reduced as load is applied due to reduction of void ratio. If such soil is saturated, then load is initially carried by pore water which causes a pressure, the hydrostatic excess pressure to develop which finally dissipate and transfer the load to soil structure (Padfield and Sharrock, 1983 as cited in Bell, 2000).

Lower limestone unit: This unit is also called Abay Limestone and categorized into Middle Jurassicin age (Gani and Abdelsalam, 2006). It has a thickness ranging from 400 m to 450 m in the type area measured with alternating layers of Limestone and gypsum where limestone increases upward, bedded limestone at base sometimes bioturbated is available. Thinly bedded siltstone along with shale is also recognized in this unit. Deposition of the Lower Limestone indicates deepening of the basin. However, the alternation of gypsum and limestone in the upper part of the unit indicates repetitive drying and flooding of an evaporitic basin.

Where it is exposed in the type area, it is underlain by the glauconitic sandy mudstone unit or the Triassic–Early Jurassic Sandstone and overlain by a Middle–Late Jurassic Upper Limestone unit. The unit consists of a lower thinly bedded (average 20 cm) limestone interval and an upper interval of alternating limestone and gypsum beds (Gani and Abdelsalam, 2006). The bedded limestone, grey in color, is sparsely fossiliferous with burrows including Thalassinoides, Planolites and Ophiomorpha (Gani and Abdelsalam, 2006).

Gypsum unit: This unit having a thickness of ~ 253 m consists mainly of gypsum, shale, Limestone and dolostone that are locally inter bedded with clay stone, siltstone, and sandstone. The gypsum is gray, dark gray, bluish gray, and black, occurring either as independent beds, several meters thick , with lenses of dolostone, shale, and claystone layers. Both Nodular and lamina structures are common in this bed. The gypsum beds are characterized by mottled texture, and are inter-bedded with glauconitic mudstone beds and rare thin sandstone beds, and thin limestone bed as discussed above (Getaneh Assefa, 1980).

The limestone is gray, yellowish gray or brownish gray, occasionally containing skeletal fragments of foraminifers, echinoids and mollusks, brachiopods and calcareous algae. The dolostone is gray or dark gray with a content of dolomite ranging from 54 to 94% and averaging 84% (Getaneh Assefa, 1980).

Fig 3.5 Alternating shale, dolomite and gypsum layer within gypsum unit

The shale is greenish gray, yellow, green or dark gray, often calcareous or dolomitic, and contains a few thin laminae and veinlets of gypsum. Where it is well exposed, especially in the Gohatsion section of the Gorge this unit is found intercalated with variegated shale of differing thickness between gypsum beds among other associated rocks. On the top part of this unit fragmented gypsiferous shell with fingerprint like structure after the soft, shell part has been eroded away, is available. This unit is experienced a differing degree of weathering which caused its color to change from gray to dark gray or even black at few places.

Three well developed joint sets were noticed which contribute to slope failure. The inter-sections of these discontinuity sets (on horizontal bedding and the other two joint sets) made the unit susceptible to wedge failure as noticed on the main road cut from Gohatsion to Dejen. Other modes of failures like rock fall and plane failures are also possible.

From instability point of view, gypsum is more readily soluble than limestone. Caverns and sink holes are therefore more likely to develop rapidly in this rock unit than limestone (Eck and Redfield, 1965 as cited in Bell, 2000). This means it is also susceptible to weathering when exposed to differential surface moisture that makes it susceptible to failure through continuous degradation of its strength which in turn reduces its unconfined compressive strength mostly in case when it is available with other weak material like shale. In the present study one slope section was selected from this unit (gypsum unit) which exhibits the above mentioned typical properties.

Antalo Limestone unit: This unit is also informally called Upper Limestone. The bottom part of this unit is characterized by fossiliferous and burrowed mud-stone and oolitic limestone with corals, stromatoporoids, bivalves, gastropods, echinoids, fora-minifers and ostracods. Silty limestone with very thin marl layers follow. The upper part is formed by 5 m thick massive limestone. Total thickness reaches about 180 m. The middle and the upper part of the Antalo Limestone are characterized by marly limestone, silty limestone and hard limestone at the top and a planar laminated oolitic and reef limestone, followed by bedded mud-stones at the base. The maximum thickness of the oolitic unit is 170 m. This hard limestone unit was formed in shallow water, which is documented by the occurrence of oolitic bars, coral off shore patches and more protected inshore facies. Boundaries on both sides of this unit are transitional (Mogessie et al., 2002).

Limestone is perhaps more prone to pre and post depositional changes than any other rock type. After burial it can be modified to such an extent that their original characteristics are obscured or even obliterated. The most profound changes in texture and composition are those which lead to replacement of calcite by dolomite, silica, phosphate etc. Even many changes may takes place during transformation from soft and porous sediment to dense and less porous state as a result of disintegration change to soft sediments resulting in low unconfined compressive strength. Further, limestone is susceptible to solution which commonly removes shell fragments or even in rare cases ooids are removed. Sink holes may develop where opened joints intersect and these may lead to an integrated system of sub-terranean galleries and caverns. Fluctuation of water table with in the caverns increases the down ward seepage gradient and accelerates downward erosion contributing to instability of the area and hence affecting the stability of engineering structures (Bell, 2000).

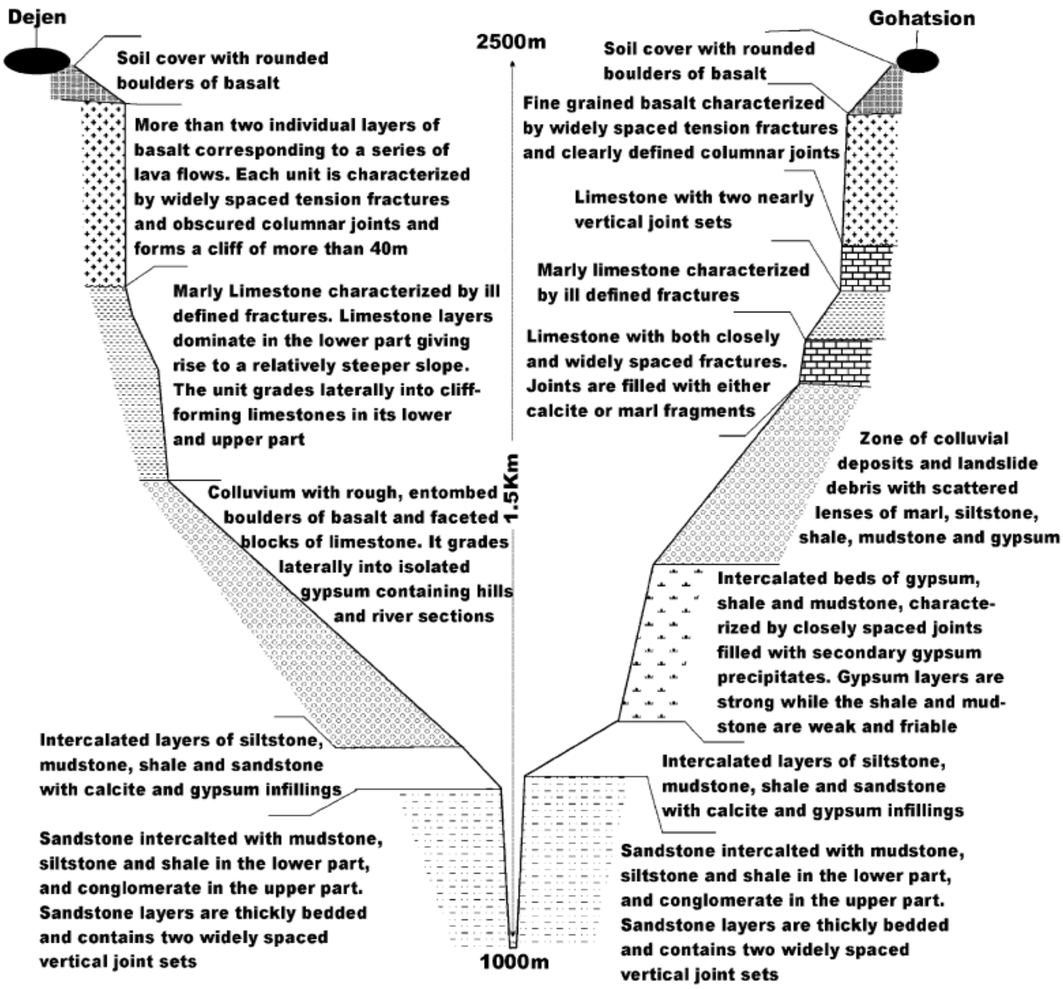
Tertiary Volcanic Rocks: This unit consists of thick basalt with thin layers of pyroclastic material and it is exposed on both sides of the gorge entrance.The exposed basalt is black with associated blocky/ massive and columnar structures. The pyroclastic material deposits of relatively less density is brownish in color with moderate degree of weathering where it is exposed on the road side that extend from Gohatsion to Dejen. Further, it is characterized by widely spaced vertical tension crack.

Locally, 1–3 cm thick sub-horizontal layering exists within the basalts which are generally aphanitic, and locally vesicular, with the vesicles sometimes filled with zeolites, calcite and quartz to form amygdaloidal texture. In a few places, the upper part of the basalts contains

~1m thick horizons of dark brown clay topped by a fine - to coarse-grained pyroclastic layer (Gani, et al., 2008).

The top part of this unit is a mixture of soil and boulder sized basalt with varying degree of weathering. The extent and areal coverage beyond the flank of the Gorge was not recognized but stream erosion of top soil up to nearly Gohatsion town revealed as there is basalt. However, map produced by different authors (Gani et al, 2006, 2008, Henok Woldegiorgis, 2008, Shiferaw Ayele, 2009) indicated as this unit covers the entire Gohatsion town and even by passes.

Fig 3.6 Pyroclastic deposit on basaltic unit (Gohatsion side)



(After; Lulseged Ayalew and Yamagishi, H., 2003)

Fig 3.7 A geological cross-section along the north–south section of the Abay River gorge

3.4 Structures in the Study Area

The pre-sedimentation, sedimentation and post-sedimentation phases of Blue Nile Gorge were affected by variable geologic discontinuities that might be associated with tectonic history of the area. Major geologic structures like faults were not mapped and interpreted from satellite images in the present study. However, joint set discontinuities which control rock slope stability in the area were measured and clearly identified. Previous studies carried out in the area gave better understanding of the effects of structures on each unit. As per Gani et al., (2008), the basement rocks (Neoproterozoic in age) are affected by normal faults with throws ranging between 5 cm to 5m, the orientation of these faults varies considerably. However, NNE- and ESE-trending normal faults are more common than NE- and NW-trending faults. In contrast, fractures within the Neoproterozoic basement rocks are dominantly NNE and ESE-trending. These fractures are clearly dilational with openings ranging between 10 cm to 50 cm, sometimes filled with tectonic breccias. Despite the above stated fault occurrence in the area, no other major fault was recognized during the present field work.

The basement rock is un-conformably overlain by lower sandstone unit in the study area. The Lower Sandstone unit is affected by dominant NW-trending normal faults and less dominant N-trending normal faults, as well as NW- and ENE-trending fractures, which are mostly dilational. Throws on the normal faults ranges between 50 cm to 8m, and fault zones range in width between 10 cm to 10 m. In some places, the normal faults are characterized by the smearing of mud layers and the presence of multiple internal fault surfaces (Gani et al., 2008). Further, this unit has three discontinuity sets on the road side of studied section having mean orientations of N310^{o}/34^{o}, N 245^{o} /68^{o} and N 122^{o} /60^{o}. Besides, there are subordinate structures including lateral accretion surfaces, horizontal stratification and ripple cross-lamination. In some places, silicified tree trunks up to 4m long, mud-cracks and vertebrate tracks are found within this unit (Gani et al., 2008).

There were no measured discontinuity orientations or recognized fault in Lower Limestone as during the present study no critical slope section was selected in this unit. However, Gani et al., (2008), discussed that this unit is cross-cut by NW-trending normal faults, NE- and NW-trending dilational fractures and less-frequent NE-trending normal faults. Similarly, gypsum unit is also affected by this type of structure. The fault planes exhibit both planar and listric geometry with throws ranging between 10 cm to 2m and fault zones ranging in width from 15 cm to 1m. Tilting roll-over anticlines and complex splay structures are common. In places, listric faults, which occasionally flatten out to become layer-parallel structures, result in the tilting of bedding planes to almost vertical.

Further, Gani et al., (2008) discussed that the Upper Limestone is affected by NW- and NE-trending normal faults, the throws of which generally range between a few cm to 60m, but with one fault having a 400 m throw. Fault zones range from a few cm to 50m in width. Fractures within this unit are dilational and dominantly N-trending with subordinate ENE- and NW-trending sets.

Normal faults in the Early–Late Oligocene basalts are dominantly N- to NE-trending and less often NW-trending. These faults have throws ranging from a few cm to 50 m, and rarely ~ 400m, with fault zones ranging between a few cm to ~50m wide. The dominant fractures are dilational and are NNE- and E-trending with subordinate NW-trending set (Gani et al., 2008).

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Chapter 4

Methodology

4.1 Basics of Deterministic and Probabilistic Approaches

In geotechnical field, slope stability analysis is very important for the safe functional design of rock and soil slopes considering different critical parameters of working stability. Since earlier time, most of the analyses were based on the conventional deterministic methods which do not consider the effect of uncertainty associated with certain input parameters for analysis. However, nature is so heterogeneous that unlike fabricated material in which we use proportional input parameter for the required output, uncertainties are unavoidable both from input parameter as well as from method of analysis point of view. Such uncertainties may cause variation in probability of failure of slopes even for those having the same factor of safety (Banakia et al., 2013).

For evaluating stability of the slopes against failure, there are two types of analyses: deterministic and probabilistic analyses. Deterministic analyses are traditional approaches like limit equilibrium methods (LEM) or finite element strength reduction methods (FE SRM). To check the stability of the slopes in the LEM, a slip surface is assumed. Soil mass above this slip surface is divided into slices and equilibrium equations are written for all slices. Factor of safety is calculated by one of the limit equilibrium methods like Spencer (Spencer, 1967), Bishop's simplified method of slices (Bishop, 1955), Janbu's method (Janbu, 1973 etc. as cited in Banakia et al (2013).

To determine the Factor of Safety (FOS) by the method of slices, a circular slip surface with radius R is assumed. The soil mass above the arc is divided into a number of vertical slices of width '*b'* and varying height '*h'*. The base of each slice is assumed to be a straight line inclined at an angle 'α' to the horizontal and with a length '*l'*.

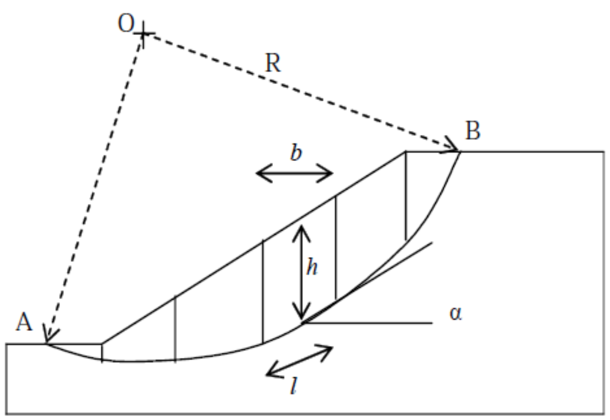


Fig. 4.1 Method of slices

The slope is divided into slices for analysis purposes only. It is assumed that all slices rotate around the Centre of the circle ‘O’ as a whole body. This implies that forces must act between the slices, termed inter slice forces (Abramson et al., 2002).

Probabilistic analyses on the other hand have been applied in slope stability more than any other geotechnical problem most recently by taking into account uncertainties of input parameters for analysis. Hence, Probability of failure (Pf) and reliability index (β) which are outputs of probabilistic analyses are calculated by one of the two procedures: simulation-based approach (e.g., Monte Carlo simulation; sub-set simulation) or analytical-based approaches (e.g., first-order second-moment; first-order reliability method). Both analytical and simulation based approaches require deterministic analysis that evaluate factor of safety. Therefore, alternative grouping for probabilistic analyses is based on their deterministic part that can be LEM-based probabilistic analyses or FE SRM-based probabilistic analyses. In the present study taking in to consideration the various limitations, LEM-based probabilistic analyses have been carried out for the accomplishments of the research objective.

4.2 General Methodological Procedure

The widely applied principle *"the past and present are keys to the future"*, which implies that landslides in future will be more likely to occur under those conditions which has led to the past and present mass movements (Varnes, 1984; Carrara et al., 1991; Carrara et al., 1995). The same principal is encompassed in the present study.

In the present study input parameters were conditioned based on the availability of data for selected slope sections. Accordingly, geological, hydrogeological, meteorological and seismic data was utilized for better characterization of the slopes. For the present study, based on field manifestation of actual or potential instability, a total of four slopes were selected, two from failed colluvium section and two from rock slope for, further analysis and study.

For successful completion of the above mentioned objectives the following methods and procedures were followed;

Previous works related to present study were reviewed to get better understandings of models used (Individual or comparative deterministic and probabilistic models). Besides, previous works especially works related to geology and geomorphology of the area were reviewed to understand the manner in which slopes have performed. Such understanding on geological and geomorphic processes and their possible effect on slope stability has helped in generating reliable results with stability models.

After literature was reviewed a clear understanding on the study area as well as on various stability methods was developed. Later, field work was conducted to collect all relevant data pertaining to slope stability studies. Before starting any data collection in the field a reconnaissance field survey was made to identify the critical slope sections. During this reconnaissance, efforts were made to observe the manifestations of slope, actual and potential, instability with particular emphasis to those slopes which fall along the road side. Thus, a total of four critical slope sections were identified and were selected for further study. Three of the slopes selected were located towards the Gohatsion side of the gorge and one towards the Dejen side of the gorge. One of the three slopes located towards the Gohatsion side of the gorge is composed of colluvium material which comprises mostly igneous rock fragments of varied dimension in sandy silty soil matrix. The other two slopes are rocky slopes within gypsum and lower sandstone units, respectively. The slope located towards the Dejen side of the gorge is characterized by colluvium material. This slide is identified as a single big slide with progressive secondary failures within the main slide.

During the present study, relevant data pertaining to various aspects related to geology, geomorphology, hydrogeology, climate etc. from both primary and secondary sources was collected or procured. From the selected critical rock slopes, structural data was collected for Kinematic analysis and for later Kinetic analysis. Those rock slopes which satisfied the kinematic conditions for respective mode of failure further stability analysis was carried out by SLIDE and Rock PLANE software. For critical colluvium slope sections, general geometry of the slopes and their relation with surface hydrology was established from field measurement and from secondary data pertaining to depth to ground water table. Further, critical failure surface was obtained by the grid search method of SLIDE software. Another, important data that has a, direct or indirect, bearing on slope stability is data related to Hydrogeology and Meteorology. The data on these aspects were collected from the secondary sources. Also, seismic data was directly adopted from published paper and seismic intensity zone in which the study area falls has been deduced. Accordingly, appropriate seismic coefficient was deduced for dynamic slope stability analysis.

In the present study, deterministic Factor of safety of critical slopes was computed which is used to indicate stability condition in quantitative terms based on limit equilibrium analysis.

For those rock slopes which have satisfied the kinematic condition Factor of safety (FS) was calculated as a general slope mass and by slicing the slope mass in to block level, separately.

For both rock and colluvium slopes the deterministic input parameters are considered to be with fixed values which are generally obtained from the standard tables well supported by the expert judgment. As a matter of fact these kinds of input parameters like; material strength, joint geometry and pore water pressures in FS calculation are not precisely known. This is because of uncertainties associated with; material homogeneity, modeling methods or the personal experience with which these material properties were established and used as input for analysis. This is the reason why some slopes fail even though their factor of safety is greater than one (FS>1). Thus, representation of stability condition of a slope only by a single value does not precisely indicate complete safety condition.

Another approach that is compatible with deterministic approach and uses all its input parameters and Factor of safety by treating them as random variables to reduce uncertainty and ensure reliability is Probabilistic approach. Probabilistic computation i.e. probabilistic analysis provides a facility to utilize a range of each geo-mechanical parameter that indicates the uncertainties involved in the rock mass (Banakia et al., 2013).

To perform probabilistic analysis, probability distribution for the factor of safety is assumed. Later, the selected factor of safety is quantified under Monte Carlo Simulation (MCS) by using a selected deterministic model (limit equilibrium in the present study). This is done by generating pseudo random numbers of input parameters for a large number of times (10000 for the present study) and changing the geotechnical input parameters over wide range, which is determined by the statistical parameters (mean, standard deviation, relative minimum and relative maximum). Pseudo random numbers are a sequence of numbers that look like random numbers by deterministic calculations. This is a suitable method for a computer to generate a large number of reproducible random numbers and is used commonly today (Rubinstein, 1981).

Monte Carlo method is simply a simulation-based method for calculating probability of failure and reliability index. Regardless of the problem that MCS is used for, first step is generating random numbers from a Probability Density Functions (PDF) like normal or log normal PDF. Each of these random numbers is called a realization or a Monte Carlo seed (Banakia et al., 2013).

In the present study the shape of probability density function was considered as normal distribution. To perform analysis by Monte Carlo Simulation under SLIDE software, critical slip surface was identified through trial and error method. Thus, minimum factor of safety was obtained through realization of soil strength parameters as their mean value was determined.

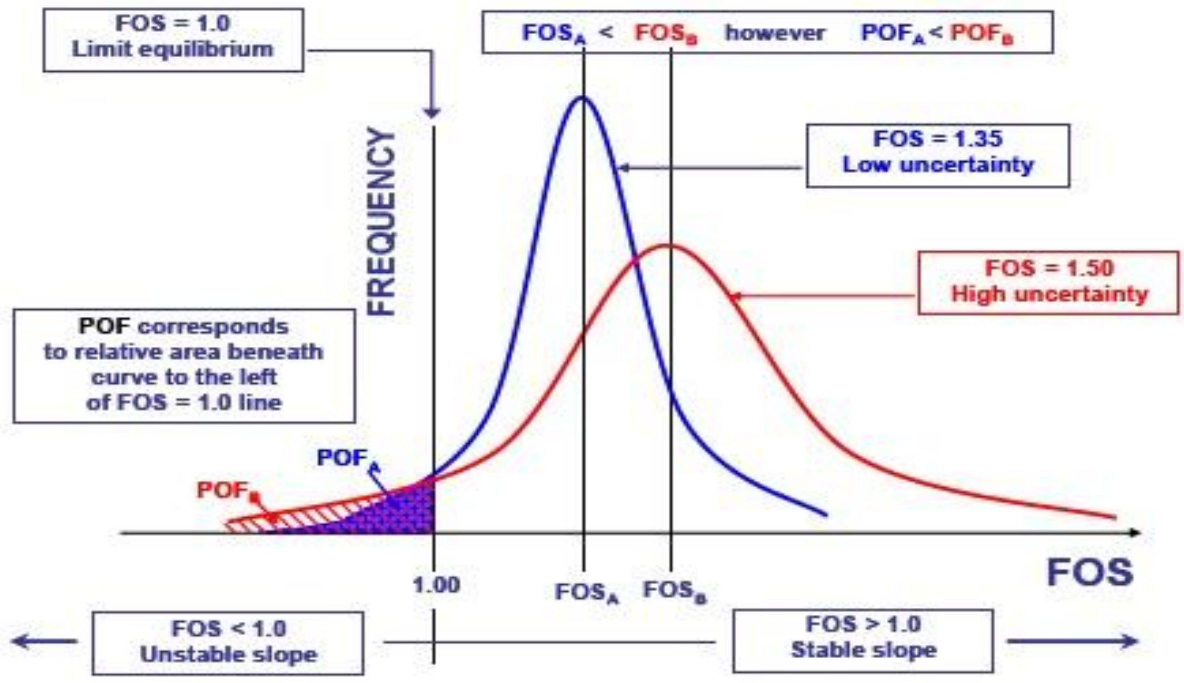
Monte Carlo simulation is performed through step wise process as per Dai et al., 1993 as follow;

i) Random numbers were generated which were independent random variables uniformly over the unit interval between 0 and 1.
ii) Random numbers were transformed from a uniform distribution to the distribution applicable to the component variable.
iii) Calculate values of all component variables based on the appropriate random numbers.
iv) Design variable was computed (i.e. factor of safety) using the generated values of the component variables.
v) Step (i) to (iv) was repeated a large number of times. The numbers of times these steps are repeated depend upon the variability of the input and output parameters and the desired accuracy of the output i.e. the large number of iterations the more accurate the output will be.
vi) Cumulative distribution was created of the design function using the data obtained from the above simulations.

As discussed above, from Monte Carlo simulation results of factor of safety, probability of failure and reliability of failure was calculated under SLIDE probabilistic sets. Reliability analysis focuses on the most important aspect of performance, namely the probability of failure (“failure” is a generic term for non-performance). The probability of failure is a more consistent and complete measure of safety because it is invariant to all mechanically equivalent definitions of safety and it incorporates additional uncertainty information (Phoon, 2008). The relationship between POF and FOS was established graphically (Fig. 4.2) (Tappia et al, 2007 as cited in Chiwaye, 2010).

Sensitivity analysis of input parameters was conducted primarily to identify the factor which contributes the most to slope instability i.e. to deduce that which input parameters may be critical for the assessment of slope stability, and which input parameters are less important.

Hence, a Sensitivity Plot was used to determine the value of a parameter which corresponds to a specified Factor of Safety (e.g. Factor of Safety = 1). For one or more selected input parameters, the user specifies a Minimum and a Maximum value. Later, each parameter can be varied in uniform increments, between the Minimum and Maximum values, and the safety factor of the Global Minimum slip surface is calculated at each value.



(Source: Tappia et al, 2007)

Fig 4.2 Definition of POF and its relationship with FOS according to uncertainty magnitude

While a parameter is being varied, all other input parameters are held constant, at their MEAN values. This results in a plot of safety factor versus the input parameter(s), and allows you to determine the “sensitivity” of the safety factor, to changes in the input parameter(s).

A steeply changing curve on a Sensitivity Plot indicates that the safety factor is sensitive to the value of the parameter. A relatively “flat” curve indicates that the safety factor is not sensitive to the value of the parameter (SLIDE, 2010).

After sensitivity of safety factor to various input parameters was determined the next step was to propose remedial measures. It was proposed on the basis of existing slope stability condition as well as for the anticipated adverse conditions to which the slope will be subjected. Besides, proper preventive measures have also been suggested to reduce any possibilities for further failures. The remedial measures have been proposed separately for both colluvium and rock slopes.

The general methodology flow chart as followed during the present study is presented as Fig 4.3.

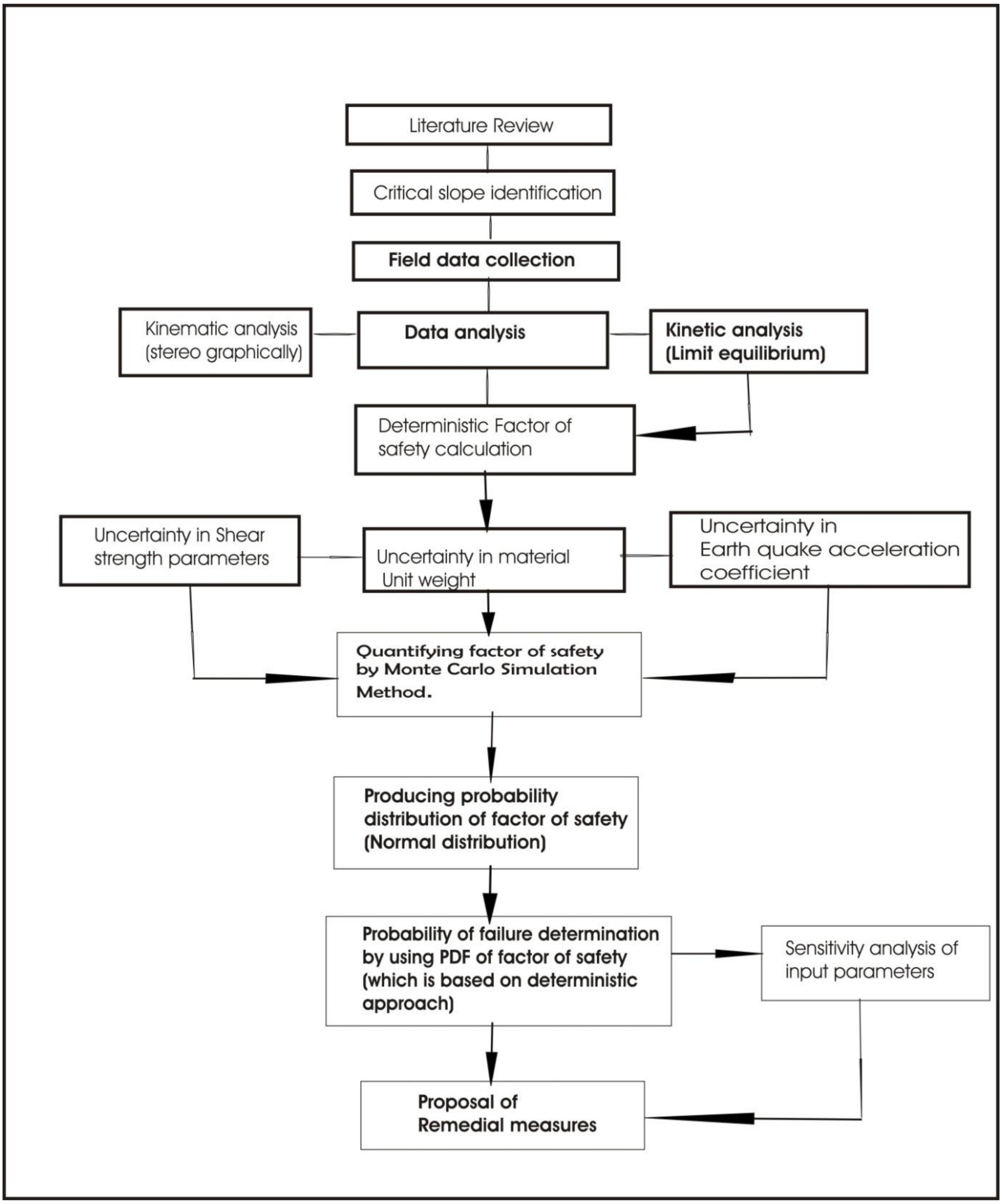


Fig 4.3 General methodological procedures for the study

4.3 Estimation/ Determination of input parameters

For rock slopes the rock mass was assumed to behave as a Coulomb material, its shear strength depends upon cohesion and angle of internal friction. For the present analysis, shear strength parameters were determined from standard table proposed by Hoek and Bray (1981), correlated with RMR data collected from the field on rock slope. On the basis of RMR values the rock mass can be classified in to five classes namely; very good (RMR 100-81), good (80-61), fair (60-41), poor (40-21) and very poor (<20). Further, for each of rock mass classes estimated range of cohesion and angle of internal friction are available. The Geometry of each of the critical slope sections was estimated/ measured either in the field or was deduced from the topographical maps. Unit weight of rock and colluviums soil mass was directly adopted from the standard tables. Horizontal acceleration and depth to ground water was adopted from secondary sources for stability analysis. A detailed description on all these input parameters used in stability analysis is presented in following Chapter.

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Chapter 5

Results and Discussion

5.1 Data Processing

During the present study in order to assess and characterize the slope stability condition of selected critical slope sections comparative approach was followed. A reconnaissance field trip was undertaken to identify the critical slope sections in the study area. For this preliminary investigations were undertaken and slopes in the area were examined for visible manifestations of actual or potential instability. Thus, based on the field manifestations four critical slope sections, two from failed colluvium section and two from rock slopes were identified for further analysis.

Further, relevant data pertaining to various aspects related to geology, geomorphology, hydrogeology, climate etc. from both primary and secondary sources was collected or procured. From selected rock slope sections systematic discontinuity data and data pertaining to Rock Mass Rating (RMR) was collected. Later, discontinuity data was analyzed to deduce the preferred orientations and RMR data was analyzed to characterize the rock mass present on the critical slope sections. To deduce the stability condition the two critical rock slope sections were subjected to Kinematic Check. Out of two only one critical rock slope section satisfies the Kinematic Check thus, it was considered for further stability analysis. Further, the two critical colluviums slope sections, which were already failed, were considered for further analysis and to define their general stability characterized under existing and anticipated adverse conditions. Thus, for stability analysis of these critical sections all required necessary data for stability analysis was either collected in the field or was procured from the secondary sources. In order to define the geometry and ground water condition within the critical slopes data related to scarp faces for the big slides, scarp faces with in failed masses, depth to ground water table, slope dimensions, distance from the scarp face to toe of the slope and expanse of soil mass between scarp faces were collected. All these data were later utilized for stability analysis.

For a slope instability assessment of individual slopes basic assumption was followed that conditions which have led for the slope failure in the past if repeated again may result in failure in future also. Besides, the conditions which were perhaps not existed in past and may have possible potential triggering effect for anticipated conditions in future were also considered in the stability analysis.

For analysis purpose another assumption which was made is that the in-situ tests/ observations which were considered has perhaps provided reliable, economic and feasibility material properties for realistic stability analysis.
Eventually the slope materials are described as such;

Basalt: The slope forming basalt in the study area is characterized by highly fractured, angular to sub angular blocks of cobble to boulder size and columnar jointed on its cliff portion covering most of the slope of the upper most part of Gohatsion side section. It is also associated with fine to course grained sandy silt soil. Water permeability of the layer is controlled by the columnar joints and wide open fractures and it is high.

Colluvium: It is characterized by a mixture of angular to sub-angular basalt, limestone and gypsum rock bocks of cobbles/boulders (5-600 cm), gravel and sandy silt soil matrixes which have come from the mountainous side cliff. The deposit overlies on the surface with 0-5 m thickness, and partly intercalated with around 20 m thickness under the artificial embankment. Water permeability of the deposit is considered to be very high (JICA, 2011).

Silt and shale: The units are mainly composed of siltstone and shale with minor limestone. The siltstone unit is composed of silt/clay particles cemented together. It is found intercalated with shale, mudstone and sandstone with calcite and/or gypsum veins in places. The color is white brown to reddish brown which have been weathered and yellowish green to brownish grey which are relatively fresh. The shale unit is argillaceous clastic sedimentary rock which contains a lot of clay minerals. A part of the shale is totally changed to clay by weathering, which is relatively soft and weak due to weathering.

Sandstone: It is horizontally bedded, moderately weathered and discontinuity sets orientation defines its mode of failure and control its stability. Further, minor block failures and potential instability was also noticed on the Lower sandstone unit. The sandstone is basically composed of quartz grain, but sometimes contains reddish fragments, including pinkish or slightly reddish color. Water permeability of the layer is considered to be relatively low.

Pyroclastic deposit: The unit is sandy tuff to tuffaceous sandstone with lenticular clayey mud layers. On the bottom side of the road particularly on embankment sides layers of unconsolidated ash deposit was observed. It is very soft and brittle, and is susceptible to weathering and erosion, resulting in a flat plane on top of the basalt. The color is mainly white to grey which is relatively fresh area and grey to black brown which is highly weathered.

5.2 Critical Failure Surface Determination

Both factor of safety and probability of failure determination in SLIDE software need a critical slip surface to be determined from each combination of slope parameters. This critical slip surface is a sliding surface with a minimum safety factor of material defined in Limit equilibrium analysis and has circular geometry owing to the nature of material under consideration. In their simplest form, limit equilibrium techniques examine static stability, where a simple balance of disturbing and resisting forces is used. For example; this is to search for the most likely depth of failure (i.e. most critical slip surface). In order to locate the critical slip surfaces, a slip center search grid of 20 x 20 intervals was used for a safety factor analysis of colluvium slopes from global minimum on SLIDE software. However, ROCPLANE was used to directly calculate safety factors from failure mass dimensions. The main focus in this analysis was along the road section. In general, such slip surface emphasized along the road side was determined and for the entire slide mass the slip surface was determined through query.

5.3 Detailed Slope Stability Analysis of Critical Colluvium Slopes by Combined Deterministic and Probabilistic Approaches

5.3.1 Deterministic Stability analysis of SL 1 Under static dry condition

Slope location 1 (SL 1) is located within basaltic section and it appears just at the entrance of the gorge from Gohatsion side. This critical slope is situated at the base of the basaltic cliff.

This slope section in general can be characterized by boulder sized colluvium material of basaltic source material intercalated with thin layer of pyroclastic deposit and a matrix of clay/ silt soil of low cohesion. Even at small scale the slope has complex geometry with alternate bulging and sinking morphology characterized by small to widely (5cm to 25cm) opened ground ruptures aligned nearly horizontal with respect to the top flat surface above the basaltic cliff. As observed during the field trip this slope showed clear manifestations of actual failure as evidenced from disposition of Gabion protection works provided along the road side. Besides, displacement was also observed on the Extensometer installed along this slope section. To obtain information about the slope performance, the length of the slope geometry was measured by GPS tracking system while minor surface irregularities were neglecting. Further, slope inclination was assessed/ estimated along road side for stability analysis purpose. Moreover, the sub-surface material condition was obtained from borehole log prepared to determine depth of ground water table (JICA, 2011). From this bore hole log slope cross section was prepared as presented in Fig 5.1. The prepared section was later utilized for stability analysis carried out by deterministic and probabilistic methods.

The input parameters and values shown in Table 5.1 was used in the stability analysis carried out for static dry, static saturated, dynamic dry and dynamic saturated conditions. Further in probabilistic analysis with inclusion of their uncertainties the values were used in the form of statistical parameters such as; mean standard deviation, relative minimum and relative maximum. Because of the nature of the colluvium material present in this slope section (SL1) sampling and laboratory analysis was not feasible therefore with careful observation and field correlation all input parameters were adopted directly from standard table proposed by Hoek and Bray (1981). To simulate the possible conditions under which the slope has possible failed in past the analysis was carried out under different anticipated conditions as listed above. The stability analysis was initiated with computation of deterministic safety factor the same was later used as an input parameter in the probabilistic stability analysis.

Table 5.1 Input values and deterministic Factor of safety results of SL (1) at static dry condition

Material type	C (kN/m^2)	Ø (deg.)	γ (kN/m^3)	SLIDE Factor of safety results				
				Janbu Simplified	Bishop	Janbu corrected	GLE	Spencer
Colluvium	0	34	17	0.99	1.013	1.04	1.01	1.025
Basalt	300	40	27					
Pyroclastic	0	32	10					

A Perusal of results reveal that the deterministic factor of safety of the slope near road section without earthquake loading (static analysis) and in dry state (ground water table was assumed at its lowest level) comes out to be 1.013 (Stable condition) and for the entire section it is 1.114 (Stable condition) as shown in the Fig. 5.2. The safety factor results displayed in Fig. 5.2 were calculated from 'Bishop Simplified' method. Further, factor of safety results as calculated by Janbu Simplified, Janbu Corrected, GLE and Spenser are presented in Table 5.1. Moreover, the critical slip surface depth as determine from this grid search was 16 m below the road section for safety factor 1.013 and 64m for safety factor value of 1.114, respectively.

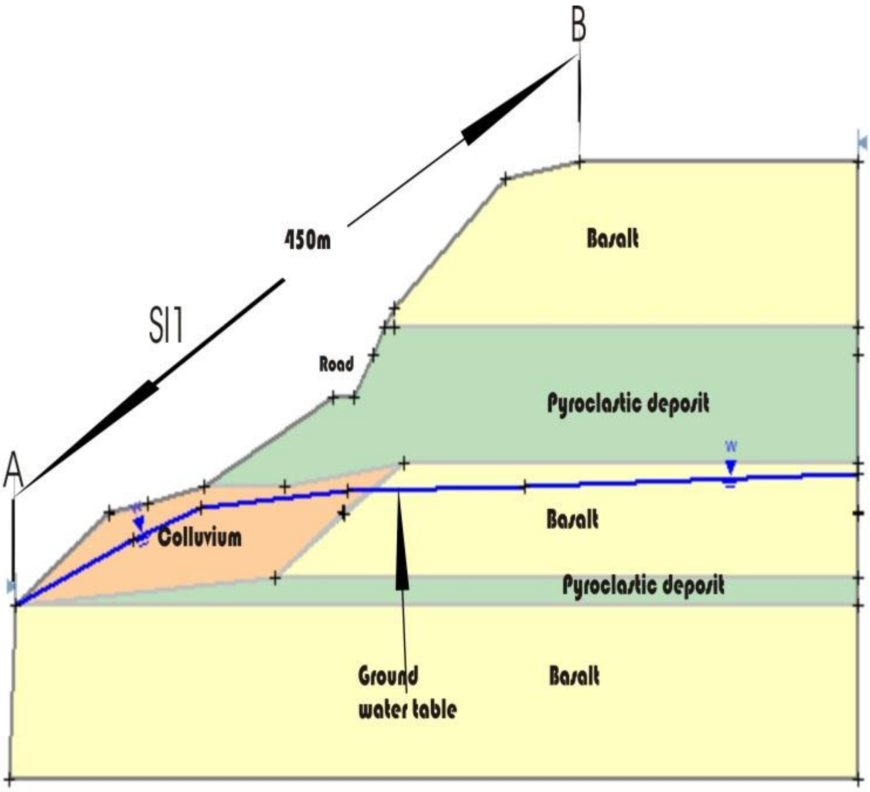


Fig 5.1a Cross section of SL1 slope section

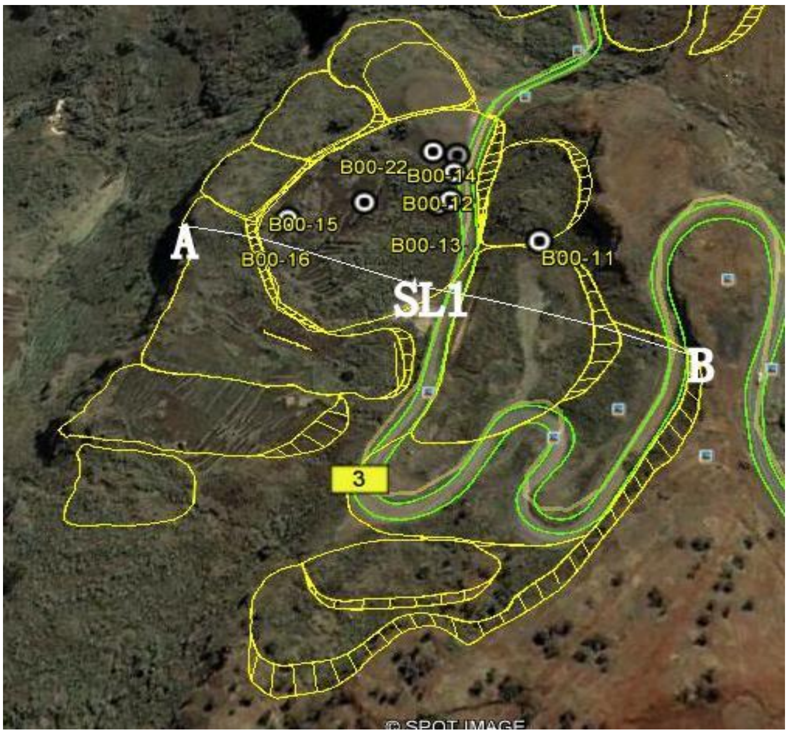


Fig 5.1b Geometry and extent of slope with surface manifestations
of instability and borehole locations

Fig 5.1 Geometry of the slope section (SL1)

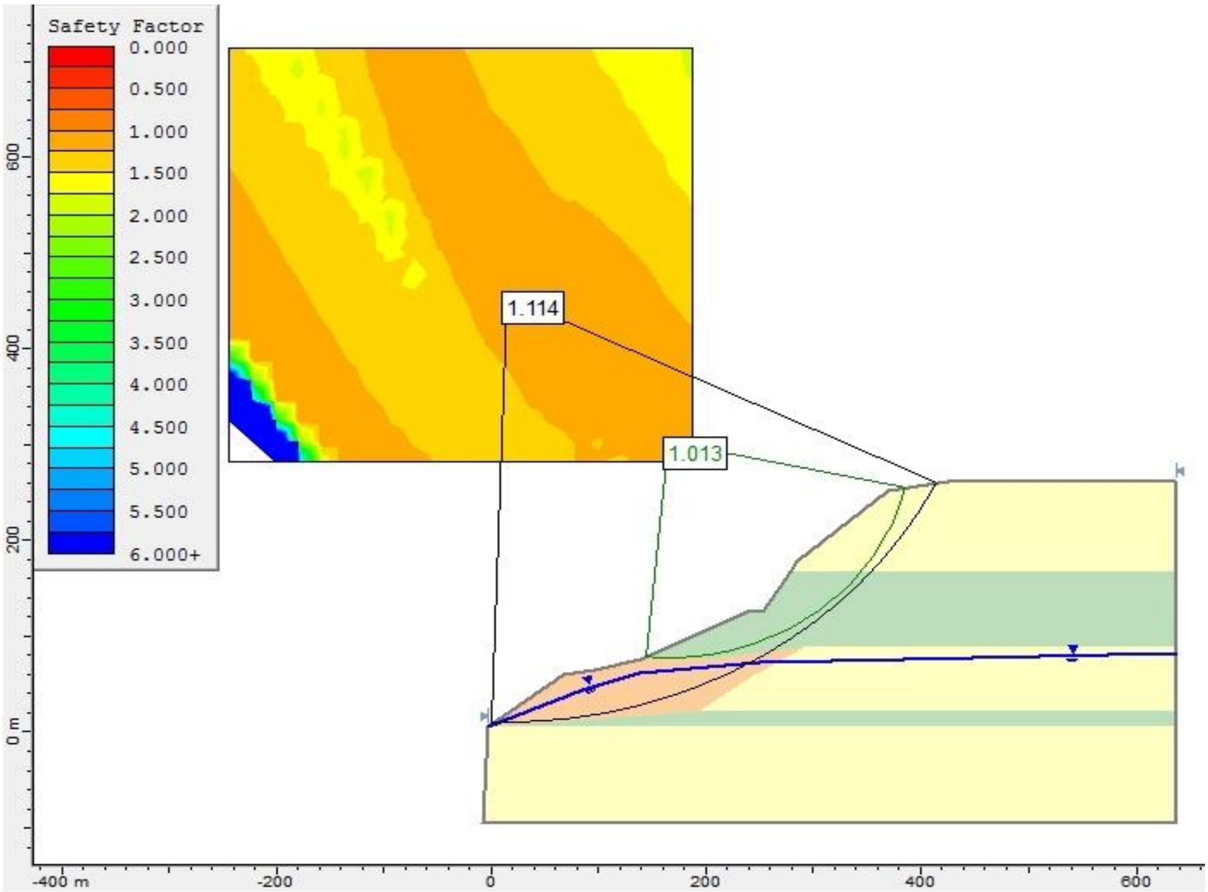


Fig 5.2 SLIDE factor of safety results of SL1 under static dry condition

5.3.2 Deterministic Stability Analysis of SL 1 Section Under Static Saturated Condition

For determination of safety factor under static saturated condition the possible worst case in which slope is fully saturated was considered which may represent the availability of water results in development of pore water pressure. Pore pressure has been defined as the portion of the normal stress that is being exerted within the soil pore space. Pore pressure can be exerted in two different forms; water pressure and tension force (Crozier, 1986 as cited in Tobing, 1997) dependent upon the saturation condition of the soil-profile. Under saturation condition, the pore pressure is exerted as water pressure; whereas, within unsaturated conditions, the tension forces make up the pore pressure. Both forces work in the opposite direction to the stability of the slope profile; water pressure reduces the slope strength, whereas tension forces tend to stabilize the slope profile (Tobing, 1997).

Hence, at static saturated condition the effect of water pressure was analyzed by assuming ground water table very close to the surface of the slope and to understand its negative influence from slope stability point of view when compared to static dry condition.

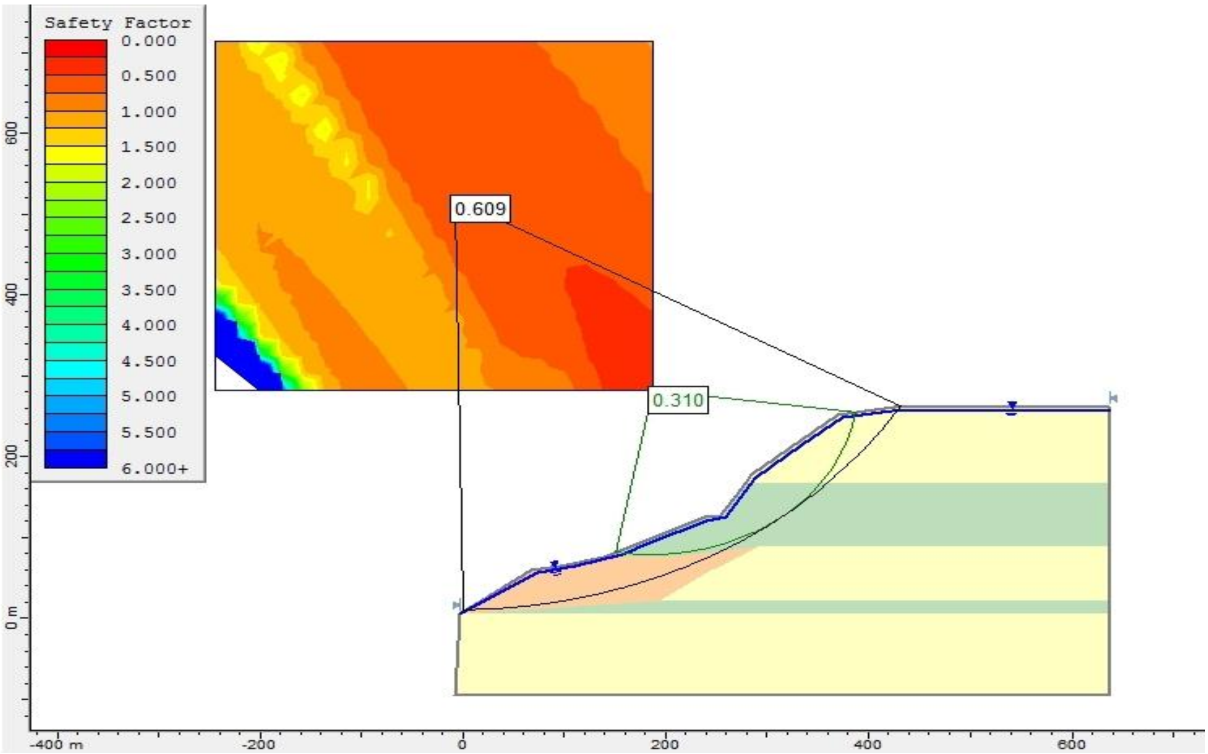


**Fig 5.3 SLIDE safety fac
tor results of SL 1 slope section under static saturated condition**

The deterministic safety factor results for SL1 slope section under static saturated condition for road section comes out to be 0.310 and for entire slope section it is 0.609. Thus, both the sections under static saturated conditions represent unstable conditions as Safety factor in both the cases is less than one.

5.3.3 Deterministic Stability Analysis of SL 1 Section Under Dynamic Dry Condition

With respect to geographical location of the study area it is very essential to consider the effect of earth quake on slope stability for anticipated adverse condition. When an earthquake occur the inertial forces from the slope's own weight serves as additional driving /resisting forces. Moreover, the strength parameters and the soil shear modulus are affected in addition to the generation of excess pore water pressure in certain types of soils (in permeable soils including colluvium); hence the response of a soil mass is totally different from the static response (Hunt, 2005).

The rigorous SLIDE stability software facilitates to include the dynamic parameter and the analysis can address the experience of slope under dynamic dry condition. Such dynamic condition can be defined as when the slope is dry (the ground water table is at its lowest level) and the slope is subjected to dynamic loading. The area fall within 7 MM intensity scale as per Seismic risk map of Ethiopia for 100 year return period as proposed by Laike Mariam Asfaw (1986). Further, the horizontal seismic coefficient used for analysis was 0.08g as determined from the MM intensity graph proposed by Johnson et al., 1988.

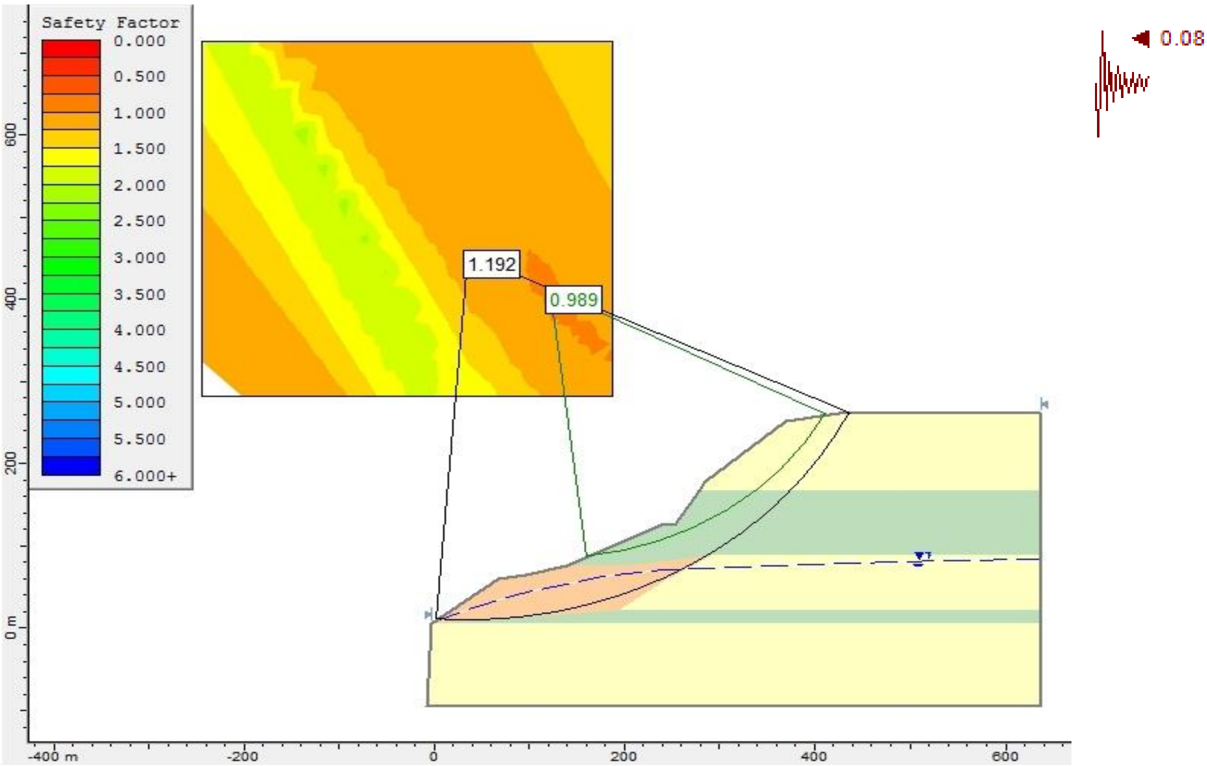


Fig. 5.4 SLIDE safety factor results of SL 1 Section under dynamic dry condition

The deterministic safety factor results for SL1 slope section under dynamic dry condition for road section comes out to be 0.989 and for entire slope section it is 1.192. Thus, the stability condition for road section under dynamic dry condition is unstable whereas, for entire section it is stable. Here, it is worth mentioning that factor of safety for natural slopes is considered safe when it is more than 1.0 however, for engineering applications such as; slopes along the road cuts the design safe factor of safety is taken as greater than 1.5 (Hoek and Bray, 1981).

5.3.4 Deterministic Stability Analysis of SL 1 Section Under Dynamic Saturated Condition

As already discussed the slope material of SL1 slope section is mainly comprises of pyroclastic, fragmented basalt bounders, colluvium and gravel to clay sized soil mixtures. When saturated, fine, especially loose sand is subjected to vibratory motions, such as those caused by an earthquake, the sand can liquefy, losing almost all its shear strength and possibly flowing like a dense fluid. Structures supported on such liquefying soil (like gabion mesh or protective walls on the road side) may fail catastrophically causing serious damage (Richter, 1958).

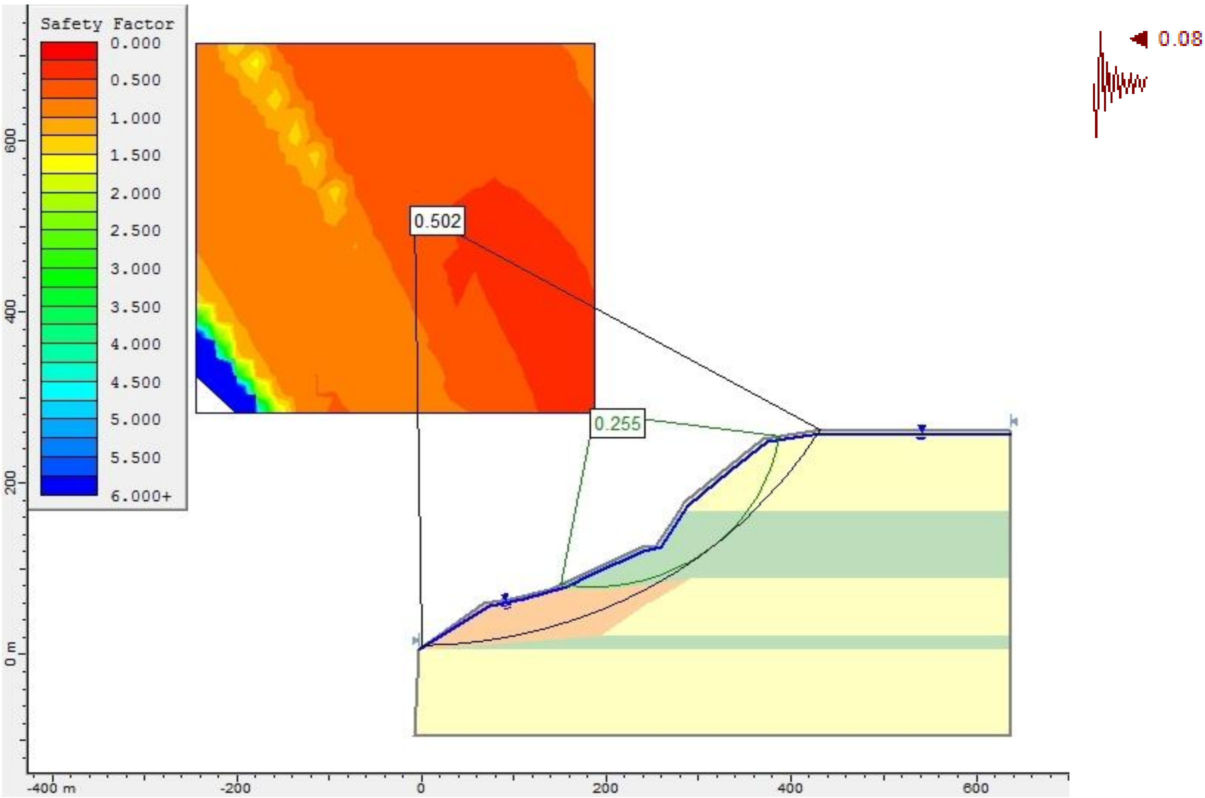


Fig 5.5 SLIDE safety factor results of SL 1 Section under dynamic saturated condition

The deterministic safety factor results for SL1 slope section under dynamic saturated condition for road section comes out to be 0.255 and for entire slope section it is 0.502. Thus, both the sections under dynamic saturated conditions represent unstable conditions as Safety factor in both the cases is less than one.

5.3.5 Probabilistic Stability analysis of SL1 Section under static dry condition

Probabilities and probabilistic methods are useful because of our ignorance of the true future frequency of events or values of different parameters, such as shear strength (Lee and Jones, 2004). Hence, input parameter used in deterministic analysis (cohesion, fiction angle and unit weights) was treated as random variables using normal (Gaussian) probability distribution. The input parameters can have infinite numbers of possible outcomes rather than deterministic which is binomial case where there are only two possible outcomes - failure or no failure. For the present analysis, the random values were generated by Monte Carlo method through repeated trials involving the random sampling of input parameters from an input probability distribution (normal distribution). Each analysis is essentially deterministic, but repeated simulation allows a probability distribution to be produced for the results (i.e. an output distribution).

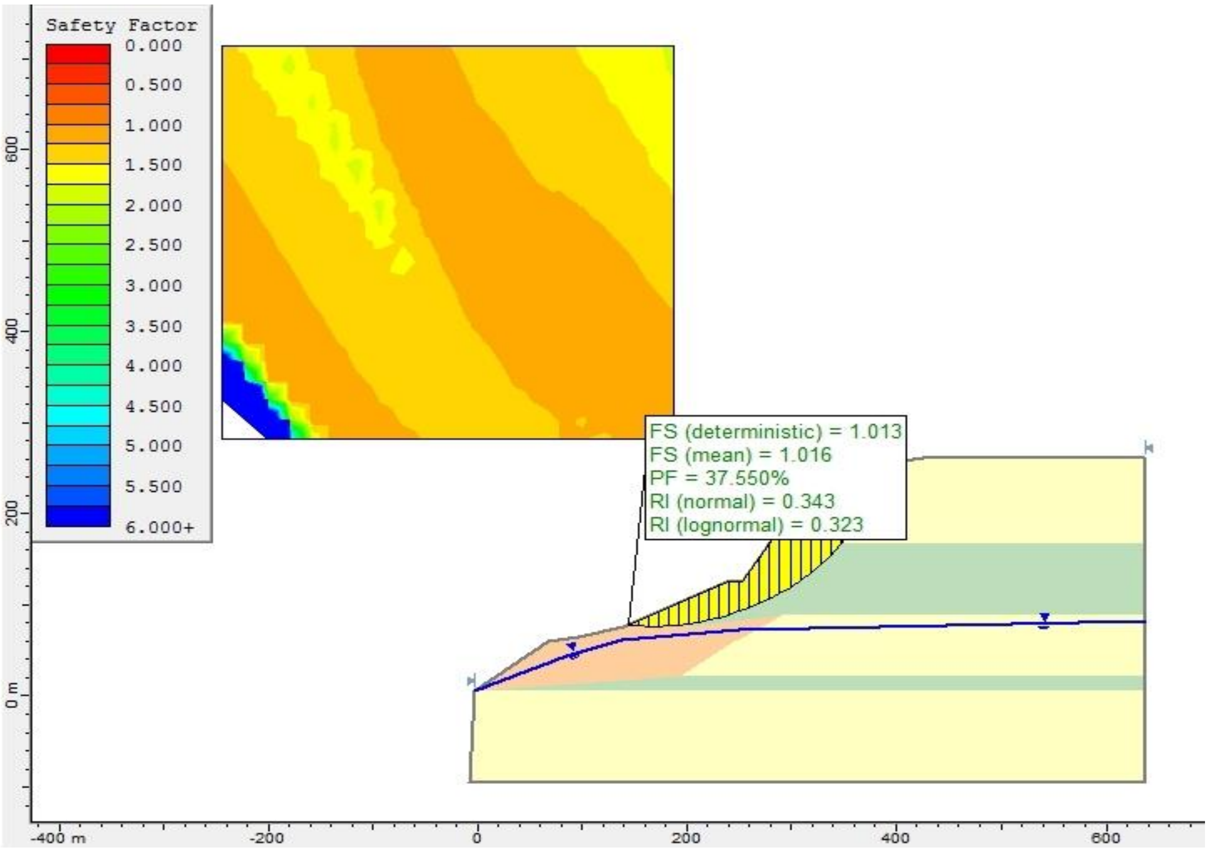


Fig 5.6 (a) SLIDE safety factor and probability of failure results of SL 1 section under static dry condition

For the present case (SL1 slope section under static dry condition) stability analysis was used to generate estimates of the probability of slope failure that is the probability that the Factor of Safety will be less than 1, based on many simulations using variable parameter values (cohesion, friction angle and material unit weight).

The probability result (Fig5.6 (a)) indicates probability of failure of the slope is 37.55% which is an indication of the beginning of failure. Further, reliability index of the slope were 0.34 and 0.32 for normal and log normal distributions, respectively. The reliability index provides a more meaningful measure of stability than the factor of safety *F*.

Geotechnical engineers have long recognized that *F* has little physical meaning and that the choice of a satisfactory value is fraught with difficulty.

In contrast, the reliability index describes safety by the number of standard deviations (i.e. the amount of uncertainty in the calculated value of *F)* separating the best estimate of *F* from its defined failure value of 1.0. It is a more explicit approach (Baecher and Christian, 2003).

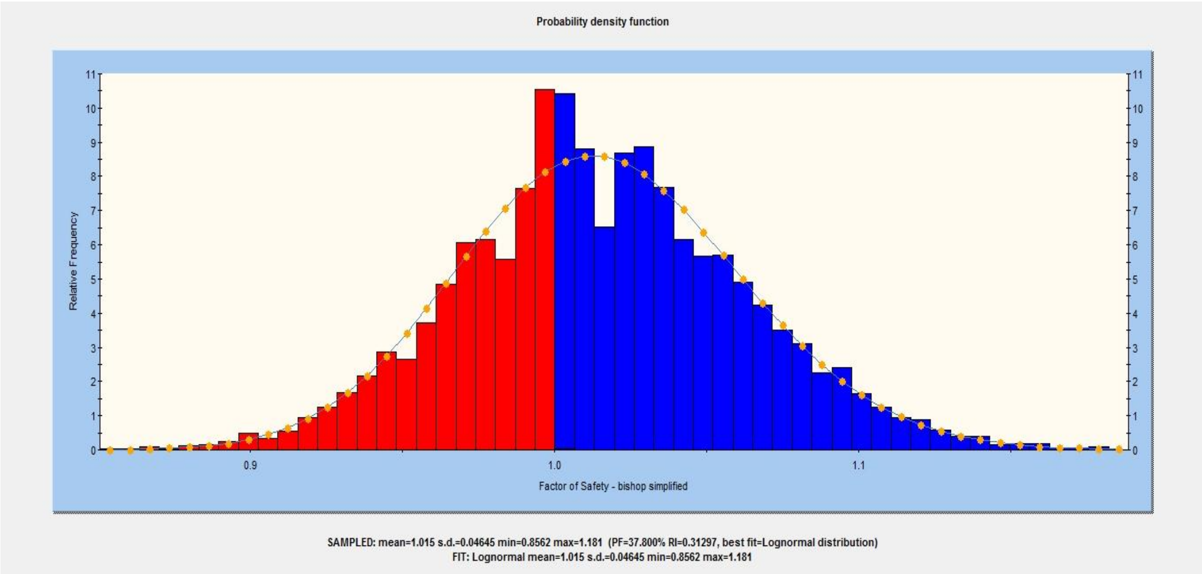


Fig 5.6 b) Probability distributions of safety factor results of SL 1 Section under static dry condition

5.3.6 Probabilistic Stability Analysis of SL1 Section Under Static Saturated Condition

The analysis of slope Section SL1 under static saturated condition assumed that the slope is nearly fully saturated. Further, all the input parameters and calculation procedures were followed in the similar manner as that was adopted for analysis under probabilistic static dry condition with the exception of adding saturated unit weight of material for the said case.

The analysis result for slope section SL 1 under static saturated condition, probable anticipated worst case, provided a probability of failure of 100% and minimum reliability indexes of -16.289 under normal distribution and -8.651 under log normal distributions. Further, the probability distribution of safety factor given in Fig. 5.7 (b) show the mean factor of safety of slope under consideration is 0.308 among a range of safety factor values the slope have.

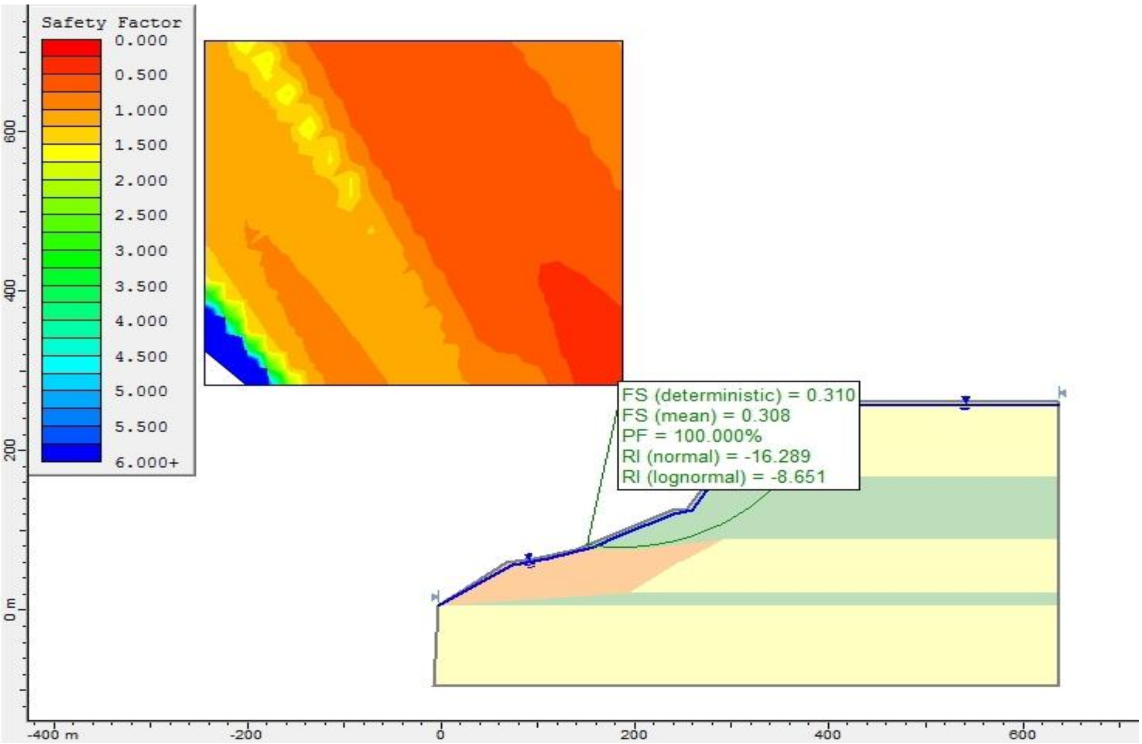


Fig 5.7 (a) SLIDE safety factor and probability of failure results of SL 1under static saturated condition

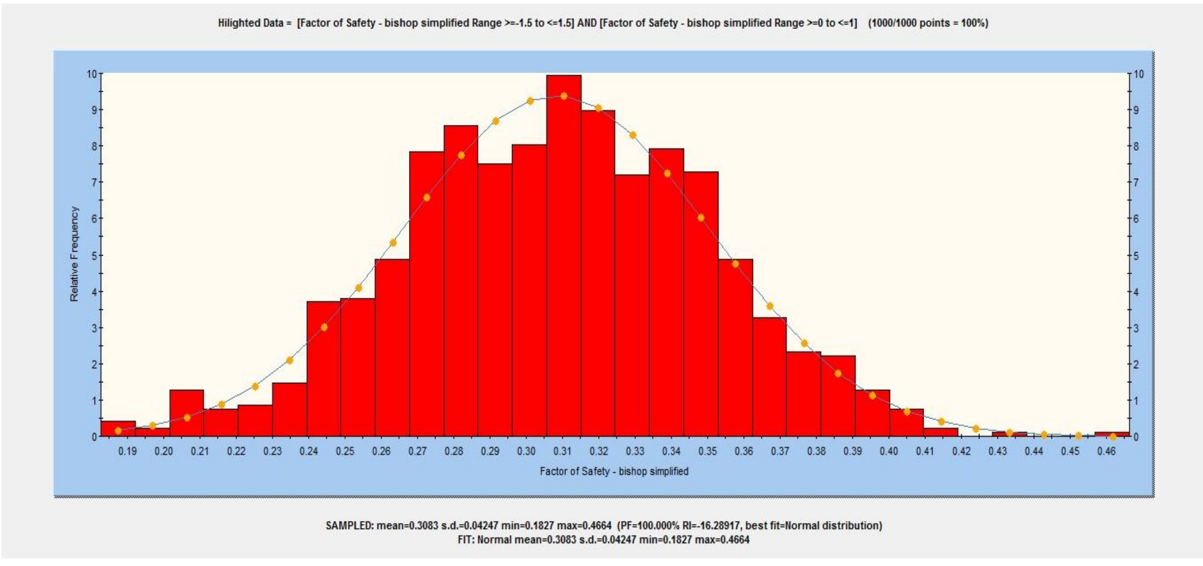


Fig 5.7 (b) probability distribution of safety factor results of SL 1under static saturated condition

5.3.7 Probabilistic Stability Analysis of SL1 Section Under Dynamic Dry Condition

The possible worst case considered for SL1 slope section was a case in which the slope is subjected to earth quake in a dry season and the slope response to such condition was analyzed by incorporating a horizontal seismic coefficient of 0.08g.

The probabilistic stability analysis for SL1 slope section under the case where earth quake can possible occur during dry condition has resulted (Fig. 5.8 (a)) in to a probability of failure of 64% with reliability indexes of -0358 and -0.373 under normal and lognormal condition, respectively.

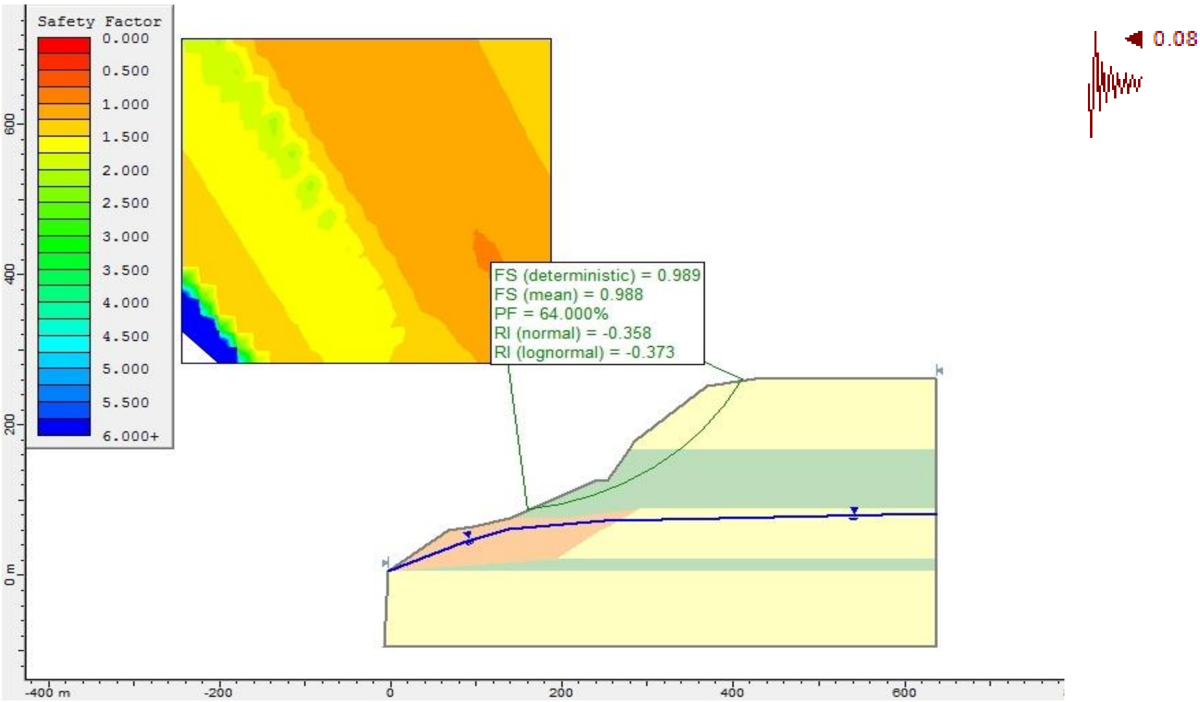


Fig 5.8 (a) SLIDE safety factor and probability of failure results of SL 1under dynamic dry condition

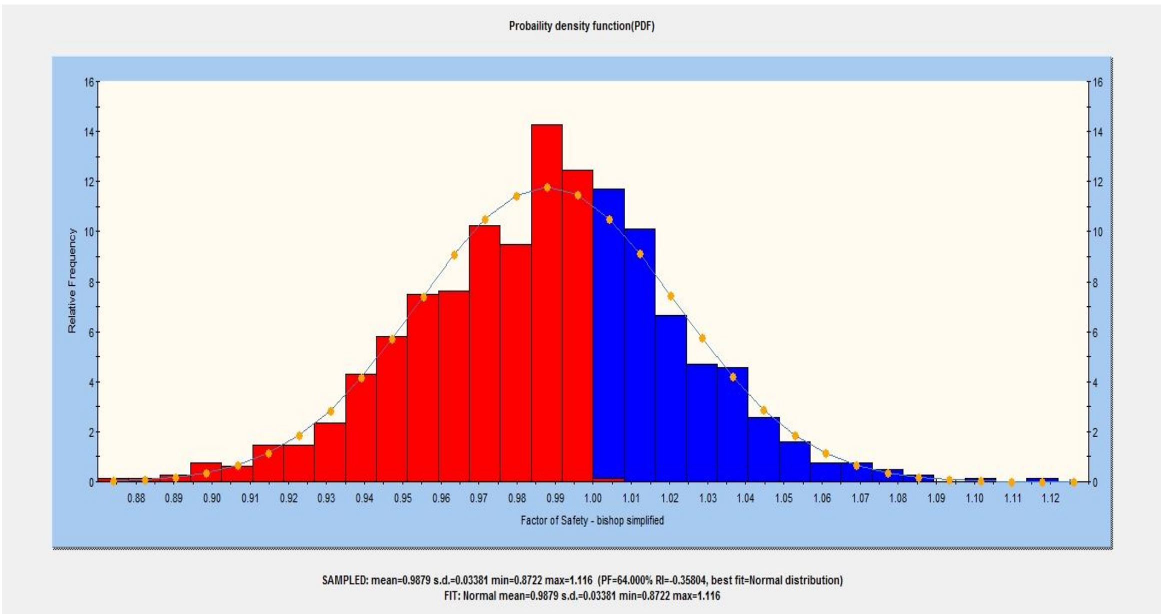


Fig 5.8 (b) probability distribution of safety factor results of SL 1Section under dynamic dry condition

Further, the probability of safety factor distribution for SL 1 Section under normal (Gaussian) distribution indicates the mean safety factor value as 0.988 which is the dominant value to unstable regions.

5.3.8 Probabilistic Stability Analysis of SL1 Section under Dynamic Saturated Condition

The most anticipated worst case for any slope is a condition when it is fully saturated and subjected to an earth quake. The same condition was considered for SL 1 section and Probabilistic stability analysis was performed.

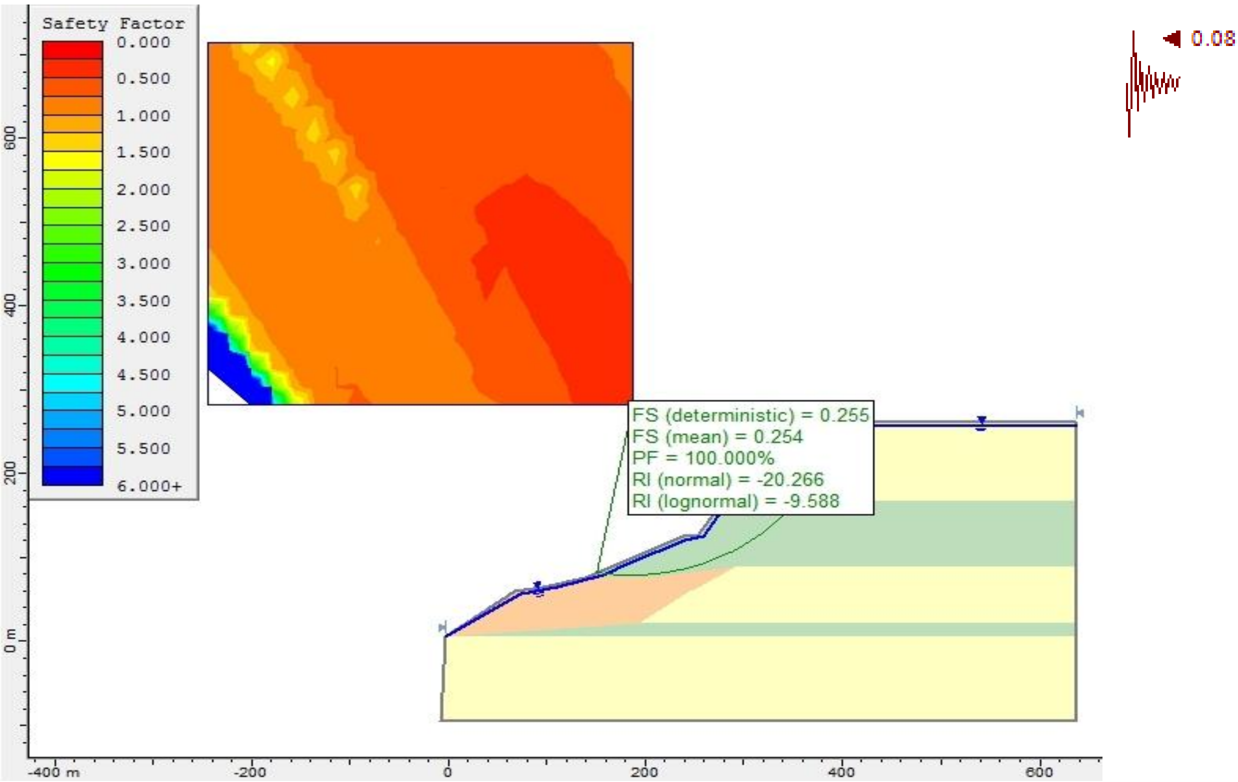


Fig 5.9 (a) SLIDE safety factor and probability of failure results of SL 1under dynamic saturated condition

The probabilistic stability analysis for SL1 slope section under the case where earth quake can possible occur during fully saturated condition has resulted (Fig. 5.9 (a)) in to a probability of failure of 100% with reliability indexes of -20.266 and -9.588 under normal and lognormal condition, respectively.

Further, the probability distribution of safety factors (Fig. 5.9 (b)) indicates that the slope is completely in a safety factor range of less than zero with a mean value of 0.254.

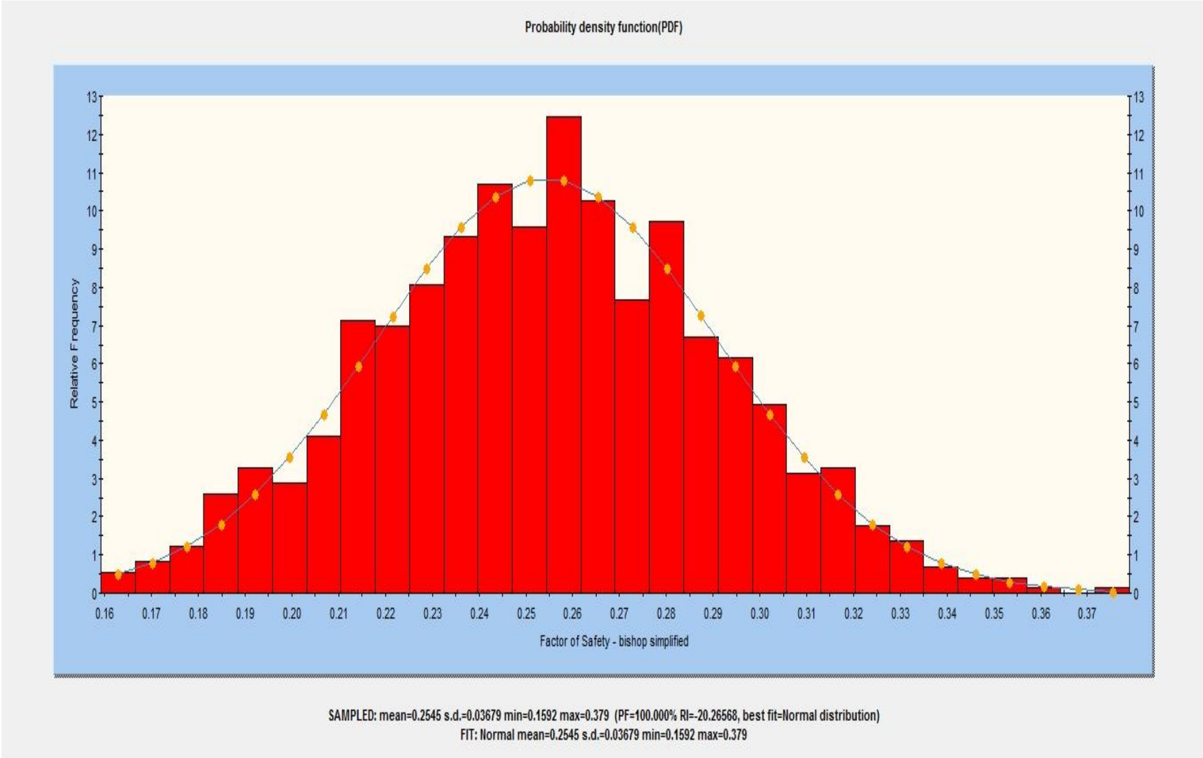


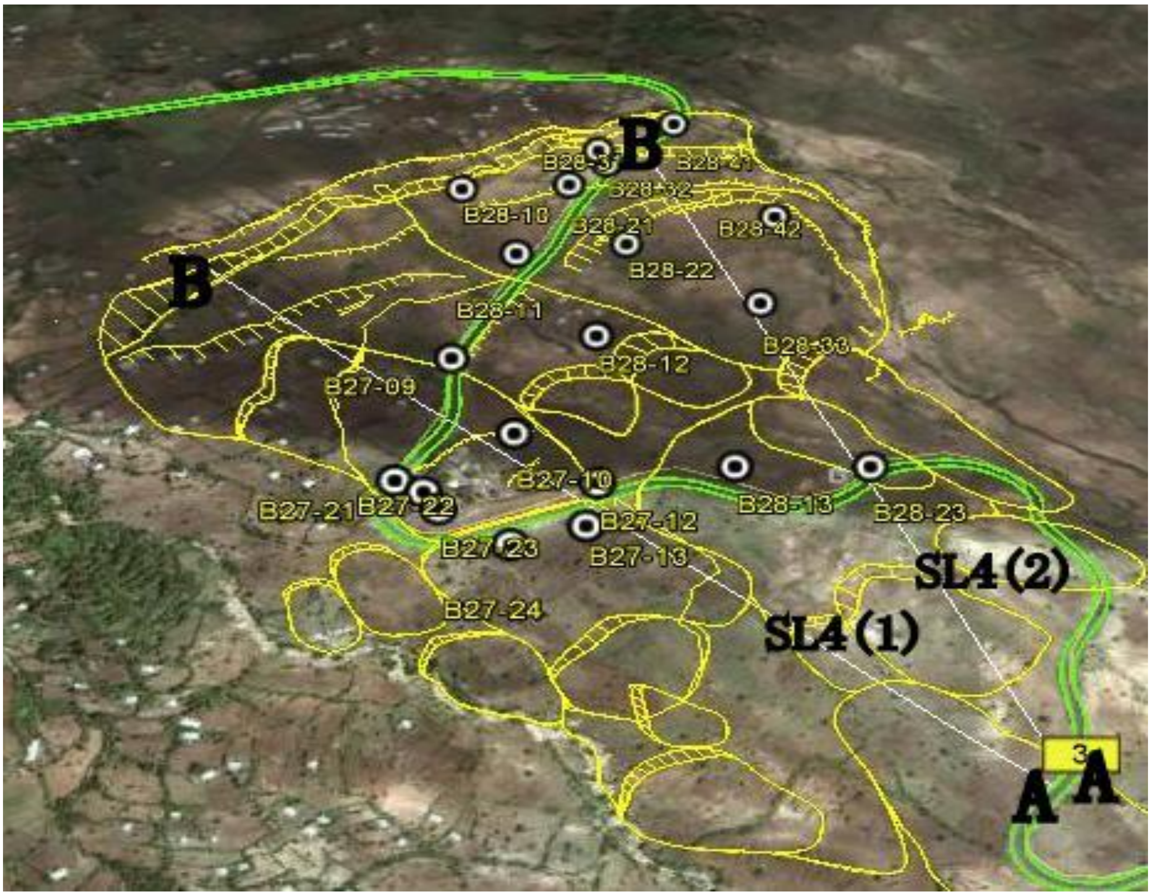
Fig 5.9 (b) Probability distribution of safety factor results of SL 1under dynamic saturated condition

5.3.9 Deterministic Stability Analysis of SL 4 (1) Slope Section Under Static Dry Condition

This particular slope section (SL 4 (1)) is located on the upper most part of Dejen side of the gorge where slow but frequent sliding is taking place over past few years. During the present study, for detailed analysis the big slide mass was divided in to two traverses with particular emphasis to the road and upper scarp face. From these traverses prepared on satellite image as shown in Fig 5.10 slope geometry was deduced. Further, with the secondary data on borehole log as well as depth to ground water table, slope cross section was prepared.

The slope mass is characterized by thick colluvium cover with boulder to clay sized rock material of gypsum, limestone, silt and clay origin. For safety factor computation and slope analysis of this failure mass the input values were directly adopted from tables proposed Hoek and Bray (1981) by careful conditioning to site specific material characteristics. During field visit observations, secondary data review and the Google earth image interpretation it was observed/ deduced that the failure mass is surrounded by two side streams and the ground water table is at shallow depth which passes through colluvium deposit. Fig.5.10 shows the slope geometry, manifestations of slope instability and location of bore holes.

For computation, in both approaches critical failure surface using Grid search method of SLIDE software was used in similar manner as discussed earlier in this chapter. However, as the main concern of this study was to assess the slope condition with respect to effect on the road, the window of search in software was placed on the road to determine the slip surface. Accordingly, slip surfaces were produced considering the road. Possible failure surfaces are also shown in Fig. 5.11.



(Source: JICA, 2011)

Fig 5.10 Satellite image showing entire failure mass of Slope Section SL4 (1), scarp faces and borehole location

The input data used for critical slope Section SL 4 (1) under static dry condition and output results for deterministic analysis by different methods is presented in Table 5.2.

Table: 5.2 Deterministic input values and static dry safety factor results of Travers 1 (SL4, 1)

Material type	C (kN/m^2)	Ø (deg.)	γ (kN/m^3)	SLIDE Factor of safety results of failure mass emphasizing road.				
				Janbu	Bishop	Janbu corrected	GLE	Spence r
Colluvium	0	34	17	1.532	1.604	1.605	1.599	1.600

Silt and clay soil	11	17	16					

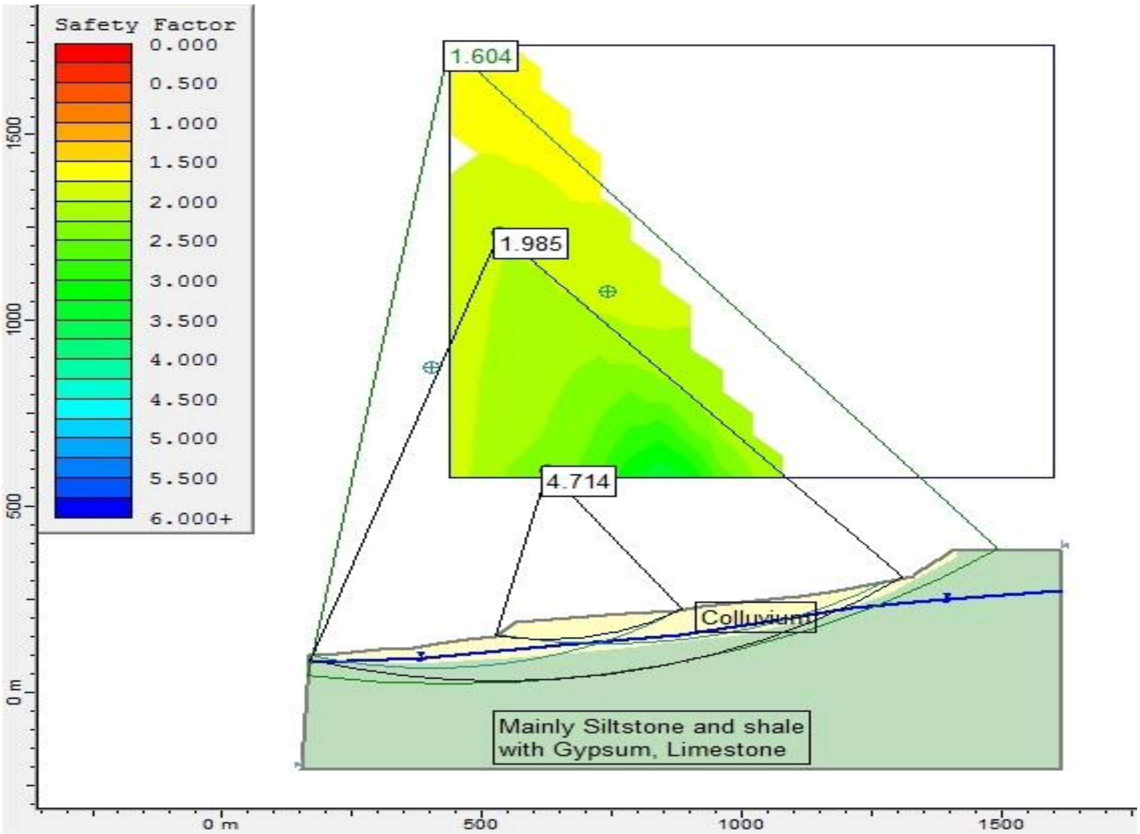


Fig 5.11 SLIDE Safety factor value under static dry by Bishop simplified and possible slips surfaces

Based on the geomorphology of the sliding mass and scarp faces of moving blocks possible slip surfaces of the failure mass was produced which are shown in Fig 5.11. The perusal of stability analysis results indicates that the assumed slip failure masses above the main moving mass have safety factors less than it except the main scarp face which is at its verge of failure with safety factor of 1.09. Under static dry condition the lowest safety factor in which the water table was assumed at its lowest point is 1.6. As the Scarp face is the point where the failure initiates, it is very important to calculate its safety from stability point of view. The results of this analysis exactly supported what has been noticed in the field that the slope was at the verge of failure. Further, this result dictates the necessity of analysis at saturated and even at dynamic condition to understand the general stability state under anticipated adverse conditions.

5.3.10 Deterministic Stability of SL 4(1) Section Under Static Saturated Condition

Although the area is located in high seismic intensity zone as discussed previously there was no slope failure that happened due to earth quake in the area. Thus, the most probable anticipated worst case for landslide behavior would be under full saturated condition.

The typical slope section for anticipated worst condition with possible slip surface and calculated safety factor is shown in Fig. 5.12.

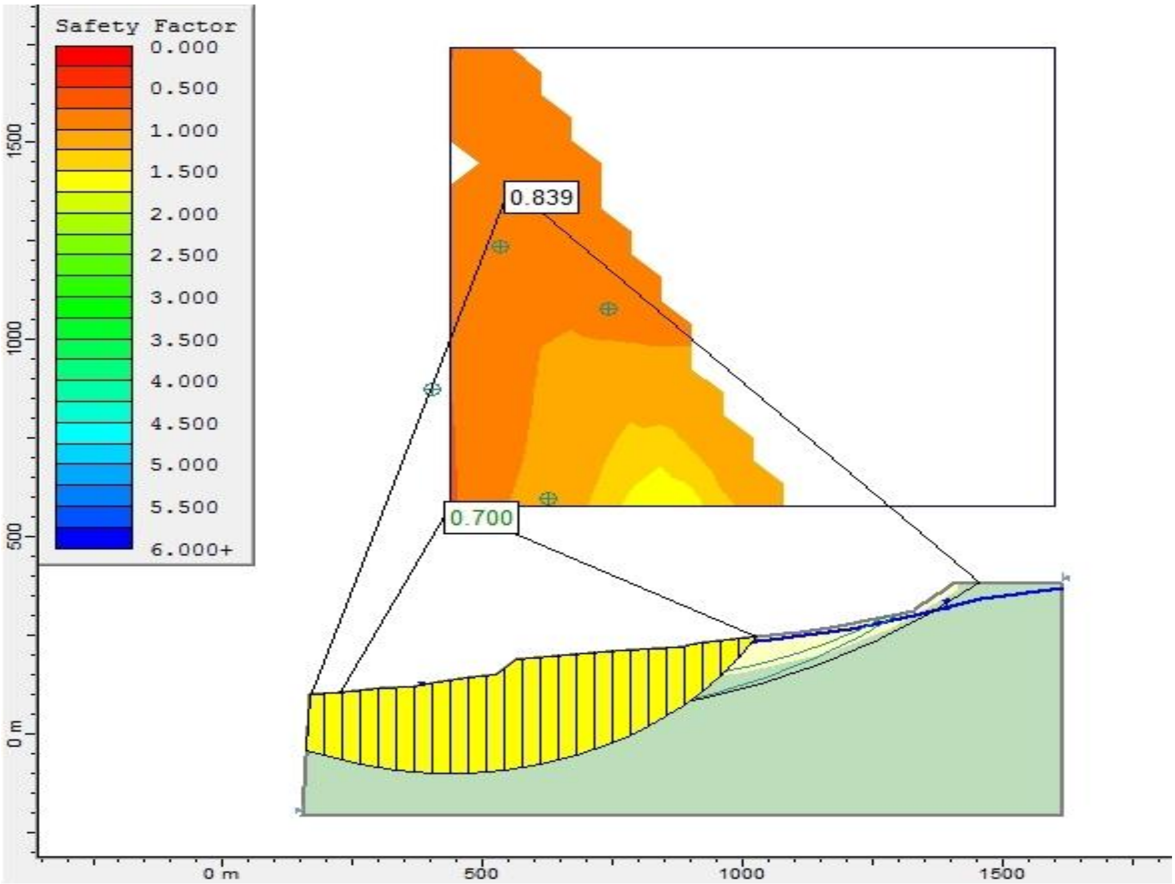


Fig 5.12 SLIDE Safety factor value for SL 4(1) Section under static saturated condition

The stability analysis results of SL 4(1) Section under static saturated condition indicate much difference in stability condition under saturated and dry state. The lowest possible safety factor of failure mass is 0.7 which occur at the bottom of the big scarp face and the entire section demonstrates safety factor of 0.83. Under anticipated worst static saturated conditions Slope Section SL (1) would be unstable. The same fact has been reported in previous studies as the progressive failure in this slope section has been observed during rainy seasons over the past years. During rainy season it is most likely that the slope would be fully saturated due to infiltration of rain water and general rise in groundwater table due to recharge. Thus, the results during the present study very well validates to the actual slope condition under worst condition.

5.3.11 Deterministic Stability of SL 4 (1) Section Under Dynamic Dry Condition

As already discussed the study area fall within 7MM earth quake seismic zone and is located at the margin of Ethiopian rift system which is seismically active therefore, significant earthquake hazard exists in the area. Further, none of the landslide and associated slope instability problems has been reported in the area which possibly triggered due to seismic activities. However, possibility of future seismic activities in the area is most likely. Therefore, it is mandatory to incorporate possible seismic factor to represent anticipated worst conditions while performing stability analysis. Therefore, possible horizontal earthquake acceleration was considered during stability analysis. Thus, the slope stability analysis was made for anticipated condition represented by dynamic dry condition. It was intended to examine the slope condition under the seismic loading when there is no effect of water saturation over the slope.

The slope section representing the geometry of SL 4 (1) slope section under dynamic dry condition and possible slip surface considered for this analysis are shown in Fig. 5.13.

The input parameters used for analysis of stability condition of SL 4 (1) slope section under static dynamic condition were similar to what were taken for static dry condition except that seismic coefficient of 0.08g was considered.

The stability analysis by deterministic approach using SLIDE indicates that safety factor of 1.144 for dynamic dry condition (Fig. 5.13). Thus, from natural slope stability point of view the slope is stable under dynamic dry condition as the safety factor is more than 1.0. However, from engineering design application point of view the slope can be considered as unstable as the safety factor is less than 1.5.

5.3.12 Probabilistic analysis of SL 4 (1) Slope Section Under Static Dry Condition

Due to heterogeneity and material under consideration as well as the level of confidence in in-situ test for material parameterization, the unit number that determines the safety or failure of slope is not enough for engineering decision for slope design by itself. Therefore, for this slope section (SL 4 (1) random values of input parameters has been generated in SLIDE software for probabilistic analysis i.e. for probability of failure and reliability of slope determination. Stability analysis was made for static condition and without taking any effect of water saturation (dry condition).

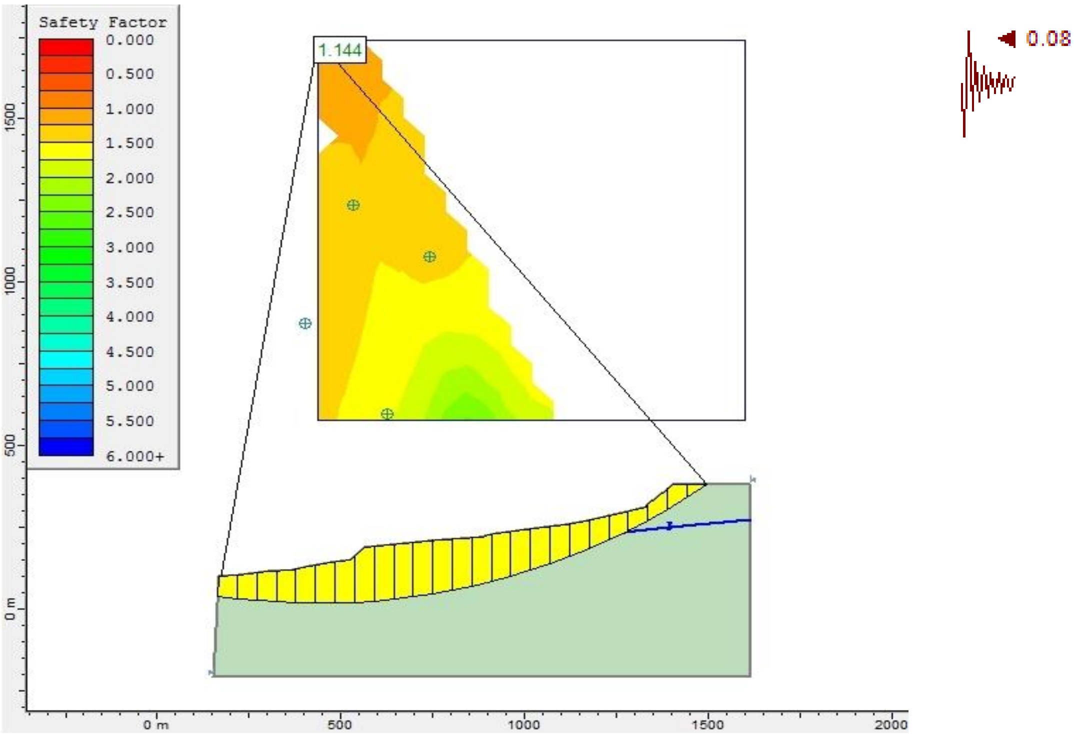


Fig 5.13 SLIDE deterministic safety factor calculation results for SL 4 (1) Slope Section under dynamic dry condition

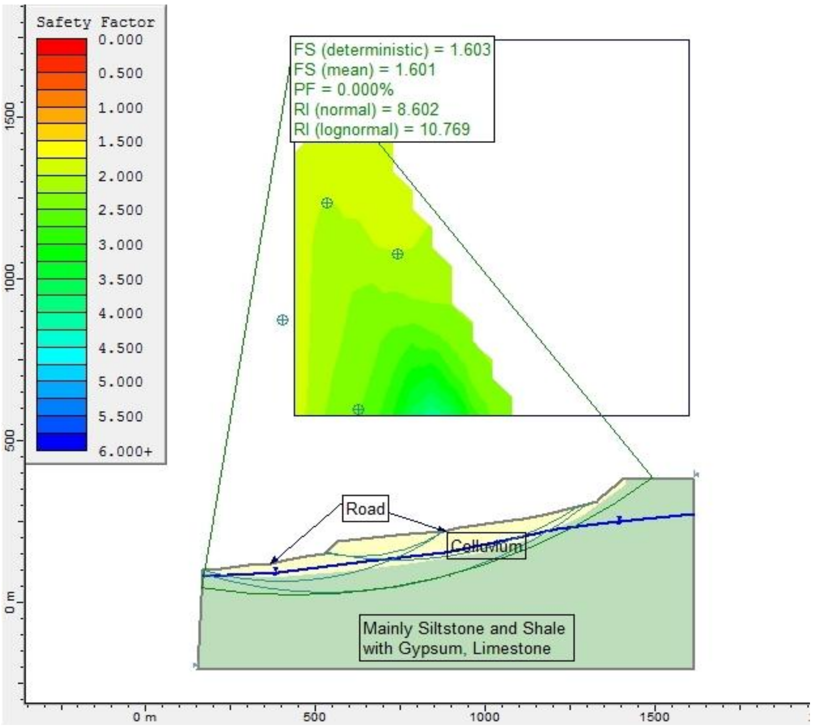


Fig 5.14 (a) SLIDE safety factor and probability of failure results of SL 4(1) under static dry condition

Under static dry condition slope section SL 4 (1) has 0% probability of failure and relatively higher reliability index. Also, the distributions of safety factor for slope section SL 4 (1) under static dry condition around its mean value is shown in Fig. 5.14 (b).

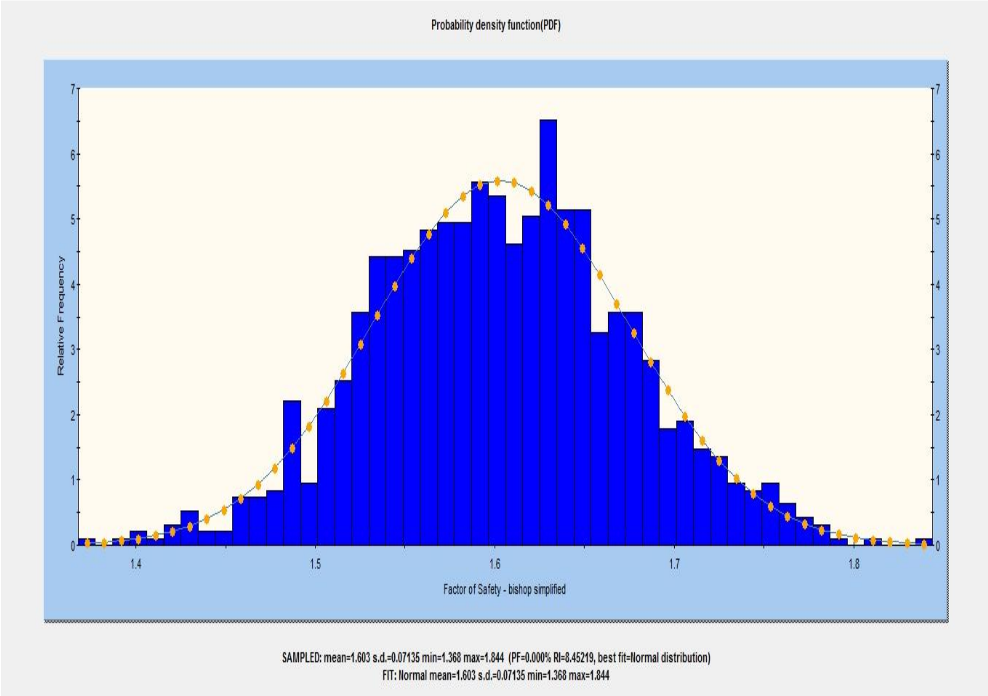


Fig. 5.14 (b) SLIDE Probability distributions of SL 4(1) slope section factor of safety under static dry condition

5.3.13 Probabilistic stability analysis of SL 4(1) Under Static Saturated Condition

The analysis of slope Section SL 4(1) under static saturated condition was carried out by assuming that the slope is nearly fully saturated.

Further, all the input parameters and calculation procedures were followed in the similar manner as that was adopted for analysis under probabilistic static dry condition with the exception of adding saturated unit weight of material for the said case.

Also, the ground water table was assumed to be at or near the top surface of the slope.

The analysis results for slope section SL 4 (1) under static saturated condition, probable anticipated worst case, provided a probability of failure of 100% and minimum reliability indexes of -6.079 under normal distribution and -5.179 under log normal distributions (Fig.5.15 (a)).

Further, the probability distribution of safety factor given in Fig. 5.15 (b) show the mean factor of safety of slope under consideration is 0.763 among a range of safety factor values the slope have.

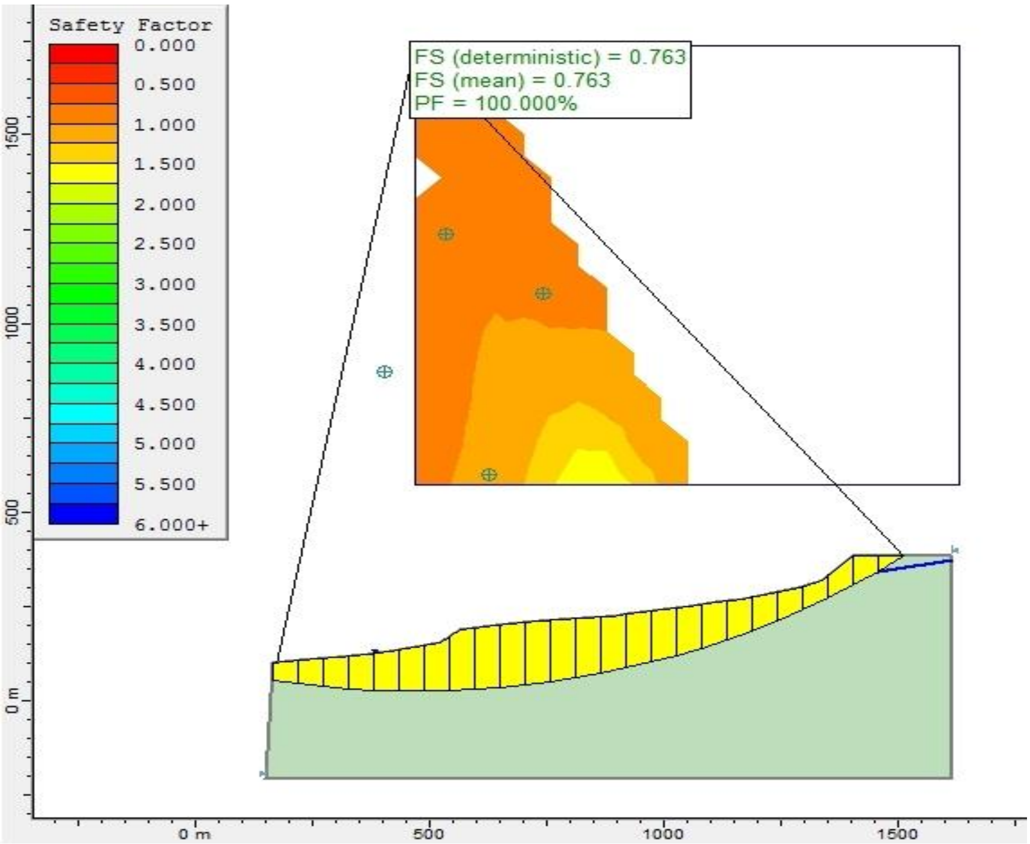


Fig 5.15 (a) SLIDE safety factor and probability of failure results of SL 4(1) slope section under static saturated condition

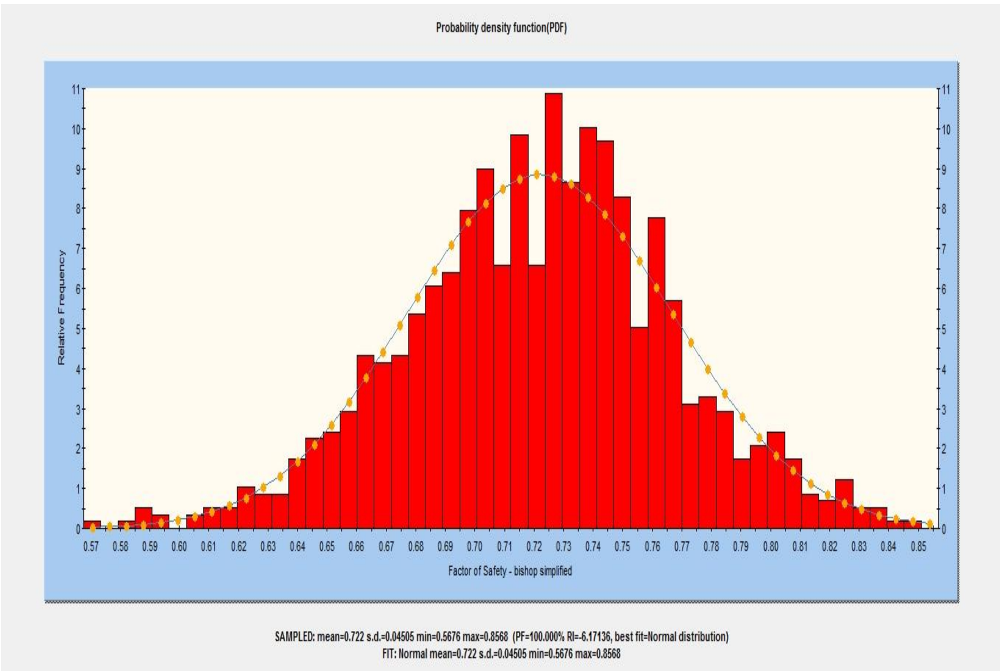


Fig 5.15 (b) SLIDE probability distributions of SL 4(1) slope section factor of safety under static saturated condition

5.3.14 Probabilistic stability analysis of SL 4(1) Slope Section Under Dynamic Dry Condition

The possible worst case considered for SL 4 (1) slope section was a case in which the slope is subjected to earth quake in a dry season and the slope response to such condition was analyzed by incorporating a horizontal seismic coefficient of 0.08g.

The probabilistic stability analysis for SL 4 (1) slope section under the case where earth quake can possible occur during dry condition has resulted (Fig. 5.16 (a)) in to a probability of failure of 0.1% with reliability indexes of 2.878 and 3.055 under normal and lognormal condition, respectively.

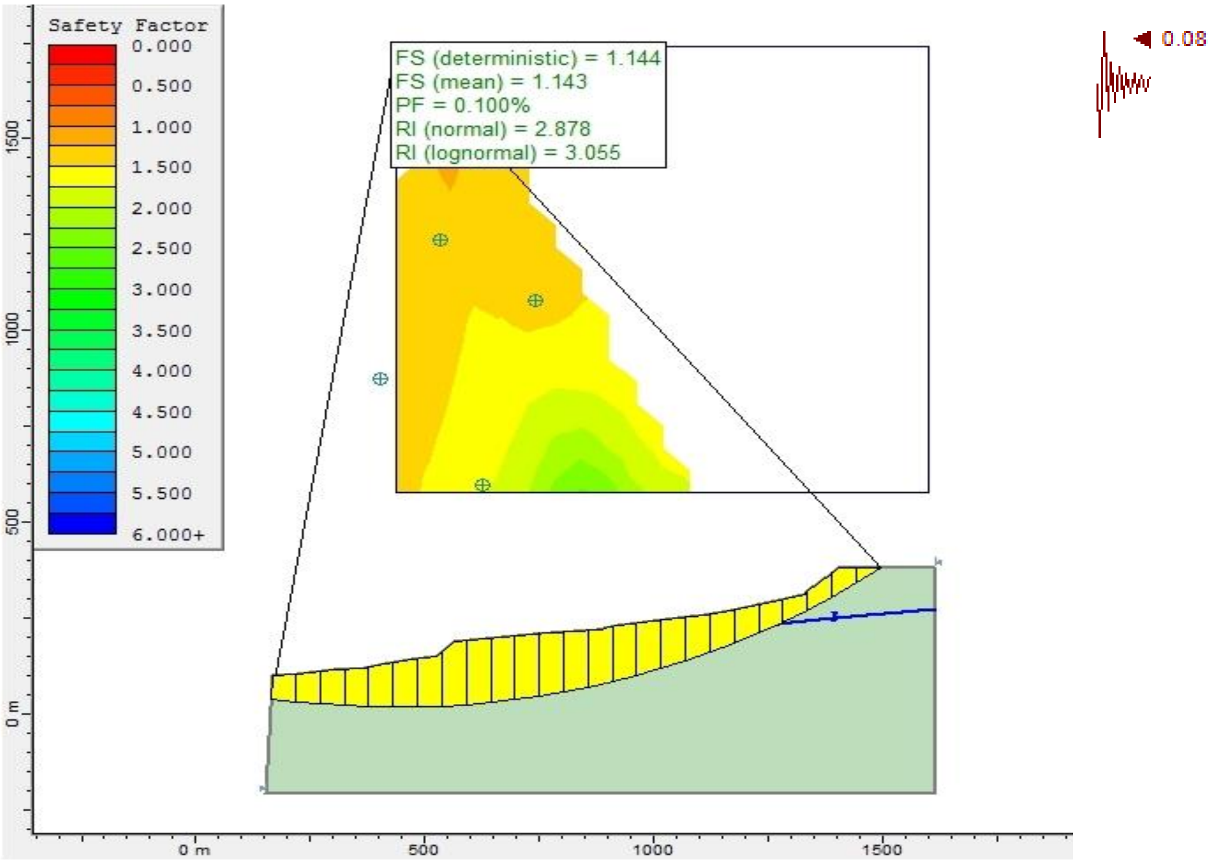


Fig 5.16 (a) SLIDE safety factor and probability of failure results of SL 4(1) slope section under dynamic dry condition

5.3.15 Probabilistic Stability Analysis of SL 4(1) Slope Section Under Dynamic Saturated Condition

The Slope Section SL 4 (1) was further analyzed for anticipated worst condition represented by dynamic loading under fully saturated condition. The input parameters used for this condition were similar to that what were used for dynamic dry condition except the saturated unit weight was considered for analysis. Thus, the analysis was carried out by considering the full water saturation and the seismic effect was considered by taking horizontal seismic coefficient of 0.08g.

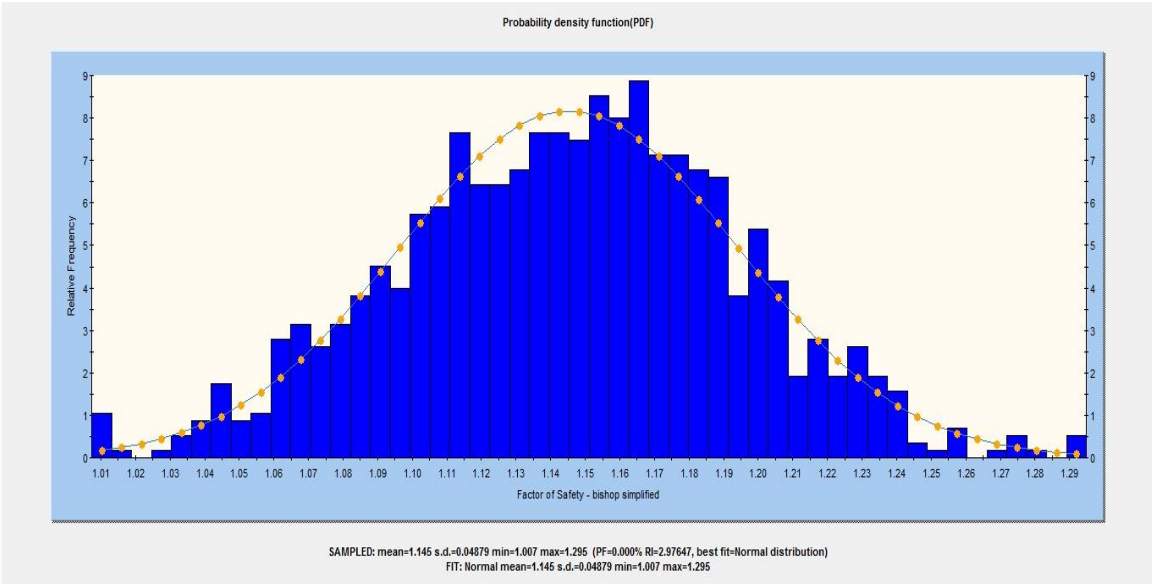


Fig 5.16 (b) SLIDE probability distributions of SL 4(1) slope Section factor of safety under dynamic dry condition

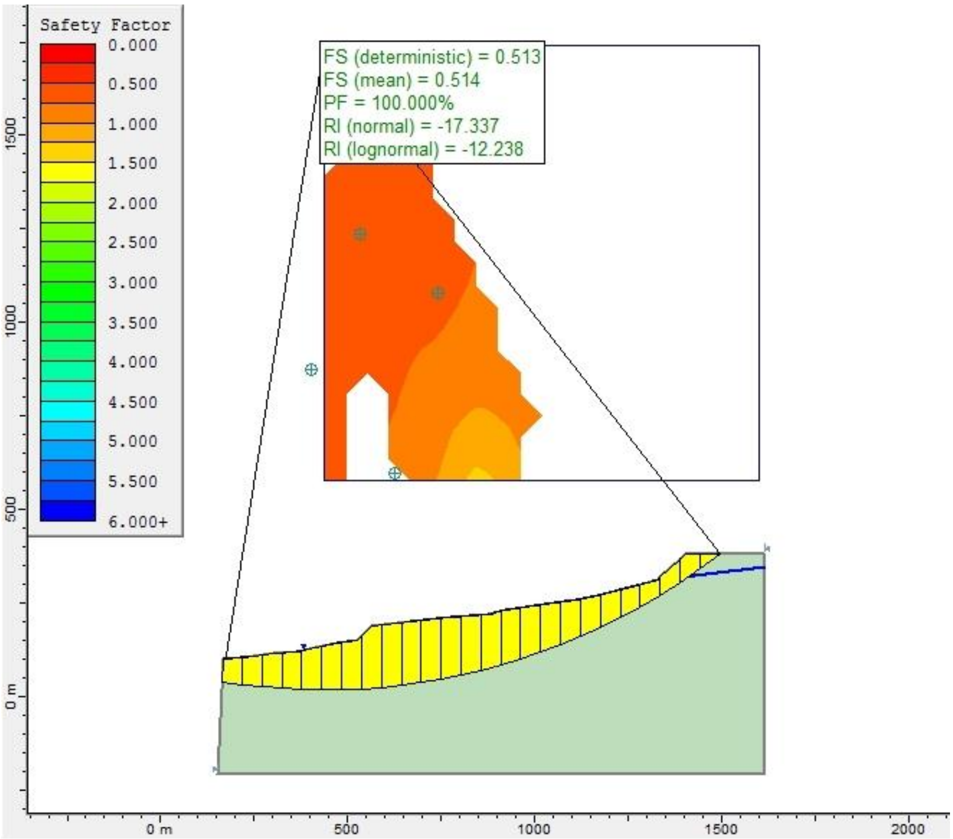


Fig 5.17 (a) SLIDE safety factor and probability of failure results of SL 4(1) Slope section under dynamic saturated condition

The stability analysis results for Slope Section SL 4 (1) indicates the probability of failure of the slope to be 100% under dynamic saturated condition. It also possesses the minimum reliability indexes of -17.337 and -12.238 under normal and log normal distribution, respectively.

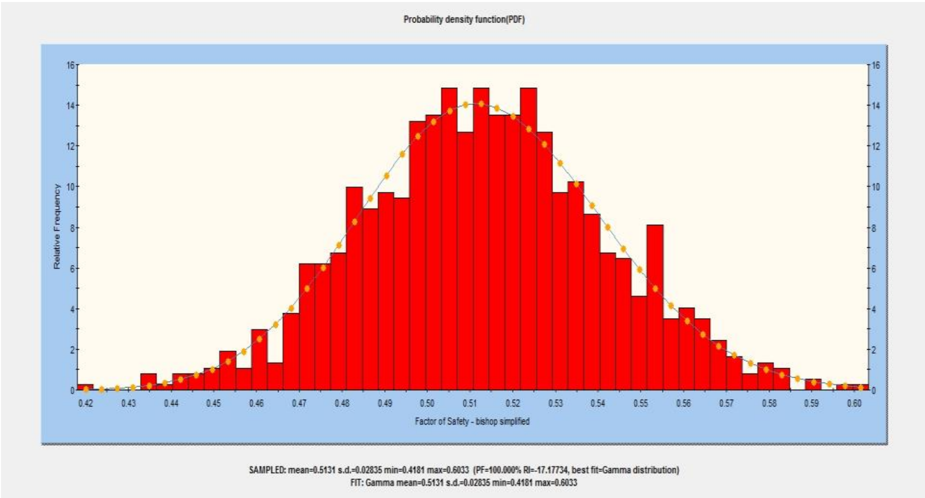


Fig 5.17 (b) SLIDE probability distributions of Slope Section SL 4(1) factor of safety under dynamic saturated condition

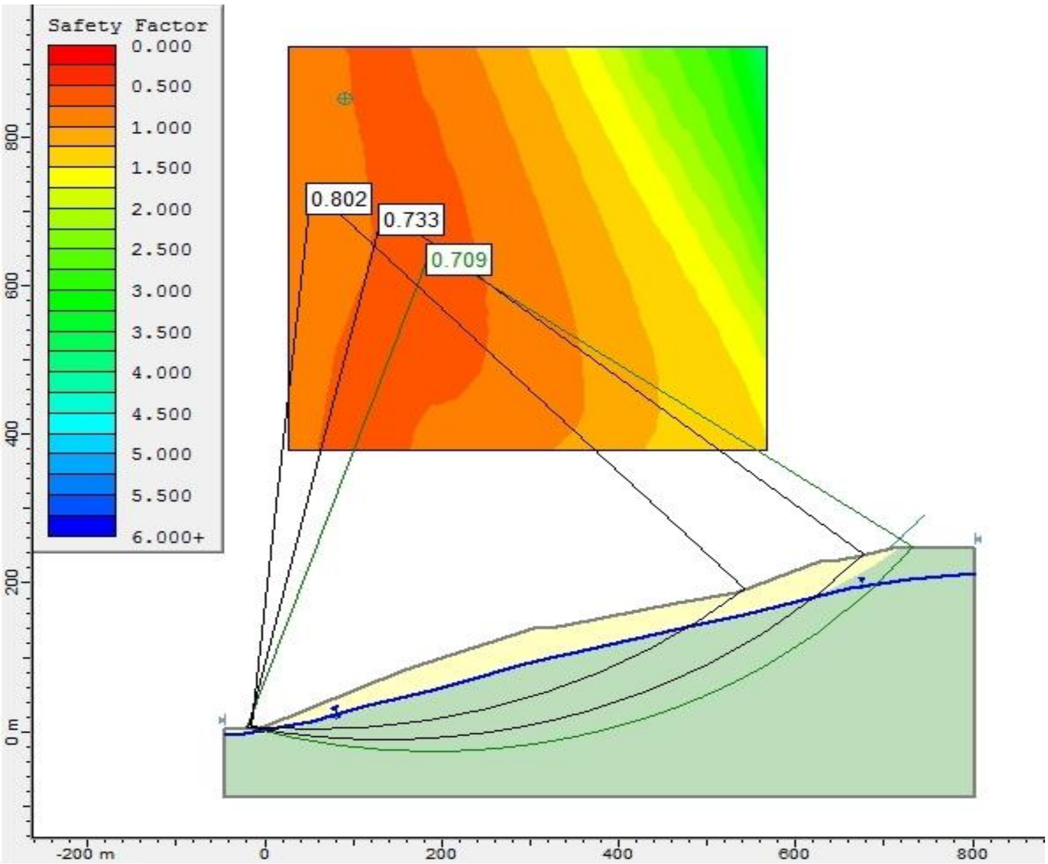
5.3.16 Deterministic Stability Analysis of Slope Section SL 4(2) Under Static Dry Condition

One of the two traverses selected for the Dejen side Colluvium side is SL 4 (2) as already shown in Fig 5.10. The instability manifestation as observed on the slope extends from the big scarp on the right side of damaged road on the way down to the gorge up to where the assumed moving side mass block ends.

Even under static dry condition the stability condition of slope has nearly shown complete instability which reduces the further necessity of conducting further analysis for the remaining possible worst conditions. It is obvious that if the slope is unstable under static dry condition than with water saturation and dynamic loading its stability will further be reduced.

The geometry of slope and possible slip surfaces and calculated safety factors under static dry conditions are shown over slope section presented in Fig. 5.18.

Different failure surfaces were assumed for the stability analysis. The minimum safety factor for SL 4 (2) slope section under static dry condition comes out to be 0.709 which clearly indicates that the slope is unstable.



Fi g 5.18 Deterministic factor of safety for SL 4 (2) slope section for possible slip surfaces.

5.3.17 Probabilistic Stability Analysis of SL 4(2) Slope Section Under Static Dry Condition

For SL 4 (2) slope section random values of input parameters were generated in SLIDE software for probabilistic analysis i.e. for probability of failure and reliability of slope determination. Stability analysis was made for static condition and without taking any effect of water saturation (dry condition).

The slope section showing the geometry of slope SL 4 (2) and probabilistic stability analysis results are shown in Fig. 5.19 (a).

The stability analysis revealed that the probability of failure of the slope section SL 4 (2) is 99.490% with relatively smaller values for reliability index. To see how the ranges of factor of safety will be distributed around its mean value probability normal distribution was analyzed and presented in Fig. 5.19 (b).

5.4 Sensitivity analysis of colluvium slopes

For two colluvium slopes on both sides of the Abay gorge, as discussed above, sensitivity analysis was conducted. This sensitivity analysis was mainly conducted to have an idea that on which input parameter the slope stability is influenced the most. In general, it was intended to know the order of importance of various parameters that possibly influenced or may influence the stability condition of the slopes.

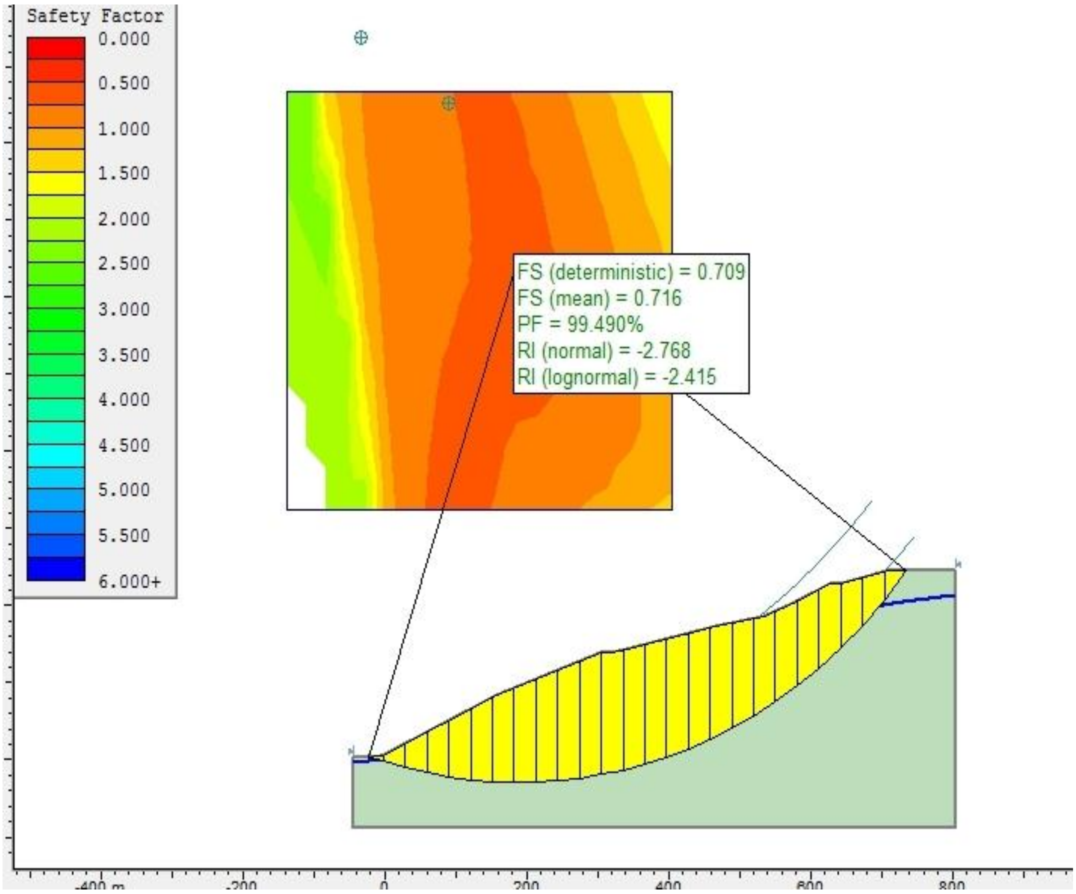


Fig 5.19 (a) SLIDE Probabilistic stability analysis of Slope Section SL 4(2) under static dry condition

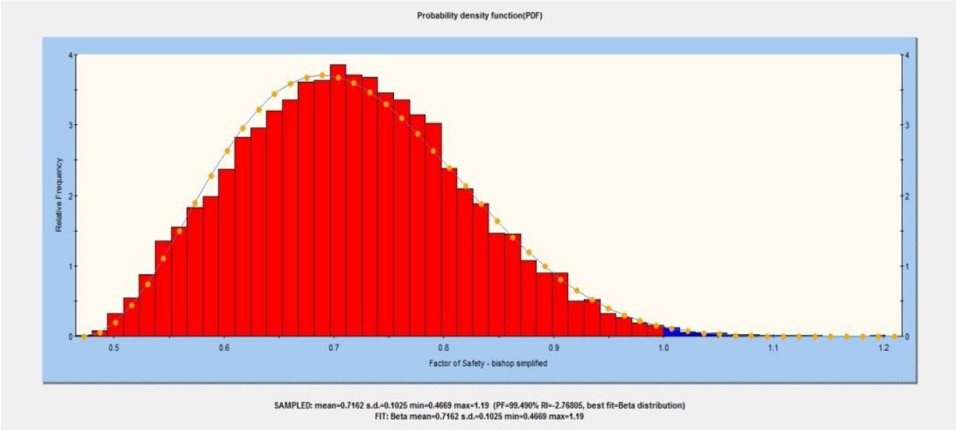


Fig 5.19 (b) SLIDE probability distributions of SL 4(2) factor of safety under static dry condition

In the present study, sensitivity of total of three traverses selected within two colluvium slopes (one on the Gohatsion side described as SL 1 and two on the Dejen side on big slide referred as SL 4(1) and SL 4(2)) was carried out.

The sensitivity of various input parameters with respect to factor of safety for slope sections SL1, SL 4 (1) and SL 4 (2) is presented through Fig. 5.20 (a), (b) and (c), respectively.

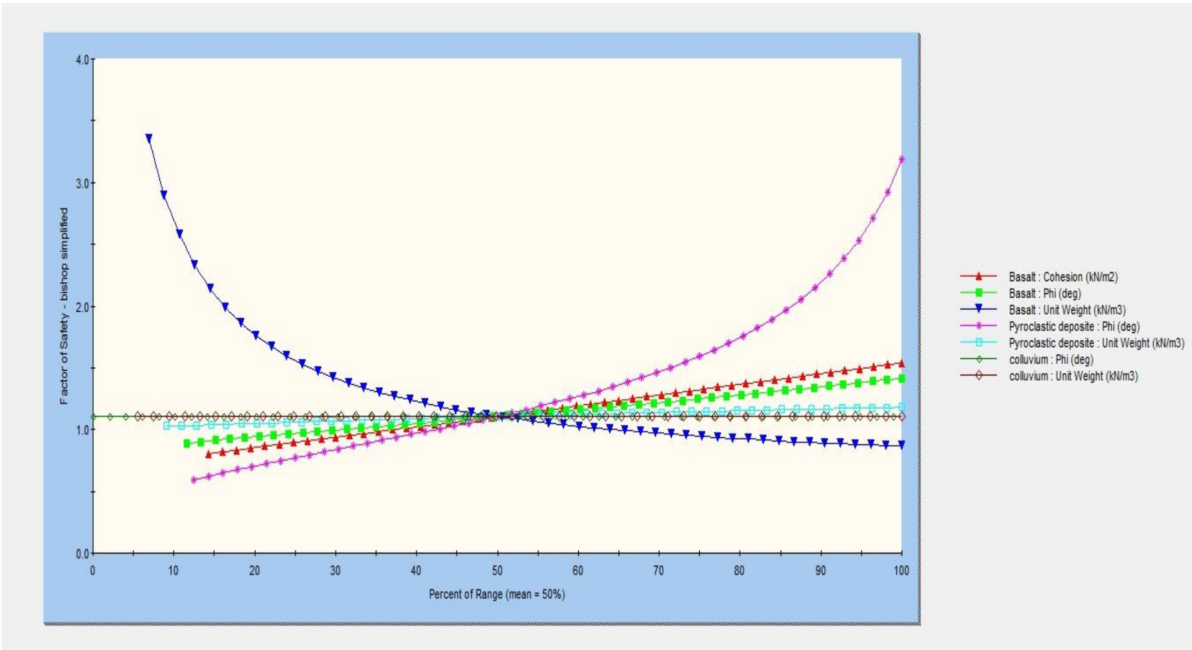


Fig 5.20 (a) Sensitivity result of factor of safety against input parameters for SL1

For slope section SL 1 the parameters for which sensitivity on factor of safety was assessed are; Cohesion of Basalt (C_b), Angle of friction of Basalt (ϕ_b), Unit weight of Basalt (γ_b), Angle of friction of Pyroclastic deposity (ϕ_p), Unit weight of Pyroclastic deposity (γ_b), Angle of friction of Colluvium (ϕ_C) and Unit weight of Colluvium (γ_C).

A perusal of Fig. 5.20 (a) clearly indicates that in terms of order of importance with respect to sensitivity of various parameters on Factor of safety the parameters in descending order can be written as;

(i) Unit weight of Basalt (γ_b),

(ii) Angle of friction of Pyroclastic deposity (ϕ_p),

(iii) Cohesion of Basalt (C_b),

(iv) Angle of friction of Basalt (ϕ_b),

(v) Angle of friction of Colluvium (ϕ_C) and

(vi) Unit weight of Colluvium (γ_C).

Thus, from above sensitivity analysis it may be concluded that Unit weight of Basalt (γ_b), Angle of friction of pyroclastic deposity (ϕ_p) and Cohesion of Basalt (C_b) are the parameters which have significant effect on the stability of SL 1 slope. Parameters Angle of friction of Basalt (ϕ_b), Angle of friction of Colluvium (ϕ_C) and Unit weight of Colluvium (γ_C) are relatively less significant for stability of slope.

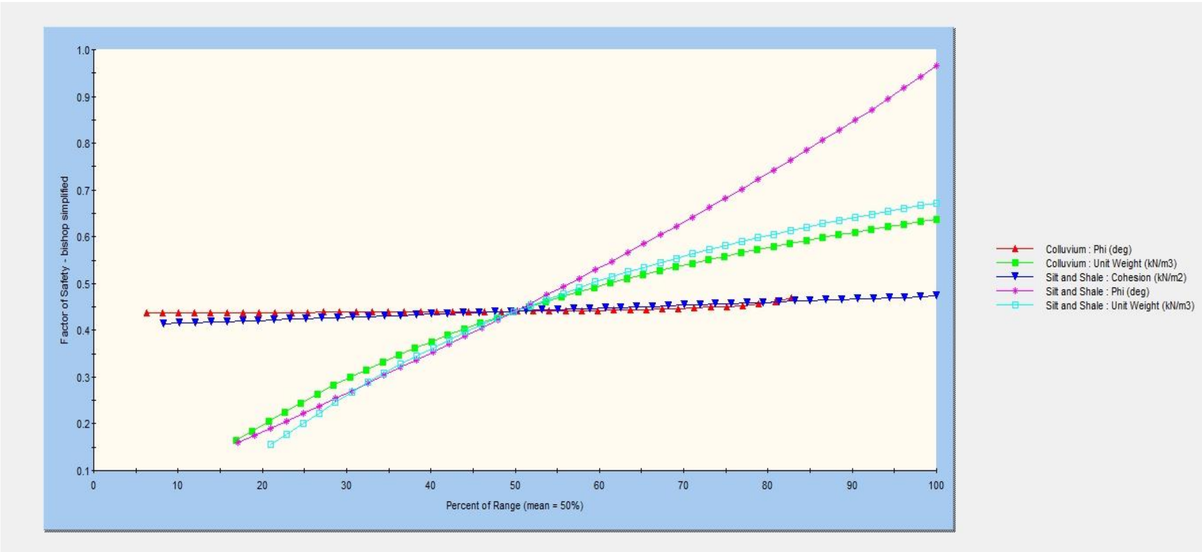


Fig 5.20 (b) Sensitivity result of factor of safety against input parameters for SL 4(1)

For slope section SL 4 (1) the parameters for which sensitivity on factor of safety was assessed are; Angle of friction of Colluvium (ϕ_C), Unit weight of Colluvium (γ_C), Cohesion of Silt and Shale (C_{ss}), Angle of friction of Silt and Shale (ϕ_{ss}) and Unit weight of Silt and Shale (γ_{ss}), A perusal of Fig. 5.20 (b) clearly indicates that in terms of order of importance with respect to sensitivity of various parameters on Factor of safety the parameters in descending order can be written as;

(i) Angle of friction of Silt and Shale (ϕ_{ss}),

(ii) Unit weight of Silt and Shale (γ_{ss}),

(iii) Unit weight of Colluvium (γ_C),

(iv) Angle of friction of Colluvium (ϕ_C),

(v) Cohesion of Silt and Shale (C_{ss})

From the above sensitivity analysis it may be concluded that Angle of friction of Silt and Shale (ϕ_{ss}), Unit weight of Silt and Shale (γ_{ss}) and Unit weight of Colluvium (γ_C) are the parameters which have significant effect on the stability of SL 4 (1) slope. Parameters Angle of friction of Colluvium (ϕ_C) and Cohesion of Silt and Shale (C_{ss}) are relatively less significant for stability of slope.

For slope section SL 4 (2) the parameters for which sensitivity on factor of safety was assessed are; Angle of friction of Colluvium (ϕ_C), Unit weight of Colluvium (γ_C), Cohesion of Silt and Shale (C_{ss}), Angle of friction of Silt and Shale (ϕ_{ss}) and Unit weight of Silt and Shale (γ_{ss}).

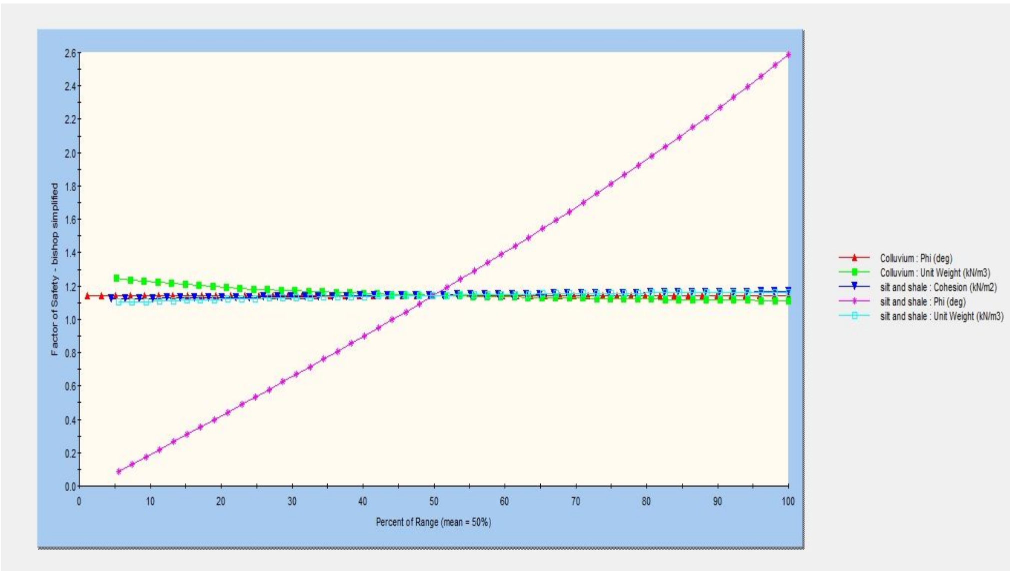


Fig 5.20 (c) Sensitivity result of factor of safety against input parameters for SL 4(2)

A perusal of Fig. 5.20 (b) clearly indicates that in terms of order of importance with respect to sensitivity of various parameters on Factor of safety the parameters in descending order can be written as;

(i) Angle of friction of Silt and Shale (ϕ_{ss}),

(ii) Unit weight of Colluvium (γ_C),

(iii) Angle of friction of Colluvium (ϕ_C),

(iv) Cohesion of Silt and Shale (C_{ss}),

(v) Unit weight of Silt and Shale (γ_{ss}).

From the above sensitivity analysis it may be concluded that Angle of friction of Silt and Shale (ϕ_{ss}) and Unit weight of Colluvium (γ_C) are the parameters which have significant effect on the stability of SL 4 (1) slope. Parameters Angle of friction of Colluvium (ϕ_C), Cohesion of Silt and Shale (C_{ss}), Unit weight of Silt and Shale (γ_{ss}) are in significant for stability of slope.

5.5 Slope Stability Analysis of Rock Slopes by Combined Deterministic and Probabilistic Approaches

5.5.1 Plane Failure Analysis

Rock slope stability critically depends on complex relationships between slope geometry and strength parameters among others. During the present study, in order to characterize the rock mass of slope of interest, discontinuity data pertaining to Rock Mass Rating System was collected.
The Geo-mechanics Classification or the Rock Mass Rating (RMR) System was initially developed at South African Council of Scientific and Industrial Research (CSIR) by Beniawisky (1973) on the basis of his experiences in shallow tunnels in sedimentary rocks (Kaiser et al., 1986) which later has undergone several modifications as cited in Singh and Goel (1999).

In general, RMR value is determined as an algebraic sum of ratings for six parameters namely; Uniaxial compressive strength (UCS) of intact rock material, Rock quality designation (RQD), Joint or discontinuity spacing, Joint condition, Groundwater condition and Joint orientation parameters.

During the present study the RMR data was collected from the rock slope which showed manifestations for potential instability. Data pertaining to Uniaxial compressive strength (UCS) of intact rock material, Rock quality designation (RQD), Joint or discontinuity spacing, Joint condition, Groundwater condition and Joint orientation parameters was collected at several locations. The average RMR value for the rock mass was 60 which indicates the rock mass is fair rock.
Table 5.2.1 data pertaining RMR and average RMR value for SL 3

Location	UCS		RQD		Condition of discontinuity					Spacing	GW condition	Orientation of discontinuity	RMR value
	SRV	UCS	JV	RQD	Persistence	Aperture	roughness	Infilling	weathering				
SL 3	26	54	11	75	1	5	2	4	3	7	10	-5	60

Further, Calculations of safety factor on this section has indicated that the slope is stable for both existing and anticipated adverse condition as discussed later in this section and shown in Table 5.3. However, during the field visit it was observed that rock blocks has failed. To account for such confusion the analysis was emphasized on individual block stability that might contribute to the whole mass failure rather than analyzing the entire slope.

Block Failure in this jointed rock slope is controlled by three discontinuity sets the horizontal bedding and two other joint sets dipping away from the slope mass with block movement in the direction of joint plane. To observe how these orientations of discontinuity sets contribute to failure, kinematic check was conducted that support analysis consideration at block level as the slope seems susceptible to failure.

General condition for Plane failure

In general, it was realized that the geometry of slope section SL3 has upper slope which is inclined at 20^0, therefore the analysis for Plane mode of failure was performed by ROCPLANE which operate by modified analytical approach proposed by Sharma et al. (1995) principles taking its advantage of considering upper slope angle. In the modified Sharma et al. (1995) technique the following general conditions must be satisfied;

(i) The failure plan must strike parallel or nearly parallel (approx. $\pm$ 20%) to the slope face.

(ii) The dip of the failure plane must be smaller than the dip of the slope face ($\alpha_p<\alpha_f$).

(iii) The angle of internal friction (φ) of the failure plane must be smaller than the dip of the failure plan ($\varphi<\alpha_p$).

(iv) The upper slope surface and tension crack must be inclined.

(v) The inclination of the upper slope must be smaller than the inclination of the failure plan ($\alpha_p>\alpha_s$).

(vi) The tension crack must be present on the upper slope surface.

Assumptions

For modified technique following assumptions were considered;

(i) The tension crack is filled with water to a vertical depth of Z_w. The water from the tension crack seeps along the failure surface and escape out on the slope face where the failure surface daylights.

(ii) It is presumed that there is no resistance to sliding at the lateral boundaries of the slide.

As per the technique of plane failure analysis (Sharma et.al, 1995) the area, weight, horizontal water forces and uplift water forces are calculated by the following equations;

Area of sliding mass

$$A=(h-ZI)\cos ec\alpha \qquad \text{..........eq.5.1}$$

Where;

$$ZL = \frac{Zsin\alpha t}{sin\alpha t - tan\alpha p\; cos\alpha t} \qquad \text{..........eq.5.2}$$

$$ZI = ZL - IG \qquad \text{..........eq.5.3}$$

$$Z = h[\left(1 - \frac{Cot\alpha f}{Cot\alpha s}\right) + \sqrt{\frac{Cot\alpha f}{Cot\alpha p}} \quad X \; \frac{Cot\alpha p}{Cot\alpha s} - 1)] \qquad \text{..........eq.5.4}$$

$$IG= \frac{h[\sqrt{Cot\alpha f Cot\alpha p} - Cot\alpha f}{Cot\alpha s} \qquad \text{..........eq.5.5}$$

Weight 'W' of the sliding mass

$$W = \frac{1}{2}\gamma[(h + a)X - DZ] \qquad \text{..........eq.5.6}$$

Where, γ is the unit weight of the rock, '*X'* and 'D' are the slope distance AF and GF, respectively and 'a' is the height EF as shown in Fig.5.20.

$$X= h(\frac{Cot\alpha f}{Cot\alpha s} \; X \; \frac{tan\alpha p - tan\alpha f}{tan\alpha s - tan\alpha p}) \qquad \text{..........eq.5.7}$$

$$D = \frac{Z}{tan\alpha p Cos\alpha s - sin\alpha s} \qquad \text{..........eq.5.8}$$

$$a= h(\frac{tan\alpha s}{tan\alpha f}) \; X \; (\frac{tan\alpha p - tan\alpha f}{tan\alpha s - tan\alpha p}) \qquad \text{..........eq.5.9}$$

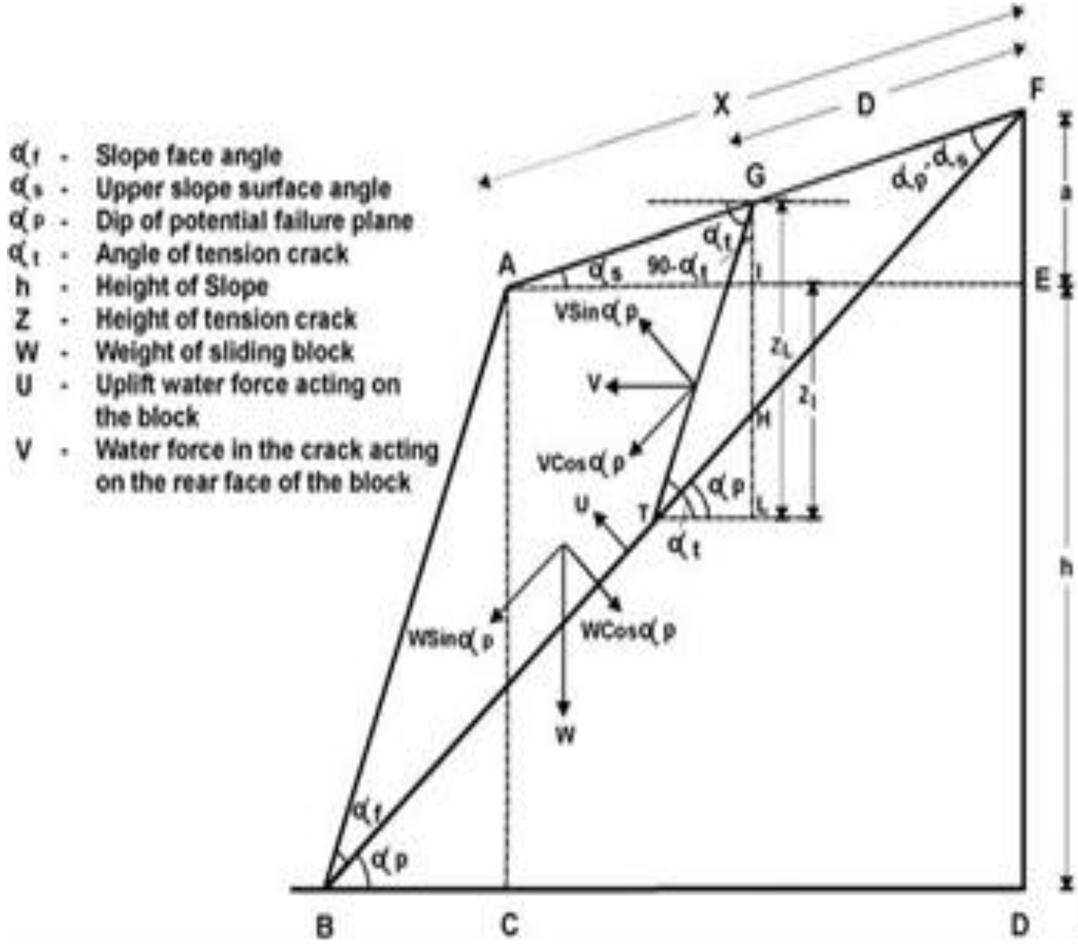
Horizontal water force, V

$V = \frac{1}{2}\gamma w\ Zw\ ^2 sin^2\ \alpha t$ eq.5.10

Where, γ_w is the unit weight of water and Z_w is the water level in the tension crack.

Uplift water force, U

$U = \frac{1}{2}\gamma w\ \ Zw\ sin\alpha t\ (h\ -\ Zf\ )cosec\alpha p$ eq.5.11



(Source: Sharma et al., 1995)

Fig. 5.21 Parameters and Geometric details to be considered in Modified analytical Plane failure approach

5.5.2 Kinematic Analysis

Plane Mode of Failure

The kinematic analysis is an analysis performed to check whether structural discontinuity orientation will result into slope failure or not by using the stereographic projection method and examines which modes of slope failure are possible in a jointed rock mass (Yoon et al., 2002) .

If the failure plane has day lighted to slope face at angle gentler than slope face and greater than friction angle the kinematic condition is satisfied for plane mode of failure. Further, the dip vector of a representative discontinuity set (failure plane) falls within the critical zone, "Kinematics" refers to the motion of bodies without reference to the forces that cause them to move (Goodman, 1989).

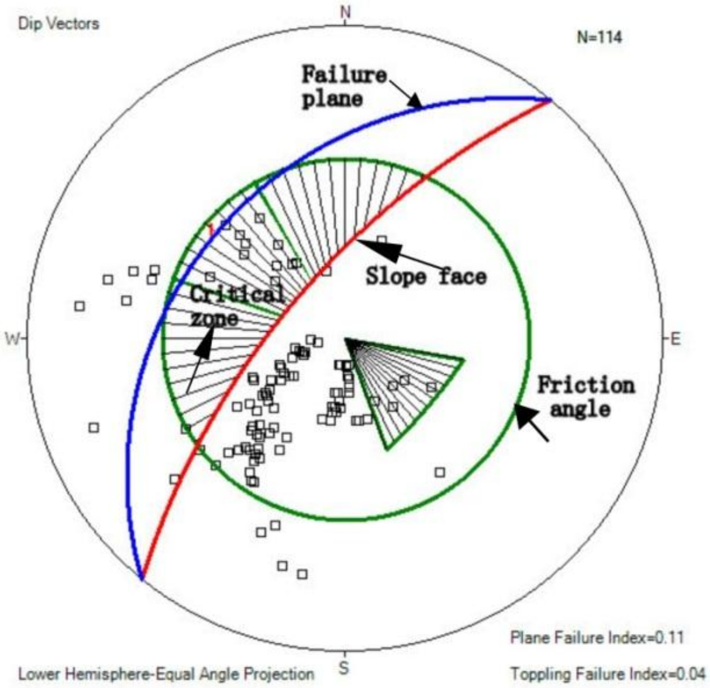


Fig 5.22 Stereographic analysis for possible plane failure

Angular relationships between discontinuities and slope surfaces are applied to determine the potential and modes of failures (Kliche, 1999). For the present slope dip and dip direction of each discontinuity orientation was collected and provided as input in to Dip Analyst software to perform the analysis.

The Markland test (Markland, 1972) is one of the kinematic analysis methods designated to evaluate the possibility of plane failure.

The kinematic condition for plane failure is;

$\alpha f > \alpha p > \varphi$ ………..eq.5.12

where; α_f is the slope inclination, α_p is the dip of the failure Plane and φ is the angle of internal friction.

If the condition specified in eq. 5.12 is satisfied, the Plane failure will possibly occur.

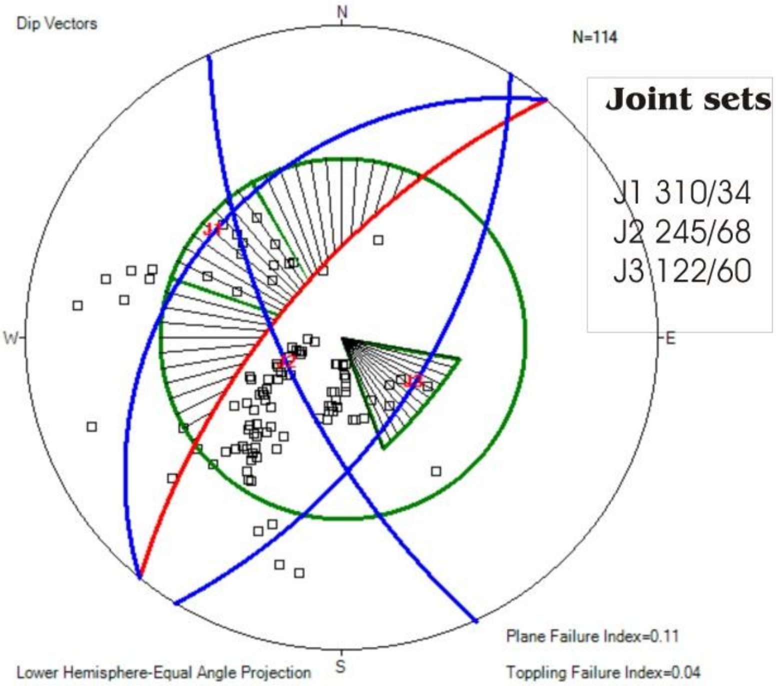


Fig 5.23 Preferred orientations of Joint sets

The greater amount of plane failure index is also evidence to support which mode of failure is likely. For the slope is then kinematically unstable further stability analysis is needed. Failure index is the ratio of total normalized weights of discontinuities that cause plane failure to total number of discontinuities. A higher index value for a given type of failure indicates a greater chance for that type of failure to occur, hence, plane failure in this case (Yonathan Admassu, 2012). A typical rock unit (Lower sandstone unit) where data was taken for preferred orientations of discontinuities developed three discontinuity sets.

Wedge Mode of Failure Analysis

Different types of failure modes are possible in rock slopes. It is the discontinuity orientation and its relationship with slope failure that determines which mode of failure is likely on the given slope. The wedge failure is a common type in jointed rock slopes than plane failure.
Markland (1972) proposed that wedge failures occur along the lines of intersection of joints and hence, these intersections must 'daylight' in the slope faces. In order for the line of intersection to 'daylight', the plunge of the intersection should be less than the dip angle of the slope face measured along the trend of the intersection. In addition, the plunge of the intersection must exceed the friction angle of the joint planes.

As per Hoek and Bray (1981) unstable wedges may be formed in rock slopes cut by at least two sets of discontinuities upon which sliding can occur.

In the present case from the stereographic results it was observed that three joint sets have intersected each other but in the opposite direction to the great circle of slope face. Hence, the kinematic conditions checked for wedge were not satisfied and it is also observed from failure indices that there is no evidence of wedge failure but small amount of rock toppling was observed. Further, if Kinematic condition is not satisfied there is no need of conducting kinetic (limit equilibrium) analysis or later probabilistic analysis for this particular slope section.

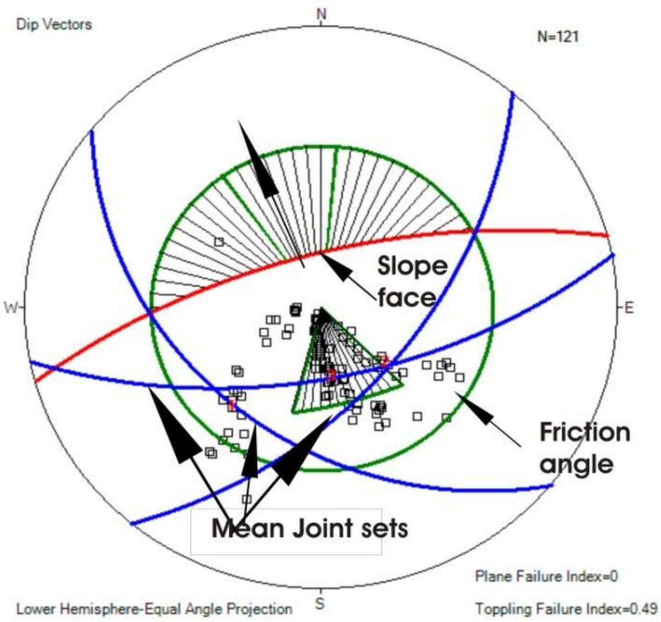


Fig 5.24 Stereographic analysis of wedge failure

The preferred orientation of joint sets were obtained from 121 Dip and dip direction observations and is presented in Table 5.3.

Table 5.3 Preferred orientation of joint sets used for wedge mode of failure

Joint set	Mean dip direction/ Mean dip
J1	219 / 41
J2	130 / 56
J3	169 / 62

5.5.3 Deterministic Factor of safety analysis of slope mass

The deterministic or physically based models are based on the physical laws of conservation of mass, energy and momentum (Van Westen and Terlien, 1996).

Limit equilibrium (LE) method is the most popular deterministic model used to calculate safety factor of slopes and same was used for analysis in present study. The LE procedure for calculating the factor of safety involves comparing the available shear strength along the sliding surface with the force required to maintain the slope in equilibrium. In this method the rock is assumed to be Mohr–Coulomb material in which the shear strength is expressed in terms of the cohesion '*C'* and friction angle '*φ'*, (Wyllie and Mah, 2004).

During the present study 'Factor of safety' was calculated for the slope having possible plane mode of failure (SL 3 Slope Section). The FOS was calculated for existing and anticipated condition separately. The conditions which were considered are; static dry, static saturated, dynamic dry and dynamic saturated.

For analysis the shear strength input parameters for analysis were obtained directly from standard table proposed by Hoek and Bray (1981) and from generalized Hoek and Brown(2002) classification made on Roclab software. Further, the slope geometry was deduced/ estimated in the field. All input data pertaining to slope geometry, shear strength parameters, unit weight and seismic parameter were fed to Rock PLANE which operate based on Sharma et al. (1995) Modified technique for Plane analysis owing to its upper slope angle.

For existing 'static dry' condition the assumption was made that there is no water pressure in the tension crack or along the sliding plane. Thus, both uplift water force (u) and horizontal water force (V) were taken equal to zero while computing FOS for 'static dry' condition.

The Factor of safety for static condition as per Sharma et al. (1995) is given by eq. 5.13

$$F = \frac{CA+(WCos\alpha p - U - Vsin\alpha p)tan\varphi}{Wsin\alpha p + Vcos\alpha p} \quad \text{………..eq.5.13}$$

Where;

'C' is the cohesion, 'A' is the base area of the sliding mass, 'W' is the weight of the sliding mass, 'U' is the uplift water force, 'V' is the water force acting on the rear face of the tension crack and 'φ' is the angle of friction.

The factor of safety for dynamic condition as per Sharma et al. (1995) is given eq. 5.14;

$$F = \frac{CA + (w(\mathrm{Cos}\,\Psi_p - \alpha\,\mathrm{Sin}\,\Psi_p) - U - V\,\mathrm{Sin}\,\Psi_p)\,\mathrm{Tan}\,\phi}{w(\mathrm{Sin}\,\Psi_p + \alpha\,Cos\,\Psi_p) + V\,\mathrm{Cos}\,\Psi_p} \quad \text{…….. eq.5.14}$$

Where 'α' is the horizontal earthquake acceleration.

The Meteorological data of the area indicated that the area receives an average annual rain fall of 1,535mm with peak value of 2,220mm. Thus, it may be considered that during rainy season the slope mass would be considerable saturated. For saturated condition analysis varied water saturation situations were considered as when the tension crack would be filled with water to its depth equal to 25%, 50%, 75%, and 100%.

Further, for dynamic condition horizontal seismic coefficient of 0.08g was considered.

The computed Factor of Safety for SL 3 slope section for various anticipated conditions along with input data used is presented in Table 5.4.

Table 5.4 Input values, Factor of safety of slope mass calculation and Probability of failure determination under different condition for SL 3 slope section having Plane Mode of Failure by ROCPLANE software.

Condition		H (m)	C Kpa	Φ Deg.	γ KN/m^3	αf (^{o})	Upper slope angle	γ of water KN/m^3	α (g)	Factor of safety	PF %
Static dry		13	587	46	23	67	20	0	0	1.86	0
Static saturated	A	13	440	34.5	24	67	20	10	0	1.25	0
	B	13	293.5	23	25	67	20	10	0	0.76	100
	C	13	147	12	26	67	20	10	0	0.38	100
	D	13	6	5	28	67	20	10	0	0.2	100
Dynamic dry		13	540	42	23	67	20	0	0.08	1.36	0
Dynamic saturated	A	13	405	39.5	24	67	20	10	0.08	1.23	0
	B	13	270	21	25	67	20	10	0.08	0.58	100
	C	13	135	11	26	67	20	10	0.08	0.3	100
	D	13	5.5	4.5	28	67	20	10	0.08	0.2	100

Note: A, B, C, D shows when water fills the tension crack 25%, 50%, 75% and 100% respectively
'H' – Height of slope, C – Cohesion, Φ- Angle of Friction, ϒ – Unit weight of rock, αf - slope inclination, α – horizontal acceleration PF – Probability of Failure

A perusal of results of Factor of Safety as presented in Table 5.5 clearly indicates that Slope Section SL 3 is a stable section at static dry and dynamic dry as well as at low percent of water filling tension crack as FOS is greater than 1.0.

Even it is safe for engineering design application such as road construction at static dry condition as it is having FOS much higher than 1.5. However, the slope is instable at anticipated adverse conditions.

5.5.4 Probabilistic Plane Failure Analysis of SL 3 Slope Section

Unlike factor of safety that shows stability of slopes in deterministic method, probability of failure shows safety of slope in probabilistic model. The probability of failure (PF) is defined as the probability that the factor of safety is less than or equal to 1.

$$Pf = P\{FOS \leq 1\} \qquad \text{……... eq.5.15}$$

However, factor of safety of greater than one doesn't indicate a hundred percent stability of slopes.

In the present analysis a probability of failure was realized to as 100% for static fully saturated condition and for dynamic fully saturated condition (Table 5.4). These results indicate slope section to be stable at existing condition however, during field visit it was observed that rock blocks have failed from this slope sections. Thus it may be concluded that the slope section as a whole is stable however isolated rock blocks may detach and slide under saturated conditions.

5.5.5 Deterministic Factor of Safety Analysis of Slide Block in SL3 Slope Section

Slope on the lower sandstone unit (SL 3) has shown manifestation of instability and failure despite the results of FOS (Table 5.4) obtained during present study and presented in previous section. The slope mass present in SL 3 slope section is basically controlled by through going, relatively closely spaced horizontal beddings generally dipping towards road and two joint sets which are nearly perpendicular to bedding joint, one parallel to slope face.

Attempt were made to perform stability analysis to understand at what slope angle, which condition and what slope height has controlled the mass failure, the slope mass was divided into block of variable height, condition and angle as shown in Table 5.5.

The shear strength input parameters and unit weight of the blocks was directly adopted from standard tables proposed by Hoek and Bray (1981) and Generalized Hoek and Brown criteria from Roclab software.

Table 5.5 presents the input values and factor of safety for slide blocks of SL 3 Slope section under different condition by Rock PLANE software.

For the analysis at block level Height of the slope was varied from 1m to 13m and potential failure plane was varied from 25^o to 40^o. This analysis was attempted to have an idea at what height and by what angle the blocks have failed and will fall in future.

Table 5.5 The input values and factor of safety for slide blocks of SL 3 Slope section under different condition by ROCPLANE software

Condition		H (m)	C Kpa	Φ Deg.	γ KN/m^3	αf (^{o})	γ_w KN/m^3	α (g)	Factor of safety for variable αp			
									25	30	35	40
Static dry		13	587	46	23	67	0	0	2.28	1.86	1.55	1.31
Static saturated	**A**	13	440	34.5	24	67	10	0	1.54	1.25	1.03	0.87
	B	13	293.5	23	25	67	10	0	0.97	0.76	0.64	0.54
	C	13	147	12	26	67	10	0	0.47	0.38	0.32	0.24
	D	13	6	5	28	67	10	0	0.28	0.22	0.15	0.1
Dynamic dry		13	540	42	23	67	0	0.08	1.58	1.36	1.12	0.98
Dynamic saturated	**A**	13	405	39.5	24	67	10	0.08	1.5	1.23	1.04	0.88
	B	13	270	21	25	67	10	0.08	0.7	0.58	0.41	0.32
	C	13	135	11	26	67	10	0.08	0.35	0.3	0.24	0.2
	D	13	5.5	4.5	28	67	10	0.08	0.25	0.2	0.18	0.12
Static dry		11	587	46	23	67	0	0	2.3	1.9	1.56	1.32
Static saturated	**A**	11	440	34.5	24	67	10	0	1.53	1.25	1.05	0.9
	B	11	293.5	23	25	67	10	0	0.95	0.77	0.65	0.55
	C	11	147	12	26	67	10	0	0.47	0.4	0.32	0.27
	D	11	6	5	28	67	10	0	0.2	0.2	0.18	0.15
Dynamic dry		11	540	42	23	67	0	0.08	1.65	1.4	1.2	1
Dynamic saturated	**A**	11	405	39.5	24	67	10	0.08	1.5	1.25	1.05	0.9
	B	11	270	21	25	67	10	0.08	0.7	0.6	0.48	0.3
	C	11	135	11	26	67	10	0.08	0.36	0.29	0.2	0.15
	D	11	5.5	4.5	28	67	10	0.08	0.15	0.11	0.10	0.1
Static dry		9	587	46	23	67	0	0	2.32	1.9	1.6	1.34
Static saturated	**A**	9	440	34.5	24	67	10	0	2	1.3	1.06	0.9
	B	9	293.5	23	25	67	10	0	1.8	0.8	0.7	0.56
	C	9	147	12	26	67	10	0	0.95	0.4	0.33	0.17
	D	9	6	5	28	67	10	0	0.2	0.18	0.15	0.11
Dynamic dry		9	540	42	23	67	0	0.08	1.65	1.4	1.2	1
Dynamic saturated	**A**	9	405	39.5	24	67	10	0.08	1.51	1.2	1.04	0.9
	B	9	270	21	25	67	10	0.08	0.71	0.65	0.55	0.5
	C	9	135	11	26	67	10	0.08	0.36	0.3	0.25	0.2
	D	9	5.5	4.5	28	67	10	0.08	0.15	0.15	0.12	0.1
Static dry		7	587	46	23	67	0	0	2.34	1.9	1.6	1.4
Static saturated	**A**	7	440	34.5	24	67	10	0	1.6	1.3	1.08	0.92
	B	7	293.5	23	25	67	10	0	1	0.8	0.7	0.48
	C	7	147	12	26	67	10	0	0.5	0.4	0.32	0.21
	D	7	6	5	28	67	10	0	0.2	0.15	0.13	0.1
Dynamic dry		7	540	42	23	67	0	0.08	1.7	1.41	1.2	1.03
Dynamic saturated	**A**	7	405	39.5	24	67	10	0.08	1.52	1.3	1.08	0.92
	B	7	270	21	25	67	10	0.08	0.72	0.66	0.56	0.41
	C	7	135	11	26	67	10	0.08	0.4	0.32	0.28	0.22
	D	7	5.5	4.5	28	67	10	0.08	0.2	0.18	0.15	0.1
Static dry		5	587	46	23	67	0	0	2.4	1.97	1.66	1.44
Static saturated	**A**	5	440	34.5	24	67	10	0	1.6	1.32	1.12	0.97
	B	5	293.5	23	25	67	10	0	1	0.82	0.69	0.6
	C	5	147	12	26	67	10	0	0.5	0.42	0.35	0.3
	D	5	6	5	28	67	10	0	0.2	0.16	0.13	0.1
Dynamic dry		5	540	42	23	67	0	0.08	1.73	1.45	1.25	1.08
Dynamic saturated	**A**	5	405	39.5	24	67	10	0.08	1.6	1.3	1.1	0.98
	B	5	270	21	25	67	10	0.08	0.75	0.6	0.54	0.4
	C	5	135	11	26	67	10	0.08	0.4	035	0.32	0.27
	D	5	5.5	4.5	28	67	10	0.08	0.2	0.16	0.14	0.12
Static dry		3	587	46	23	67	0	0	2.5	2.09	1.79	1.6
Static saturated	**A**	3	440	34.5	24	67	10	0	1.7	1.4	1.2	1.06
	B	3	293.5	23	25	67	10	0	1.05	0.9	0.76	0.67
	C	3	147	12	26	67	10	0	0.52	0.42	0.35	0.28
	D	3	6	5	28	67	10	0	0.2	0.18	0.14	0.13
Dynamic dry		3	540	42	23	67	0	0.08	1.8	1.45	1.27	1.14
Dynamic saturated	**A**	3	405	39.5	24	67	10	0.08	1.62	1.35	1.17	1.03
	B	3	270	21	25	67	10	0.08	0.9	0.67	0.59	0.53
	C	3	135	11	26	67	10	0.08	0.4	0.34	0.31	0.29
	D	3	5.5	4.5	28	67	10	0.08	0.21	0.19	0.16	0.13
Static dry		1	587	46	23	67	0	0	3.1	2.7	2.4	2.26
Static saturated	**A**	1	440	34.5	24	67	10	0	2.1	1.84	1.66	1.58
	B	1	293.5	23	25	67	10	0	1.4	1.2	1.07	1.02
	C	1	147	12	26	67	10	0	0.63	0.6	0.53	0.5
	D	1	6	5	28	67	10	0	0.5	0.46	0.42	0.32
Dynamic dry		1	540	42	23	67	0	0.08	2.3	2	1.86	1.78
Dynamic saturated	**A**	1	405	39.5	24	67	10	0.08	2	1.7	1.6	1.5
	B	1	270	21	25	67	10	0.08	1.09	0.9	0.85	0.82
	C	1	135	11	26	67	10	0.08	0.5	0.41	0.35	0.32

	D	1	5.5	4.5	28	67	10	0.08	0.23	0.21	0.19	0.17

Note: A, B, C, D shows when water fills the tension crack 25%, 50%, 75% and 100% respectively. 'H' – Height of slope, C – Cohesion, Φ- Angle of Friction, ϒ – Unit weight of rock, αf - slope inclination, α – horizontal acceleration Yellow marks show unstable slopes at all given sets of conditions and red mark shows unstable slopes for considered long term/permanent slope design.

Further, for each combination of slope height and potential failure plane inclination FOS was determined for static and dynamic conditions with varied water saturation conditions. The results thus obtained are presented in Table 5.5.

A perusal of Table 5.5 indicates FOS of individual blocks, which varies considerable. Many blocks become unstable under water saturation and dynamic conditions.

In Table 5.5 all those factor of safety which are shown in Yellow background indicates unstable condition from natural slope stability point of view whereas those shown with red background may become unstable for long term engineering design.

5.5.6 Probability Analysis of Slide Block in SL3 Slope Section

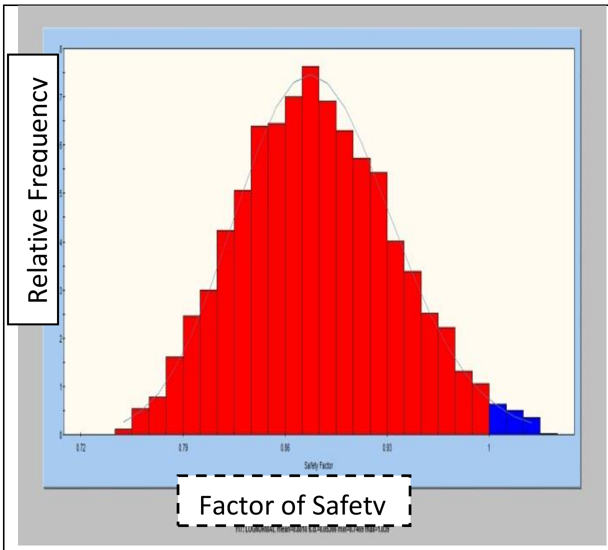
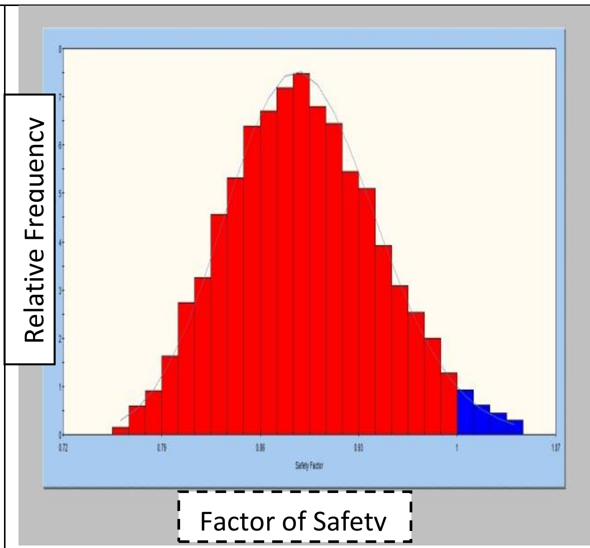
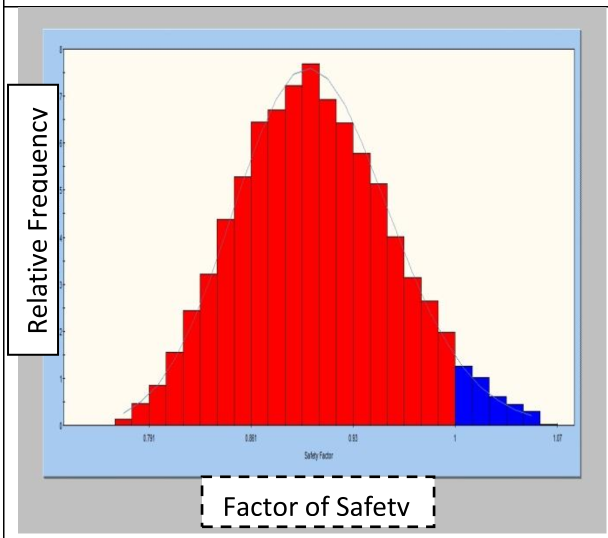
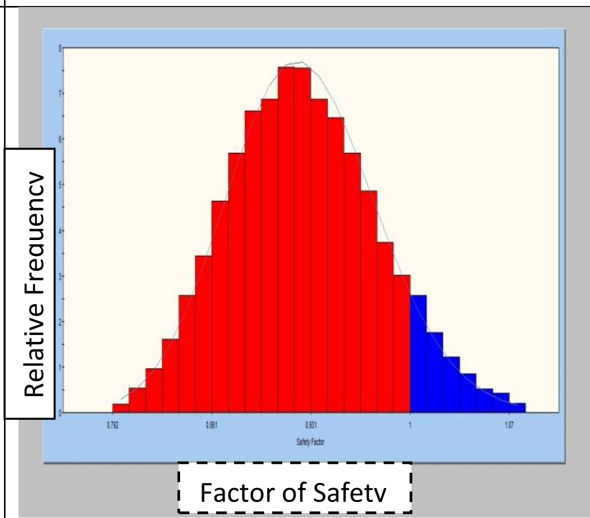
On the basis of the above deterministic calculation and by treating uncertain parameters under 10,000 Monte Carlo simulations the probability of failures of individual blocks was determined under the most favorable and the worst condition for variable slope height as shown in Table 5.6.

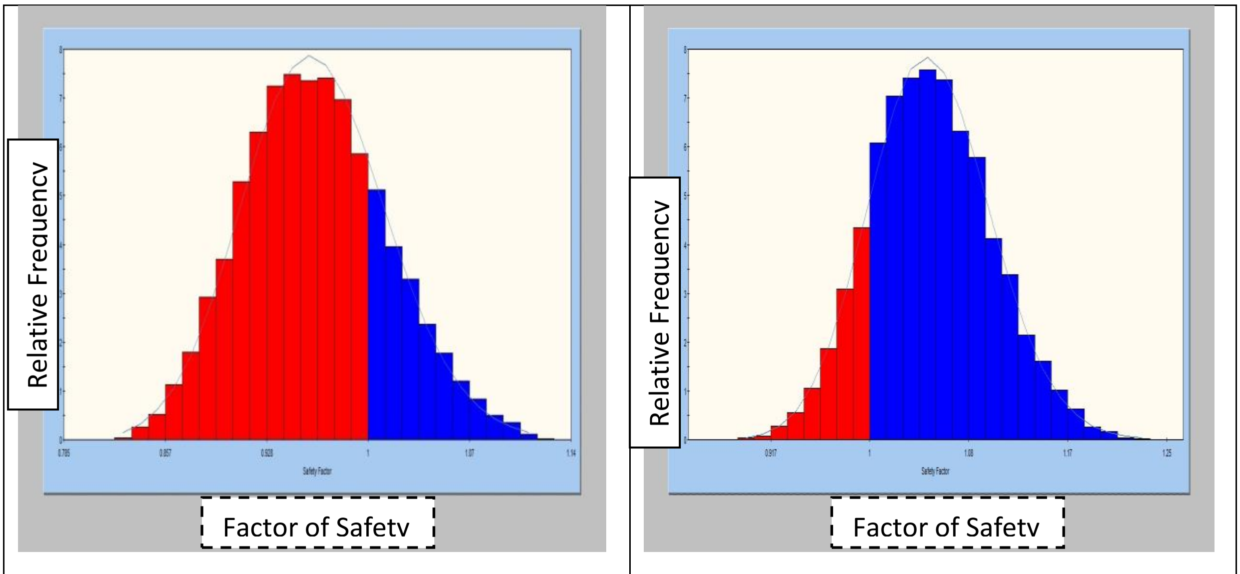
Table 5.6 Deterministic FOS for mean αp (30◦) and Probability of failure for Slide Block in SL3 Slope Section by ROCPLANE software

Parameters			**Deterministic FOS for mean αp**				**Probability of failure (%) For variable Slope Height**							
			Worst case U_{max} and a_{max}	**Best case U=0 and a =0**	**No water but earth quake**	**No earth quake but water**	**Cases**	**13**	**11**	**9**	**7**	**5**	**3**	**1**
Fixed parameters	**H**	**13**	0.2	1.86	1.36	0.22	Best case =	0	0	0	0	0	0	0
		11	0.11	1.91	1.4	0.2								
		9	0.13	1.93	1.41	0.18								
		7	0.18	1.95	1.3	0.15	Worst case =	100	100	100	100	100	100	100
		5	0..16	1.97	1.45	0.16								
		3	0.19	2.09	1.45	0.19								
		1	0.21	2.7	2	0.46								
	αf		67	67	67	67	U but no a =	100	100	100	100	100	100	100
	αp		30	30	30	30								
	Us angle		20	20	20	20	a but no U =	0	0	0	0	0	0	0
Ra. pa	**U**		100%	0	0	100%								
	a		0.08g	0g	0.08g	0g								

Where αf = slope angle, αp = failure plane angle, a= horizontal seismic coefficient, U = water pore pressure, H = height of the slide block, Ra. Pa= random parameter, Us angle= upper slope angle.

Table 5.6 presents the comparison between Deterministic FOS with mean failure plane inclination (αp) and Probability of failure for variable slope height blocks.

 **Height (H) = 13m**	 **Height (H) = 11m**
 **Height (H) = 9m**	 **Height (H) = 7m**



Height (H) = 5m **Height (H) = 3m**

Fig 5.25 Probability distributions of safety factor for variable height (H) around their mean value for slide blocks in Slope Section SL 3

A perusal of results presented in Table 5.6 indicates that Deterministic FOS with mean failure plane inclination (αp) for worst conditions (full water saturation and dynamic loading) indicates that the slope block of all height becomes unstable. The probability analysis also indicated similar type of results. These blocks under worst condition have probability of failure of almost 100%.

5.6 Sensitivity Analysis F for Rock Slope

From the probabilistic calculation results rock block presented in Table 5.6, sensitivity of input variables was made to understand the relative influence on Factor of Safety. The parameters for which senstivity analysis was made are; Slope angle, Slope height, Unit weight, Failure plane angle, Friction angle, Water saturation, upper slope angle and cohesion. Perusal of Fig. 5.26 clearly indicates the order of importance in terms of senstivity to FOS. In order of importance the most significant factors are; Slope angle, Cohesion and Water saturation. The other factors; Slope height, Unit weight, Failure plane angle, Friction angle etc. are less senstivity for FOS.

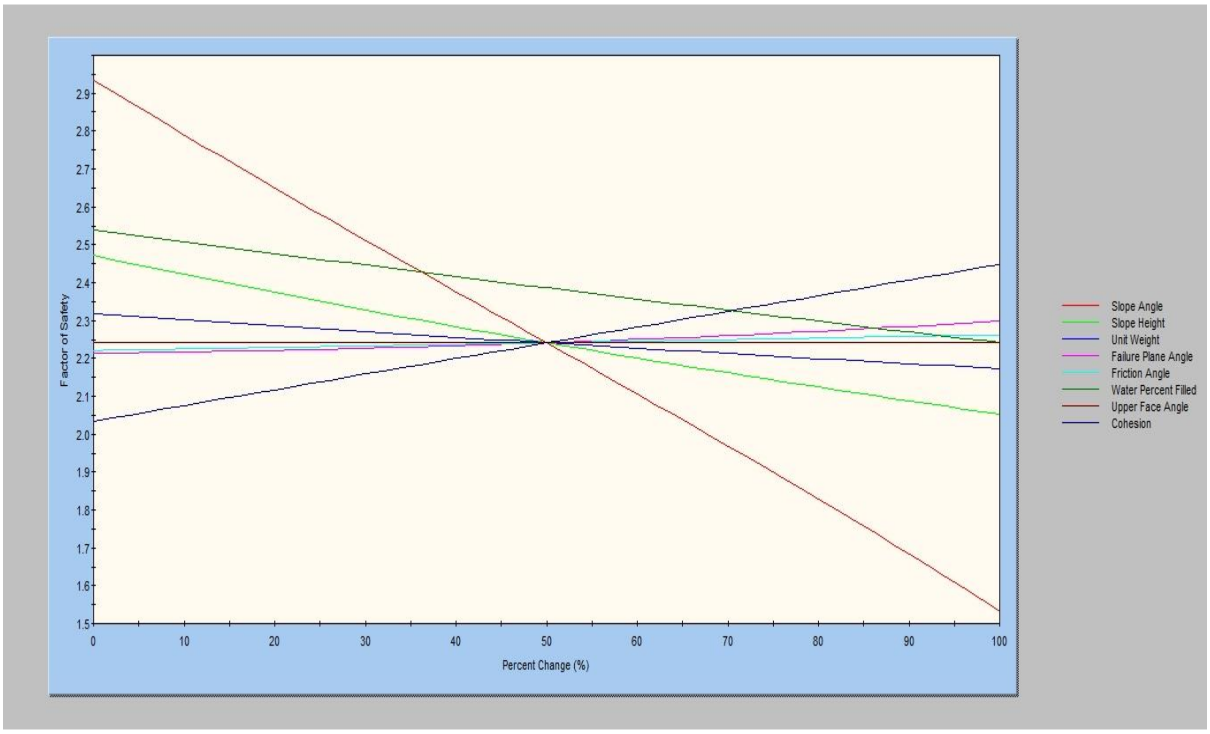


Fig 5.26 Sensitivity of safety factor for various input parameters for slide blocks in SL 3 Slope Section

5.7 Discussion and Interpretation of Results

In the present study analysis of colluvium slopes were made for the existing and anticipated possible worst conditions by utilizing SLIDE software that supports both deterministic and probabilistic approaches. Similarly, for rock slope having planar mode of failure ROCPLANE software that operate based on the principles of Modified Analytical Technique proposed by Sharma et al., (1995) and support both methods were also utilized for analysis.

Under static state the shear strength exceeds shear stress and the factor of safety is greater than 1.0, which indicates stable natural slope. For slopes during verge of movement, shear strength is just balanced by shear stress and the factor of safety is assumed to be 1.0 (Selby, 1993) which is at its critical state. However, from design point of view the minimum safety factor of slope along road side is 1.2 (Hoek and Bray, 1989). For short term or temporary stability factor of safety of greater than or equal to 1.3 and for long term or permanent stability safety factor of 1.5 or greater can be considered.

The stability analysis results of the colluvium slope section (SL 1) found towards the Gohatsion side of the gorge indicates that the factor of safety of the slope is 1.106 under static dry condition. Further, under static fully saturated condition the factor of safety has dropped from to 0.31 for the upper failure surface (SL 1) that passes below the road. From stability point of view this slope is irresistible for the anticipated adverse condition and shows signs of devastation. This extreme drop in FOS (0.31) also tells how water is significant in instability of the slope. This is because the slope material is permeable and it lets water pass through it and facilitate the pore water pressure development in the slope reducing slope material strength. Further, the analysis results for slope section SL 1 under dynamic dry condition indicates a FOS value of 0.989 and for dynamic saturated condition FOS reduces to 0.25.

Moreover, the stability of the entire slope is not represented by a unit stability value only. Hence, probabilistic analysis was conducted by taking deterministic input parameters and treating them as random variable by using 10,000 Monte Carlo simulations for better reliability of output. Accordingly, the probability of failure of the slope in static dry condition is 0.1% whereas; with water saturation (fully) the probability of failure is 100%. Under dynamic dry conditions the probability of failure is 64% and with water saturation (fully) the probability of failure is 100%. Thus, the results indicate that the water saturation under static or dynamic conditions will make slope unstable. Even, without water saturation under dynamic condition the probability of failure of this slope (SL 1) are high.

Further, the sensitivity analysis conducted for this slope (SL 1) indicates that Unit weight of Basalt (γ_b), Angle of friction of pyroclastic depository (ϕ_p) and Cohesion of Basalt (C_b) are the parameters which have significant effect on the stability of SL 1 slope. This is important indication as heavy basalt is resting on lighter pyroclastic deposit.

Slope section SL 4 slope which is found on the Dejen side of the gorge is characterized by colluvium type material of limestone, gypsum and basalt. For this slope section stability analysis was carried out along two slope sections; SL 4(1) and SL 4(2). The SL 4(2) is located towards the upper side of SL 4(1).

The stability analysis results for SL 4 (1) by deterministic method for static dry condition indicates that the FOS for 3 failure surfaces from top to inside have values 4.7, 1.9 and 1.6, respectively. The results reveal that firstly that it is a deep seated failure and secondly the progressive mode of failure is controlled by material characteristic and slope morphology. Further, under ‘static saturated’ condition FOS value for slip surface corresponding to existing roads and scarp face indicates a decrease in FOS from 1.6 (static dry) to 0.83. This indicates significant effect of water saturation which converts the most stable conditions to totally unstable condition.

The response of dynamic loading on SL 4(1) slope section in dynamic dry condition is not that much significant as the FOS for entire slope section only reduces from 1.6 (static dry) to 1.144. Though from natural slope stability point of view value of FOS of 1.144 is stable but from engineering design point of view it is unstable. Further, with water saturation and dynamic loading the FOS value comes out to be 0.5, which indicates complete instability of slope.

The probabilistic analysis for SL 4 (1) slope section for static dry condition indicates probability of failure for entire slope equal to 0% which validates the deterministic value of FOS equals to 1.6 and a stable condition. However, the probability of failure is 100% under saturated condition for both static and dynamic situations.

The sensitivity analysis for SL 4 (1) slope section indicated that the slope is highly sensitive to friction angle and unit weight of silt and shale. Factor of safety of the slope increased with increase in silt and shale friction angle and unit weight as well as minor increase of colluvium weight.

The deterministic stability analysis for SL 4 (2) slope section indicates a FOS of 0.709 under static dry condition. The result indicates that slope is unstable even under the best condition (static dry), therefore it is obvious that with water saturation and dynamic condition the slope will be unstable. Thus, no further analysis was made for the anticipated worst conditions.

The probabilistic analysis for SL 4 (2) slope section also indicated the similar results. The probability of failure under static dry condition comes out to be 99.4%. Thus, from the results it can safely be deduced that SL 4(2) slope section is a massive deep seated progressive failure. The instability was this slope can be attributed to nature of slope material, stratigraphical settings and topographical controlled structure. From stratigraphic point of view heavy colluvium material is resting on less dense but thinly bedded, shale and silts which facilitate down slope progressive failure.

From sensitivity analysis of SL 4(2) slope section it may be concluded that Angle of friction of Silt and Shale (ϕ_{ss}) and Unit weight of Colluvium (γ_C) are the parameters which have significant effect on the stability of SL 4 (1) slope. Parameters Angle of friction of Colluvium (ϕ_C), Cohesion of Silt and Shale (C_{ss}), Unit weight of Silt and Shale (γ_{ss}) are in significant for stability of slope.

Stability analysis of rock slope having planar mode of failure (SL 3) was carried out for entire slope section and separately at a slide block level. From the results it was observed that the slope mass is stable under static dry condition and instable under anticipated worst conditions.

The stability analysis carried out for SL 3 slope section by deterministic and probabilistic approaches has revealed that the slope is stable for existing conditions and would be instable under anticipated conditions. However, manifestation of instability of this slope section inspired to carry out analysis that observed during the field trip to have an idea how the blocks have failed in the past or how they will fail in future, under which condition blocks have failed and what combinations of slope height and failure plane inclination may result into block failures.

For the analysis at block level Height of slope was varied from 1m to 13m by a difference of 2m and potential failure plane was varied from 25^{o} to 40^{o}. Further, for each combination of slope height and potential failure plane inclination FOS was determined for static and dynamic conditions with varied water saturation conditions.

The results indicates that Deterministic FOS with mean failure plane inclination (αp) for worst conditions (full water saturation and dynamic loading) indicates that the slope block with greater height and greater failure plane inclination has more probability of failure and more becomes unstable.

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Chapter 6

Remedial Measures and Preventive Options

6.1 Preamble

As per slope stability analysis in the present study area most of the slope sections require long-term stability for anticipated adverse conditions. However, identification and selection of remedial measures for failed slope sections require an understanding of the mechanism that led to the original failure, as well as potential remedies that can be used to eliminate or reduce the chance of recurrence of the problem (Saleh and Wright, 1997).

There are many factors to cause failure of a slope and very often these factors are interrelated. When a slope failed and remedial works are required, it is essential to carry out failure investigation to find out the possible causes. Suitable remedial design can only be carried out after knowing the causes of failure. Once the main causes of slope failure have been identified, the remedial design can be carried out to correct the problems (Chen and Lim, 2005).

Further, Terzaghi (1950) stated that, “If a slope has started to move, the means for stopping movement must be adapted to the processes which started the slide”. A number of remedial measures exist for improving stability of soil and rock slopes. Some may be best suited as preventive measures and some as remedial measures still others may be used as both preventive and remedial measures. The remedial measure proposed for colluvium slopes may not be restrictive but can be used for rock slopes based on its necessity. Hence, the major causes of slope failures and the appropriate corrective measures for the potentially critical slopes in the present study area are discussed in the followed paragraphs;

6.2 Major Causes of Slope Failure in the Area

6.2.1 Geological Factors

It may be reasonably expected that the properties of the slope-forming materials, such as; strength and permeability that are involved in the failure, are related to the lithology, which therefore should affect the likelihood of failure. The slope forming materials of the area are mostly sedimentary rocks of different class caped by igneous rock on the top. The top basalt unit is mostly fractured through which water can easily infiltrate to the underlying sedimentary unit. Further, it is fragmented into blocks of various size intercalated with thin layers of loose silt and clay soils of unconsolidated materials those that have least shear strength and more prone to failure. Particularly, if they are charged with water, they show high potential to failure as seen from stability analysis results. The rocks underlying this weak unconsolidated material (colluvium) is mostly limestone (locally, upper limestone) in the area which is genetically carbonate and susceptible to solution when it is in contact with acidic water. Besides, this limestone occur with layers of silt and shale which are relatively weak and can easily erode when subjected to intense floods as in the case of summer season of the country. These induce instability and result into failure of different level based on the intensity of triggering factors.

The other geologic factor that plays a role in slope instability is structural discontinuities of primary and secondary origin in the rocks. Geological structures are other geological factors. There were no recognized small scale faults on the critical sections selected but as per Gani, et al (2008) the Mesozoic sedimentary section is dominated by NW and NE-trending normal faults where the lower part fracture of the stratigraphic section are NE and NW-trending while the upper part of the Mesozoic section shows a fracture pattern in which N-trend dominates. Hence, the rock slope failures of the area are mostly controlled by joints and beddings while soil slopes are controlled by lithology.

6.2.2 Geomorphic Factors

Geomorphic factors include slope shape, aspect and gradient. The distribution of slope categories is dependent on the geomorphological history of the area. The angle of slope of each unit is a reflection of a series of localized processes and controls, which have been imposed on the slope (Long, 2008).

The geomorphology of the area is so rugged that it is characterized by the above mentions landforms. The main reason is related to landform structure affecting largely the concentration or dispersion of surface and sub-surface water. For example Convergent hill slope tend to concentrate sub-surface water into small areas of the slope, thereby generating rapid pore water pressure increase during storms or periods of rainfall. If pore pressures develop in the hollow, the soil shear strength reduces to a critical level and a landslide can occur. Hence, hollows are susceptible sites for initiation of debris slide and debris flows (Hack and Goodlett, 1960; Dietrich and Dunne, 1978; Benda, 1990 as cited in Long, 2008).

Similarly, Slope aspect (direction) strongly affects hydrologic processes via evapotranspiration and thus affects weathering processes and vegetation and root development, especially in drier environments (Sidle and Ochiai, 2006).

Altitude or elevation is usually associated with landslides by virtue of other factors such as; slope gradient, lithology, weathering, precipitation, ground motion, soil thickness and land use.

The relative relief (gradient) refers to the local height of the slope between the ridge top and valley floor in a slope facet. It has also an important role in forming the size of the unstable wedges. The slope having higher relative relief may form unstable rock wedges of big size with more probability of failures (Dai and Lee, 2002).

6.2.3 Hydrologic Factors

The meteorological data in the area dictates as the area receives a high mean monthly rainfall in between June to September (summer season). There is no exact record of landslide history in the area. An interview made with local people, previous research and stability analysis results for saturated condition revealed that most of the slope failures commenced in the area during rainy season.

As per Long, (2008) hydrologic processes in the form of precipitation (spatial and temporal distribution of rainfall), water recharge into soil (and the potential for overland flow), lateral and vertical movement within the regolith, evapotranspiration and interception contribute to slope instability. Hence, water is commonly the primary factor triggering a landslide and slides often occur following intense rainfall, when storm water runoff saturates soils on steep slopes or when infiltration causes a rapid rise in groundwater levels.

Groundwater may rise as a result of heavy rains or a prolonged wet spell and this is why ground water table is close to the surface at most of the studied sites. As water table rises, some slopes become unstable due to increase in slope saturation thus, building up positive pore water pressures. This causes a decrease in the effective normal stress acting along the potential failure plane, which in turn diminishes the available shear strength to a point where equilibrium can no longer be sustained in the slope (Orense, 2004).

Moreover, UNIMIB (2012) stated two processes mainly control changes in slope hydrology that may lead to failure; (i) infiltration of water from the surface and (ii) rising of groundwater levels by impoundment of water in a reservoir. However, in case of sub-surface water, it is only the shallow water close to the surface that is important from the landslides point of view as they can sufficiently reduce strength of surface materials. The sub-surface water flowing at deeper levels which are not day-lighted on surface is not important to be considered in slope stability analysis. This conform the fact, as observed in the area where the ground water table is close to the surface also reported in secondary data (JICA, 2011).

6.2.4 Human Intervention

Slope instability may result directly or indirectly from the activities of people. Among them are improper or uncontrolled discharge from sanitation or drainage works and water pipes, typically associated with human settlements and roads, especially in rural areas, which increase water infiltration in the slope, and possibly soil erosion at points of concentrated surface discharge. Excavation at the base of or on slopes and/or placement of fill at the top or on slopes during road construction owing to its rugged topography, altering the geometry of and the stress distribution in the slope is also another influence. Quarrying and mining activities, altering the geometry of and the stress distribution in the slope is an important factor. Vibration from blasting and heavy construction equipment; besides causing transient stress changes which in extreme cases can trigger landslides by themselves, vibrations can disturb the microstructure of soils or rocks, induce fracturing in rock (UNIMIB, 2012).

In the present study area there is gypsum quarry that alter slope geometry of the slope and at the same time vibration from the excavators may contribute to rock fracture that indirectly weaken the shale gypsum alternating layer in the unit finally inducing slope instability.

There is relatively low agricultural practice in the area. However, in and around big colluviums slide towards Dejen side farming is being practiced. Farming related irrigation practice may likely alter the natural ground water conditions. In the present study area there was no clue about historical vegetation cover but removal of vegetation for change of land use due to agricultural practice can also facilitate slope instability, reducing evapotranspiration, allowing increased infiltration and increased surface erosion through loss of the shallow reinforcement provided by the root system (UNIMIB, 2012). Besides, heavy traffic movement which passes frequently on the road in the area may also gradually effect grain to grain contacts of the underlying soft material and thus, inducing instability as manifested on the road through distress.

6.3 Remedial Measures for Soil Slope/ Colluvium

Consideration was given to a wide range of remedial measures based on providing long term stability to the highway**.** Feasible remedial measures proposed will be as per the instability contributing factors but they are interrelated. There are many methods to increase the stability of a slope and to stabilize a failed slope. These methods may be adopted independently or in combination. In general, commonly adopted remedial measures can be grouped into three main categories: Geometrical method, Drainage method and Retaining method (Broms and Wong, 1985 as cited in Chen and Lim, 2005).

6.3.1 Geometrical Method

This is the simplest and mostly cost effective method and involves changing the slope geometry from a steep slope to a gentler slope to increase slope stability and hence these method reduces the driving force. The geometry of the slope can be modified through unloading/ excavation which involve:

i) Removal of the Head of the Slide

It involves removal of large quantities of material from the head of the slope to balance the failure (Abramson, 2002).

The quantities of material to be removed depend on the geology of the area but as a guide line one to two materials originally removed at the toe during construction should be excavated from the head as stated by (Eckel, 1958 as cited in Rudolph et al., 1974).

However, as a practical matter the slope of interest (SL 1) in the present study is a weak slope (colluvium) and it experiences two modes of failure. From the field observation it was noticed as there is slope failure (i.e. failure occurs along the surface that intersects the slope above the toe with road as a reference) as evidenced by failure of gabion protections at most of the places. Further, the analysis also shows that it has circular base failure (i.e. the failure surface passes below the toe of the cut slope that is below the road). Hence, removal of the head of the slide may help to reduce driving force in the slope. Relatively as such it is a small scale circular failure but care has to be taken that where to dump the excavated mass. This is because if it is dumped on the toe it will increase the load at the base and eventually it will contribute for future failure. The colluvium slope towards the Gohatsion side (SL 1), there is a village named "Shonkor" that is at the verge of failure and whenever excavation has to be done the mass has to be taken far away from that village and the critical areas.

ii) Benching the Slope

The purpose of benching is to transform the behavior of one high slope into several lower ones (Abramson, 2002).

For the big slide (SL 4) located on the Dejen side this is not feasible because the slope face is long and there is also relatively wide agricultural practice on that slope. However, the slope (SL 1) on the Gohatsion side is relatively short in length thus; applying this method would be feasible especially emphasizing slope failure along the road. In general, these benches have to be wide and as such it will reduce the subsequent maintenance costs and thereby offset increased construction costs. It can also be used to control erosion and to establish vegetation but it has to be provided with appropriate drainage systems.

6.3.2 Drainage Method

Drainage of surface and sub-surface water appears to be the most successful remedial measure for the correction of active landslide problems. One of the slope failure factors is saturation and pore water pressure building up in the sub-soil. If drainage system had been provided, the chances of building up pore water pressure and saturation of subsoil can be minimized (Chen and Lim, 2005).

In terms of hydrologic control of slope instability in the study area it can be stated that the area is characterized by shallow ground water table and the area receives high intensity of monthly rainfall during rainy season. Hence, providing appropriate drainage system is the best corrective measure for the slopes under study as it has been observed that surface and groundwater contributes significantly to instability. There are various drainage techniques which are available to control ground water levels and the surface drainage. These can be Surface drainage systems (mostly for preventive measures) and/or sub-surface drainage systems.

Horizontal Drainage

The purpose of the horizontal or sub-horizontal drain is to remove excess water from a hillside, cut slope, or embankment. A horizontal drainage system usually consists of 2-inch to 4-inch diameter steel pipes installed in the face of the slope. Although described as horizontal drains, the pipes usually vary in inclination from 2^{o} to 20^{o} above the horizontal. The pipe is usually perforated with 3/8-inch holes on approximately 3-inch centers (Rudolph J. et al., 1974).

Horizontal drains have been used in three ways to remove excess water from the slope. They may be used to divert water from its source, to lower the groundwater table in the slide area or in adjacent areas, or to drain a pervious or artesian stratum. However, they have been used only after site investigations indicate the presence of a high groundwater table, unfavorable seepage forces, or possible locations of pervious strata which conforms exactly to the site condition of the area. It have been successfully used on a wide variety of slope profiles and in soils of markedly different engineering characteristics and most applicable when used to drain water sources in deep seated slides (Baker, 1953 as cited in Rudolph J. et al., 1974).

As stated before in the present study area both critical slopes (SL 1 and SL 4) have irregular surface geometry where alternate bulging and sinking is observed at many places including at the toe of the slope. In such case horizontal drains may be installed around lower portions of the slope possibly below the presumed phreatic water surface. However, care must be taken to drain out water away from the slope face. Since, both the slides (SL 1 and SL 4) are active and possibly comprise large saturated mass particularly during rainy season therefore large diameter precast concrete pipes can be provided to collect and divert water away from the slope sections.

Vertical Drains and Well Systems

Well systems have been employed in slope stabilization to control adverse groundwater conditions in both cut and embankment sections. As landslide prevention or correction measure well systems are most commonly used in conjunction with horizontal drains to provide relief of hydrostatic pressure and gravity discharge of the subsurface water respectively (Schweizer and Wright, 1974).

When employed for slope stabilization, vertical drains have been used for three basic purposes as cited by Rudolph et al., (1974):

(i) To provide a drainage path between lenses or strata of water bearing material which are separated by impervious layers (Palmer, 1950; Parrott, 1955);

(ii) To relieve artesian conditions which may develop at or below the surface of rupture (Holm, 1969; Smith, 1964; Smith, 1969); and

(iii) To relieve excess hydrostatic pressures in slopes of saturated clay and therefore expedite conso1idation and increase the shear strength of the soil (Holm, 1969; Fe11enius, 1955).

Root (1958) states that vertical drainage wells are equally applicable as corrective or preventive measures. He also indicated that vertical drainage wells have been used with a greater degree of success than horizontal drains in the correction and prevention of slides. However, the statement that vertical drain have been more successful than horizontal drains as a stabilization procedure may be misleading, according to the literature, vertical drains are most often used in conjunction with horizontal drains to form an integrated drainage system, rather than as a remedy in themselves (Rudolph et al., 1974).

In the present study area, combination usages of drainage systems may be necessary on both the critical slope sections (SL1 and SL 4). This measure possibly will help to lower down the ground water table as much as possible which is relatively shallow to the surface. Such combined drainage measure may also help to keep the slope mass relatively dry and thus the stability of slope may eventually improve.

6.3.3 Retaining Structures

This method is generally more costly. However, due to its flexibility in a constrained site, it is always the most commonly adopted method. The principle of this method is to use a retaining structure to resist the downward forces of the soil mass. The wall should be installed deep enough so that the critical slip surface passes around it with an adequate factor of safety. Several retention walls are there and due to its difficulty to block ground water flow, it should be provided with appropriate drainage path.

Gravity retaining wall is trapezoidal in shape and require firm base for construction. It achieves resistance to base sliding and overturning moment by its own weight.

For the present case gravity retaining walls are not feasible for both critical slope sections (SL 1 and SL 4). In both the slope sections the thickness of colluviums mass is more and slip surface is deep seated. Thus, getting a firm foundation for the wall below the failure surface is practically not feasible. Also, the expected earth pressures due to thick colluviums material must be very large therefore the required wall needs to be thick and high, which will not be economic.

Tieback wall can be used when space constraints limit excavation for footings of conventional gravity wall. It carries the lateral earth pressure on the wall by tie system that transfers the imposed load to a zone behind the potential or existing slip plane where satisfactory resistance can be established.

In the present study area Gabion wall has been used to stabilize road side slides on both sides of the gorge critical sites. This method as a means of permanent remedial measure cannot be recommended for both critical sections (SL 1 and SL 4) as both the slides are active and gabion may not withstand high lateral pressure. However, it may be used as a temporary protection particularly along road side sections.

Retention net is also necessary for steep slopes susceptible to rock fall as in the case of the present study slope of interests on the gypsum unit. Retaining wall was provided in the past to arrest the rock fall however, rock fall has filled some of its portion. The situation is serous as in near future the retention wall be ineffective and the rock fall will have direct impact on road. Hence, retention wall to a desired height is required in this section along with retention net.

6.4 Remedial Measure for Rock Slopes.

For the present study two rock slopes (SL 2 and SL 3) were identified as potentially critical slopes. However, kinematic check revealed that only SL 3 slope section is potentially unstable for possible plane failure thus, further stability analysis was made for it. However, it is necessary to propose a corrective measure even for SL 2 slope section which did not satisfied the kinematic condition. This is because some rock fall and minor block toppling has been observed during the field work, particularly along the road side. There are several methods available by which hard rock slopes can be stabilized.

Removal of unstable rock is typically necessary for slope rehabilitation whether to ensure long term performance or simply for workers safety. The same is required in the present study area, particularly slopes formed by lower sandstone unit in which unstable rock blocks in upper reaches were observed. Also, where ever in this unit the bedding plane dips at more angle towards the valley sliding is anticipated, the same fact was also realized in slide block analysis for SL 3 slope section, presented in Chapter 5. Thus, it is required to remove all such loose blocks with care so that the safety of existing road which passes nearby is ensured.

Rock anchors are tensile units, fixed at one end, used to place large blocks in compression, and should be installed nearly perpendicular to the joint. The ordinary types consist of rods installed in drill holes either by driving and wedging, driving and expanding, or by grouting with mortar or resins. Bolt heads are then attached to the rod and torqued against a metal plate to impose the compressive force on the mass. Weathering of rock around the bolt head may cause a loss in tension; therefore, heads are usually protected with concrete or other means, or used in conjunction with concrete straps in high-risk conditions (Hunt, 2005).

Fully grouted rock bolts, provide a more permanent anchor than the ordinary anchor which is subject to loss in tension with time from several possible sources including corrosion from attack by aggressive water, anchorage slip or rock spalling around.

Regardless of rehabilitation methods chosen, additional catchment/ retention measures for falling rock should be provided. This is because most of the rock slopes in the present study area (like that of the slope in gypsum unit) contain small blocks of rock that may loosen up in future though at present the rock mass as a whole seems stable. Hence, to avoid any future rock falling or sliding particularly in slopes in gypsum unit catchments/ retention measures like engineered benches, ditches, wide shoulders, berms, steel barriers, net may be provided along with retention walls.

6.5 Preventive Options

The first step in stabilization correction should be to ensure that all surface runoff is prevented from entering the slide area. This is required to minimize the possibility of surface water percolating into a potentially weak or unstable area.

There are different types of surface drain used to control surface runoff.

(a) *Catch water or interceptor drains* In order to intercept and divert the water from the hill slope, catch water drains shall be located very carefully, after the topography of the ground is studied in detail. Catch water drains shall be lined and properly maintained and shall be given a gradient of 1 in 50 to 1 in 33 to avoid high water velocity and possible wash out. A number of inter-connecting lined catch water drains may need to be reconstructed on the slope to collect the surface run-off if the area of slide is large. Water from the catch water drains shall be diverted into a chute or a natural hill-side drain or diverted by sloping drains and lead into culverts at a lower level finally to be lead through chutes into the nearest natural water course (Bhavan et al., 1999).

(b) *Road side drains* are provided on the road side at the foot of the hill slope to drain out water from the road surface and the water from the portion of the hill slope below the catch water drains. Road side drains are constructed of dry rubble stone masonry with semi-circular saucer, rectangular, trapezoidal, angle drain and channel drain in sections. Angle or kerb and channel drains are suitable where road width available is restricted and in emergencies, it serves as an extra width and not easily damaged. The slope of the bed shall be 1:20 to 1:25 to allow water to flow at self-cleaning velocity. If the grades are rather steep, the side drains shall be stepped to break the velocity of water or provided with small dry rubble stone masonry check walls to provide falls to minimize bed scour. A shoulder of 0.3 m width may be provided between the edge of the drain and the hill slope. Generally, lined side drains shall be constructed. However, unlined side drains are sometimes provided on hard/stiffer strata (Bhavan et al., 1999).

In the present study area such road side drains are provided which are designed with sufficient capacity to collect and carry water down slope. These are rectangular in section and properly lined. However, during field visit it was observed that this drain was blocked at several locations due to slided debris into it. For effective drainage and slope protection from any surface flows it is required that these catch drains must be cleaned regularly.

(c) *Cross drains* shall be provided at intervals of 4 to 6 per km depending upon the nature of the terrain to prevent the road side drains from being overloaded and flooding the road surface. These shall be provided at every point of natural barrier and water crossing. The cross drainage structures are culverts, scuppers, causeways and minor or major bridges (Bhavan et al., 1999).

Shotcrete can also be used to protect the slope from rainfall infiltration and maintain the slope in its dry or partially dry state. Shotcrete may be effective for potentially unstable rock slopes not only to reduce surface water infiltration but also to retain loose blocks. However, techno-economic feasibility of this measure needs to be assessed over other measures before its implementation.

Finally, in the present study area as observed and deduced from the stability analysis of the colluvium slides (SL 1 and SL 4) water seems to be the most significant factor in inducing instability of the soil mass. Therefore, effort must be made to provide measures by which soil mass may be depressurize and on the same time surface drainage should also be provided to reduce the water infiltration. For depressurizing and surface drainage several methods were discussed in the previous paragraphs any of these measures in combination may be adopted. However, it may require further study to estimate the quantity of water to be depressurize or drain on surface, accordingly the selection of method and its design can be worked out.

In general, rock slope sections in the study area are stable for existing and anticipated adverse conditions. However, several slope sections manifested for loose block instability, particularly in gypsum and lower sandstone units. As a preventive measure removal of such rock blocks is required, if not for entire section at least for those sections where the road passes nearby. Further, to avoid any future rock falling or sliding particularly in slopes in gypsum unit catchments/ retention measures like engineered benches, ditches, wide shoulders, berms, steel barriers, net may be provided along with retention walls.

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Chapter 7

Conclusion and Recommendations

7.1 Conclusion

The present study was conducted in Abay Gorge, in between Gohatsion and Dejen town. The study area is located 185Kms north of Addis Ababa on the main road that connects Addis Ababa to Bahir Dar town.

The present study area, especially along the main road from Gohatsion to Dejen Towns experiences major landslide problems during the rainy season. These slope failures have resulted into loss of property and damage to other infrastructure present in the area. Such slope failures in the area also hampered the safe functioning of the road, which is the important link between the Addis Ababa, the capital city and the northern part of the country. Landslides in the area include deep-seated rotational slumps, massive translational slides, progressive creep movements and debris and mud flows. Rock falls also takes place largely as discernible block topples and wedge failures all along mountains, valley walls, and road cuts.

The main objective of the present study was to perform stability analysis of selected critical slope sections by combined deterministic and probabilistic methods under static and dynamic loading conditions. Deterministic analysis was carried out by Limit equilibrium approach, i.e., a limiting value that can be reached when the forces acting to cause failure are in balance with the forces acting to resist failure to find safety factor of the slope. Hence, resistance to failure is provided by the shear strength mobilized along the failure surface. For compatibility of the two approaches probabilistic analysis was carried out using deterministic parameters by treating them as random variable for determination of probability of failure and reliability index of slopes.

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

Based on the field manifestations of actual and potential instability, a total of four critical slope sections; two from failed colluvium section and two from rock slopes were identified for further analysis. Later, relevant data pertaining to various aspects related to geology, geomorphology, hydrogeology, climate etc. from both primary and secondary sources was collected or procured. The critical slope sections were treated by rigorous and simplified limit equilibrium methods like; Janbu simplified, Bishop, Janbu corrected, GLE and Spencer for safety factor determination under existing and anticipated adverse conditions. Later each deterministic input parameter were treated as random values and quantified by Monte Carlo simulation for further probabilistic analysis. Hence, the geotechnical input parameters has been changed over wide range which is determined by statistical parameters (mean, relative minimum, relative maximum and standard deviation) to calculate a range of safety factors represented on histogram and to determine probability of failure under SLIDE and rock PLANE software. The two rock slope sections were subjected to kinematic check. Only one rock slope section satisfied the kinematic conditions for possible plane mode of failure. The plane failure analysis was carried out by ROCPLANE that function on the basis of Modified analytical approach proposed by Sharma et al. (1995).

From the results obtained from deterministic and probabilistic analysis the following conclusions are drawn;

- The slope section SL 1 is found to be in critical state under deterministic static dry condition with FOS of 1.01 and this value further drop to 0.31 under static saturated condition. The value drops even further under dynamic saturated condition. These results indicate that slope section SL 1 would be unstable under anticipated adverse conditions. From probabilistic analysis it is deduced that probability of failure of SL 1 slope in static dry condition is 37.5% whereas; with water saturation (fully) the probability of failure is 100%. Under dynamic dry conditions the probability of failure is 64% and with water saturation (fully) the probability of failure is 100%. Thus, the results indicate that the water saturation under static or dynamic conditions will make slope unstable. Even, without water saturation under dynamic condition the probability of failure of this slope is high.

- Similarly, stability analysis was carried out for other colluvium slope section (Dejen side) SL 4 by deterministic and probabilistic approach. For better representation of slope stability condition during existing and anticipated worst conditions this analysis was made along two critical slope sections SL 4 (1) and SL 4 (2).

The slide is progressive therefore stability analysis by deterministic approach was attempted along three possible slip surfaces. The stability analysis results for SL 4 (1) by deterministic method for static dry condition indicates that the FOS for 3 failure surfaces from top to inside have values 4.7, 1.9 and 1.6, respectively. Further, under 'static saturated' condition FOS value for slip surface corresponding to existing roads and scarp face indicates a decrease in FOS from 1.9 (static dry) to 0.83. This indicates significant effect of water saturation which converts the most stable conditions in to totally unstable condition. The response of dynamic loading on SL 4(1) slope section in dry condition is not that much significant as the FOS for entire slope section only reduces from 1.6 (static dry) to 1.1. Further, with water saturation and dynamic loading the FOS value comes out to be 0.513, which indicates complete instability of slope. The probabilistic analysis for SL 4 (1) slope section for static dry condition indicates probability of failure for entire slope equal to 0% which validates the deterministic value of FOS equals to 1.6 and a stable condition. However, the probability of failure is 100% under saturated condition for both static and dynamic situations. The sensitivity analysis for SL 4 (1) slope section indicated that the slope is highly sensitive to friction angle and unit weight of silt and shale.

- Further, the deterministic stability analysis for SL 4 (2) slope section indicates a FOS of 0.709 under static dry condition. The result indicates that slope is unstable even under the best condition (static dry), therefore it is obvious that with water saturation and dynamic condition the slope will be unstable. Thus, no further analysis was made for the anticipated worst conditions. The probabilistic analysis for SL 4 (2) slope section also indicated the similar results. The probability of failure under static dry condition comes out to be 99.4%. Thus, from the results it can safely be deduced that SL 4(2) slope section is a massive deep seated progressive failure. From sensitivity analysis of SL 4(2) slope section it may be concluded that Angle of friction of Silt and Shale (ϕ_{ss}) and Unit weight of Colluvium (γ_C) are the parameters which have significant effect on the stability of SL 4 (1) slope.

- Stability analysis of rock slope having planar mode of failure (SL 3) was carried out for entire slope section and separately at a slide block level. From the results it was observed that the slope mass is stable under static dry condition and almost instable under anticipated worst conditions.

The results of slide block analysis indicates that Deterministic FOS with mean failure plane inclination for worst conditions (full water saturation and dynamic loading) for block with higher height the more becomes they are unstable.

Finally, the main slope instability causative factors that were identified to be significant in the present study area are; geologic factor (lithology and joint set discontinuity), geomorphology, hydrology/hydrogeology, and human interventions. In general, lithologically stiff and competent layers overlay the weak units which eventually induce instability of slopes over the time. In rock slopes stability is mainly controlled by characteristics of structural discontinuity. The variation in lithology has shaped the geomorphology of the area in to different slope shape, direction and gradient that probably made the area vulnerable to slope failures. The area receives high mean monthly rainfall, particularly during rainy season which contributes to surface water increment and significant recharge to ground water and soil saturation thus it increasing slope instable. Further, human interaction through unplanned cultivation and irrigation practices, mining activity and frequent alteration of slope geometry for road widening and maintenance also contributes substantially for slope instability in the study area.

7.2 Recommendations

In general, based on the results of the present study following recommendations are forwarded;

- In terms of hydrologic control of slope instability particularly for colluviums active and potential slope sections (SL 1 and SL 4) in the study area it can be stated that the area is characterized by shallow ground water table and the area receives high intensity of monthly rainfall during rainy season. Hence, providing appropriate drainage system is the best corrective measure for the slopes under study as it has been observed that surface and groundwater contributes significantly to instability. Effort must be made to provide measures by which soil mass may be depressurize and on the same time surface drainage should also be provided to reduce the water infiltration. For depressurizing and surface drainage measures in combination may be adopted. However, it may require further study to estimate the quantity of water to be depressurize or drain on surface, accordingly the selection of method and its design can be worked out.

- Rock slope sections in the study area are stable for existing and anticipated adverse conditions. However, several slope sections manifested for loose block instability, particularly in gypsum and lower sandstone units.

  As a preventive measure removal of such rock blocks is required, if not for entire section at least for those sections where the road passes nearby. Further, to avoid any future rock falling or sliding particularly in slopes in gypsum unit catchments/ retention measures like engineered benches, ditches, wide shoulders, berms, steel barriers, net may be provided along with retention walls.

- In selected sections shotcrete may be effective for potentially unstable rock slopes not only to reduce surface water infiltration but also to retain loose rock blocks. However, techno-economic feasibility of this measure needs to be assessed over other measures before its implementation.

- For slope sections in lower sandstone unit (SL 3 and nearby sections) most of the potential rock blocks are wide and it is appropriate to use rock anchor as a remedial measure together with other rock block retention measures particularly in reaches where road crosses nearby.
- With current knowledge obtained from present slope stability analysis for anticipated adverse conditions, a warning system can be implemented along with a proper disaster management in the area. This may be helpful to alert the public according to the observed and predicted probability of slope failure.

- Shonkor village is located in the close vicinity of colluvium slide (SL 1) towards Gohatsion side of the valley. An early warning and rehabilitation of village is required to avoid any future damage and adverse effect on human life.

- Further study to determine the extent and size of mass movements in active landslides of the area are needed.

- The results from the present study in general indicates that the critical colluvium slope sections would be unstable during anticipated adverse conditions and likely loss to property and life would be unavoidable during the event of failure. The general characteristics of these slides with respect to the mass involved in large quantities, deep seated failure plane, groundwater conditions and progressive failure mode made it difficult to suggest and implement any techno-economic feasible remedial measures. Therefore, avoidance of these slope sections for any developmental activities would be more economic and safer option.

In the present study all efforts were made to come up with reliable results by following a combined deterministic and probabilistic slope stability analysis methods. A comprehensive methodology was followed during the present study which was based on the actual field data collection, procurement of data from secondary sources, exhaustive analysis through standard graphical, analytical and empirical approaches, logical interpretations of results and actual result based evaluation of feasible recommendations. However, this comprehensive methodology was applied to selected slopes only which showed manifestations of actual or potential instability. Thus, due to constrain of time, resources and financial limitations only 4 most critical slope sections were selected and studied during the present study. Thus, there is a need to extend this methodology to cover remaining slopes also in future studies so that entire area can be thoroughly studied for all kinds of slopes and based on the anticipated slope stability conditions appropriate engineering design for slopes can be worked out.

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