BE CIVIL ENGINEERING

PROJECT REPORT

ROCK SLOPE STABILITY ANALYSIS ALONG SWAT MOTORWAY AT RD-37 SITE

Project submitted in partial fulfillment of the requirements for the degree of

BE Civil Engineering

SUBMITTED BY

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MILTARY COLLEGE OF ENGINEERING

NATIONAL UNIVERSITY OF SCIENCE AND TECHNOLOGY, ISLAMABAD, PAKISTAN

2019

This to certify that the

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Has been accepted towards the partial fulfillment of the requirements for

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At the outset, we would like to thank Allah Almighty for giving us the strength to complete the research work assigned to us within the prescribed time and with a profound sense of satisfaction. We are in great debt to those who made this research possible especially to our advisor Dr Mazhar Iqbal and our Co Advisor Dr Sarfaraz Ali who always encouraged us and under whose able guidance we were able to complete the magnanimous task at hand with great ease. They have always been symbols of great inspiration and encouragement for us. We would also like to mention Sir Sohail (Geologist) and lab Engr Ajab Noor who took out time to help us during our research work. At the last, we would like to extend our special thanks to our parents for their prayers and boundless love.

DEDICATION

Dedicated to our beloved Parents and Teachers, whose prayers and guidance have been a source of motivation for us

Abstract

Swat is an increasingly popular tourist destination but not easily accessible to general public due to its rugged and mountainous terrain. Swat Motorway Project was launched to mitigate these issues. Current geometrical design of the road exposes the motorway to geotechnical distress, slope failure and rock fall. On and off landslide and rock fall takes place blocking the motorway thus hindering the economic and daily life of region. The research was undertaken to characterize the material of slope, study the underlying factors and evaluate the factor of safety. This research involved geotechnical investigation encompassing field and laboratory testing. The research aims to present a solution basing on the field observation, literature review and analysis carried out using SLIDE software.

Contents

CHAPTER - 1
INTRODUCTION

1.1 Introduction

In this writeup/research analysis of slope and slope stabilization techniques that have been used at RD-37 located on Swat Motorway which has been opened recently for public use by KPK Highway Authority in collaboration with Frontier Works Organization are discussed. In this project we will discuss stabilization as well as protection measures that are applied on the respective parts of the slopes that were analyzed. Various field tests were conducted and data was collected which was assessed using softwares related to Rock Mechanics and Slope stabilization.

Figure 1-1 Geological map of Pakistan

1.2 Background/General

Swat is considered as one of the most popular tourist destinations of Pakistan. Initially Swat was not easily accessible to general public due to its rugged and mountainous terrain. With the increase in the traffic and influx of activities there was a need for the enhancement of the communications network for this area. As the railway line is not available in that area the only possible method is by road. Considering these needs, Government of Pakistan initiated the Swat Motorway project (from Karnal Sher Khan Interchange, Swabi to Chakdara, Swat). Project was taken over by KPK Highways Authority with the assistance of Frontier Works Organization. Slope stabilization and pavement analysis was done by a team of Military College of Engineering. With completion of the Motorway, travel time from Nowshera to Chakdara will be reduced. The Nowshera-Swat Motorway is being constructed by the frontier works organizations.

The Swat Motorway should not be confused with the E90 Expressway (Besham–Khwazakhela Expressway) proposed by the National Highway Authority, which also terminates in Swat.

1.3 Problem statement

Swat Motorway passes through mountainous and rugged terrain which exposes it to geotechnical distresses, slope failure, and rock fall which could damage the road and result in loss of life. Due to its economic value it becomes a matter of public importance to ensure uninterrupted flow of traffic across the Motorway. A need is felt to carryout research to evaluate site specific causal factors and analyze the rock slope stability at RD 37 to fulfill serviceability and design requirements.

1.4 Scope and Objectives

To analyze the rock cut slope stabilization and propose various protection and stabilization measures for a specific site at Swat Motorway (RD 37). To achieve optimum protection from Rock fall and Landslides, ensuring maximum strength of protection barriers within aesthetic and economic constraints. The objectives of project are:

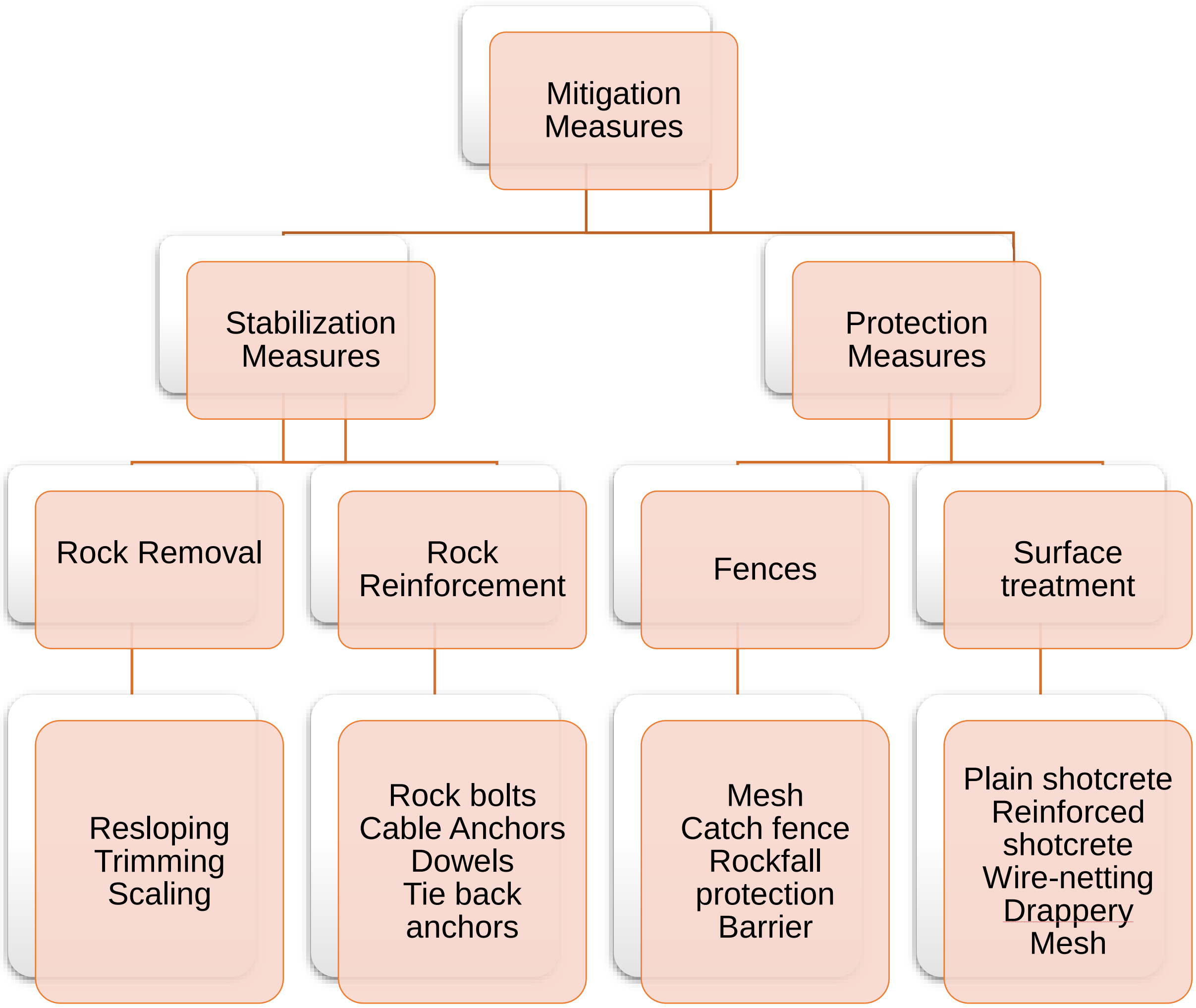
- To characterize the geological and geotechnical conditions of strata at RD 37 site.
- To perform geological mapping for detailed investigation to integrate it into a numerical model.
- To perform kinematic analysis using stereoenets.
- To analyze various hazards that are to be encountered, accurately and to provide effective and economical solution for slope stabilization.

1.5 Research Questions

The research will help us understand the geotechnical behavior of rock cut slope at RD 37 and will help us finding remedial solution and mitigation measures for the slope stability problem. It also answers the following questions:

- What is the geological profile of the rock cut at RD-37 (Swat Motorway)?
- How the discontinuities effect the stability of rock?
- What measures we must take for effective protection from rockfall and landslides maintaining serviceability of Swat Motorway?
- What is the most effective and economical solution for slope stability problem (stabilization and protection measures)?

1.6 Methodology/Conceptual Framework



- Site visit and field investigation.
- Literature review.
- Site characterization.
- Geological mapping and scanline survey.
- Lab investigation.
- Evaluation of geotechnical parameters for analysis.
- Numerical modelling.
- Analysis of rock slope stability and FOS.
- Evaluation of Protection and stabilization measure

1.7 Research Outcomes

- The Geomechanical stability analysis of rock cut slope at Swat motorway
- Rock fall mitigation suggested, fulfilling serviceably requirements
- Long lasting remedial solution proposed and the expenditure on road maintenance is minimized

1.8 Significance of Swat Motorway

Swat Motorway is 81 km long, starts from city Swabi (KPK) at the Kernal Sher Khan interchange and is located on the Peshawar-Islamabad Motorway. The motorway ends at Chakdara in the district of Lower Dir (KPK). It comprises of six interchanges and two tunnels each of 1.5 km length.

Swat is a city located in the North western province of Khyber Pakhtunkhwa popularly known as the Switzerland of the East. The city was hit by terrorism in the last decade and went into the control of terrorists. A successful military operation was launched to take the control of the city from terrorists in 2008. The local economy was annihilated because of its dependence on tourism which drastically reduced because of the prevalent security situation. Tourism gradually increased in the following years as people started visiting the area because of improvement in the security environment. Presently, the Swat valley is peaceful and people from all over Pakistan and the world have started to throng the place generating much needed revenue for the local population and the country.

There are problems of lack of quality infrastructure specially road network that still deters people from visiting the place. In addition to this, the adjacent valleys to Swat still remain unexplored by the tourists because of lack of connectivity to these areas. This presents an opportunity to the government to generate revenue.

Keeping in view the need to provide quality road network to the area, government of KPK decided to construct swat motorway which was initially perceived as an expressway but was then upgraded as motorway. Swat motorway is not only going to provide a fresh impetus to tourism in the region but also give an upward boost to the fruit, agriculture, horticulture and woolen sectors of the area. It will also create a new services market in the Malakand division.

The KP Govt has planned to construct CPEC City at Nowshera to promote industries, business and housing sector in the province. Tourism would be immensely promoted in Malakand division after construction of Swat Motorway and fruits, horticulture, agriculture and woolen sectors would get upward boost.

In addition to this, Swat motorway is also of strategic value to the country as it would serve as entry point towards Western corridor of CPEC after its completion. The construction of Rashakai Economic zone further increases the importance of swat motorway for the Country.

1.8 Thesis Structure

Thesis work will comprise of six chapters as under.

- Chapter – 1. Introduction.
- Chapter – 2. Literature Review.
- Chapter – 3. Research Methodology.
- Chapter – 4. Site Characterization.
- Chapter – 5. Slope stability analysis and remedial measures.
- Chapter – 6. Conclusions.

1.9 Conclusion

This chapter describes the need for adaptation of geostructural slope stabilization technique under mega geo-economic CPEC project. Problem statement, scope of research work, desired objectives, research question and research methodology are crafted in this chapter. Significance of research work and expected outcomes are also chalked out.

CHAPTER – 2
LITERATURE REVIEW

2.1 General

The continuous geological changes occurring on the surface of the earth are resulting in the creation of new rock, topographical structures and distribution of land, oceans, mountains and plains. Rock and soil slopes either exist in natural conditions or are man-made. Slope stability issues occur when natural slopes are disturbed by force. The scope is very high for slope problems, investigation, analysis tools and procedures of stabilization in projects of construction. Thoughtful studies of hydrology, seismicity, geology and rock properties is an important factor applied to the slope stability analysis.

2.2 Site Characterization

2.2.1 Geological Map Study

Geological maps are best source of information about rocks. Rock history like type, age, surface conditions and structure information from a three directional point of view can be obtained from the map. Field mapping is the method to understand and establish the confined geology and geological structure, correct understanding of these is necessary for design and quarry of rock slopes. Existing slopes and cuts and surface discontinuities and similarities, in which the digging will be done is obtained from the in sequence of occurring, slope design, for spot environment. Even as gathering data and projecting and displaying maps is a fundamental fraction of examination agenda, it is a vague procedure given that a confident sum of decision is there to broaden the information from surface outcrops to complete cut slope.

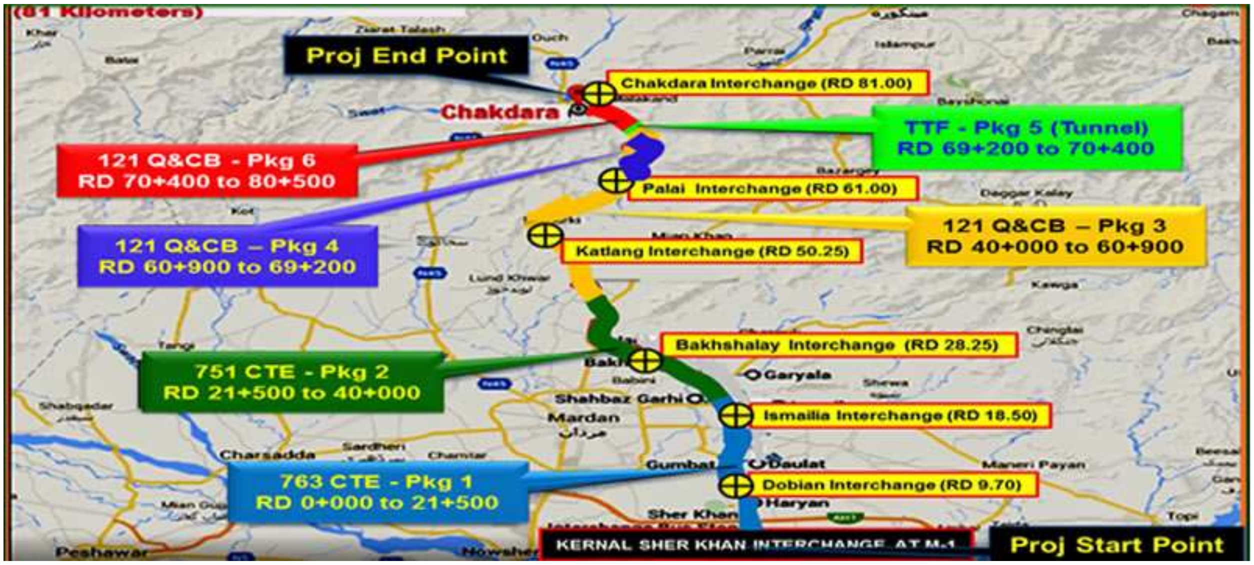
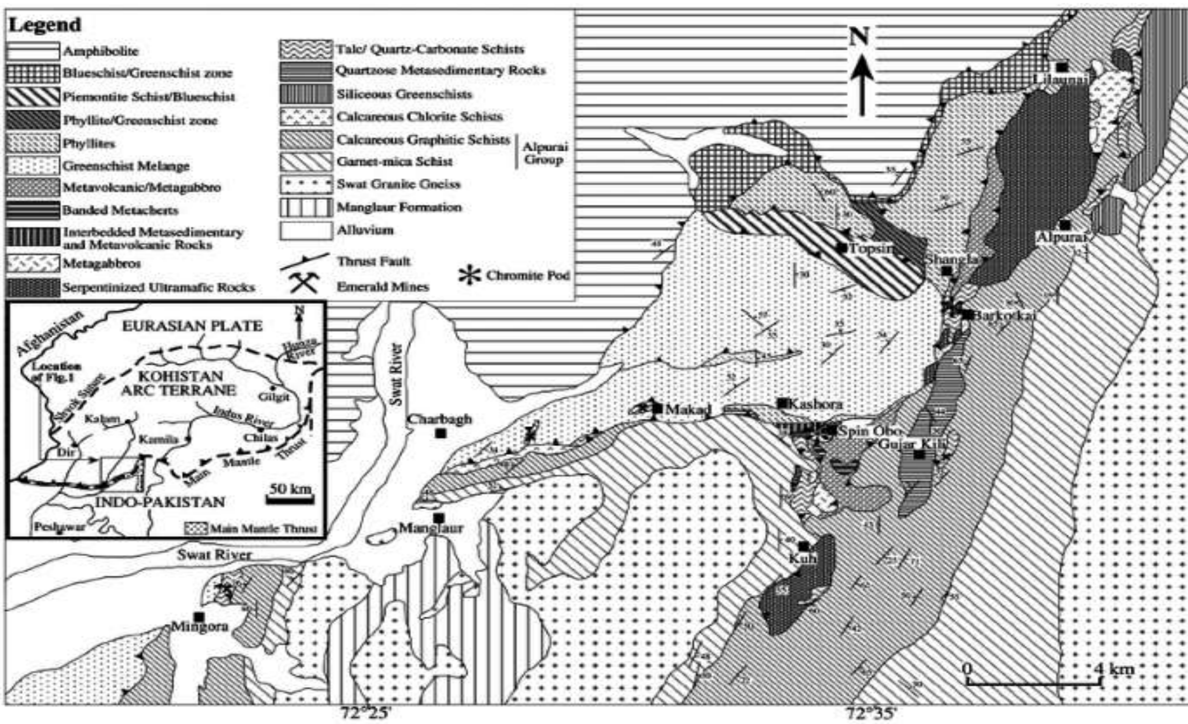


Figure 2-1 Arial view of Swat motorway



Geological map of the Lilaunai–Mingora area, Swat, north-western Pakistan (after Kazmi et al., 1984, 1986). The inset map shows general location of the study area.

Figure 2-2 Topographical map

2.2.2 Geological Setting

Many engineering projects are based on large rock strata. Rock slopes could be cut on sides of road, a tunnel may be dug through a mountain range or even a foundation may rest on rock mass. Mining also deals with rock stata/mass.

2.2.3 Aims of geological studying

The general thought of geological map-making is to group an array or arrays of discontinuities, or single property, for example, a fault, which will control dependability on a specific slope. For instance, the bedding may plunge out of the face and structure a plane disappointment, or a couple of joint sets may converge to frame a progression of wedges. Usually the discontinuities will happen in three ortho-gonalsets (commonly at right angles), with perhaps one extra set. At max four sets can be suited in an incline plan and if there are sure different sets there, they are probably to represent scatter in the direction of the sets.

2.2.4 Site Characterization

In site characterization, an understanding is developed of the hydrologic, geologic and engineering properties at site that include rock, soil, ground water and the conditions in subsurface that are man-modified that can impact conditions at site. It includes the temporal and spatial assessment of the contaminants in case they are present.

2.2.5 Site Reconnaissance

Site reconnaissance is one of the basic elements in geological investigation which assists in identifying, in observing, recording, evaluating the ground conditions by which the stability of slope is to be effected. It also acts as a support to the data which was collected by geological study. There are various causal factors of a failure but attempting to narrow down the things and making assumptions about the reason of failure is not technically correct gesture. In most of the cases, the deciding factor is the one that has been weak in the past, about to fail and by which the body is set in motion. Site reconnaissance is started by demarcating zones, where geological properties of all the elements are same in one section with respect to the necessities of on going project. Around the pit, stability conditions could differ from each other around the pit or with alignment and hence a comprehensive study of the area could be carried out where there are possible dangers of instability.

2.2.6 Preliminary routing

On the off chance that the venture includes the assessment of elective courses, constrained examinations could be completed of each course involving outcrop mapping, geophysics to discover overburden thickness and record trial of shake properties. For an unlock pit mine, there will for the most part be extensive land data on the property produced amid the investigation series. This motivation will be frequently incorporate mapping, earth physics and boring from which geotechnical information can be gotten. It is gainful to the structure of the pit slants if geotechnical information can be gathered as a major aspect of the investigation program.

2.2.7 Rock Exploration

2.2.7.1 Objectives of Rock Exploration

Rock exploration is necessary to be able to characterize and understand the geological and the sub-strata conditions through the surveys conducted. To obtain sufficient data for site characterization at different stages of project:

- Uniform zones and their degrees.
- Engineering attributes and kind of rock mass
- Geomechanical importance and geological separations
- The strata and earth crust conditions specifically of joints and cracks
- The intensity and disturbance between the joint structures, and amount of jointing upto which it could be done
- Hydro circumstances

2.2.7.2 Methods of Rock Exploration

Field exploration strategies involve excavation techniques by which data about the site geologic conditions could be obtained. Based on size of excavation done, there are two techniques:

Borings (Core Borings)

During boring operations soil samples are taken for on-site field identification and testing as well as future laboratory testing. Core holes are vertical but inclined borings are also done. Core diameters may range from 22 mm to 100 mm and length depends on core barrel.

Large excavations (Large Diameter Calyx Holes)

This is a costly method but gives true picture of rock strata inside the hole. Rock can be examined in situ using this technique. With the help of camera, photographs are taken inside the hole for various subsurface strata and inference is drawn with these photographs.

2.2.8 In-situ Field Tests

Shear Test

Shear strength is a crucial issue for the potential surface failure which can include one continuity plane or a posh path following many discontinuities and involving some fractures of intact rock material.

Accept assortment of sees tests were scratch from a piece of shake containing a smooth, two-dimensional detachment. Likewise, the partition contains an established fillable stuff indicated a tractable power would got the chance to be connected to the 2 parts of the specimen in order to isolate these. Each specimen is exposed to a power at right edges to the partition surface (typical pressure, σ), and a power is connected inside the heading parallel to the detachment (shear pressure, τ) while the shear dislodging (δs) is estimated.

For a test completed at a consistent ordinary pressure, a run of the mill plot of the shear stress against the shear relocation is appeared.

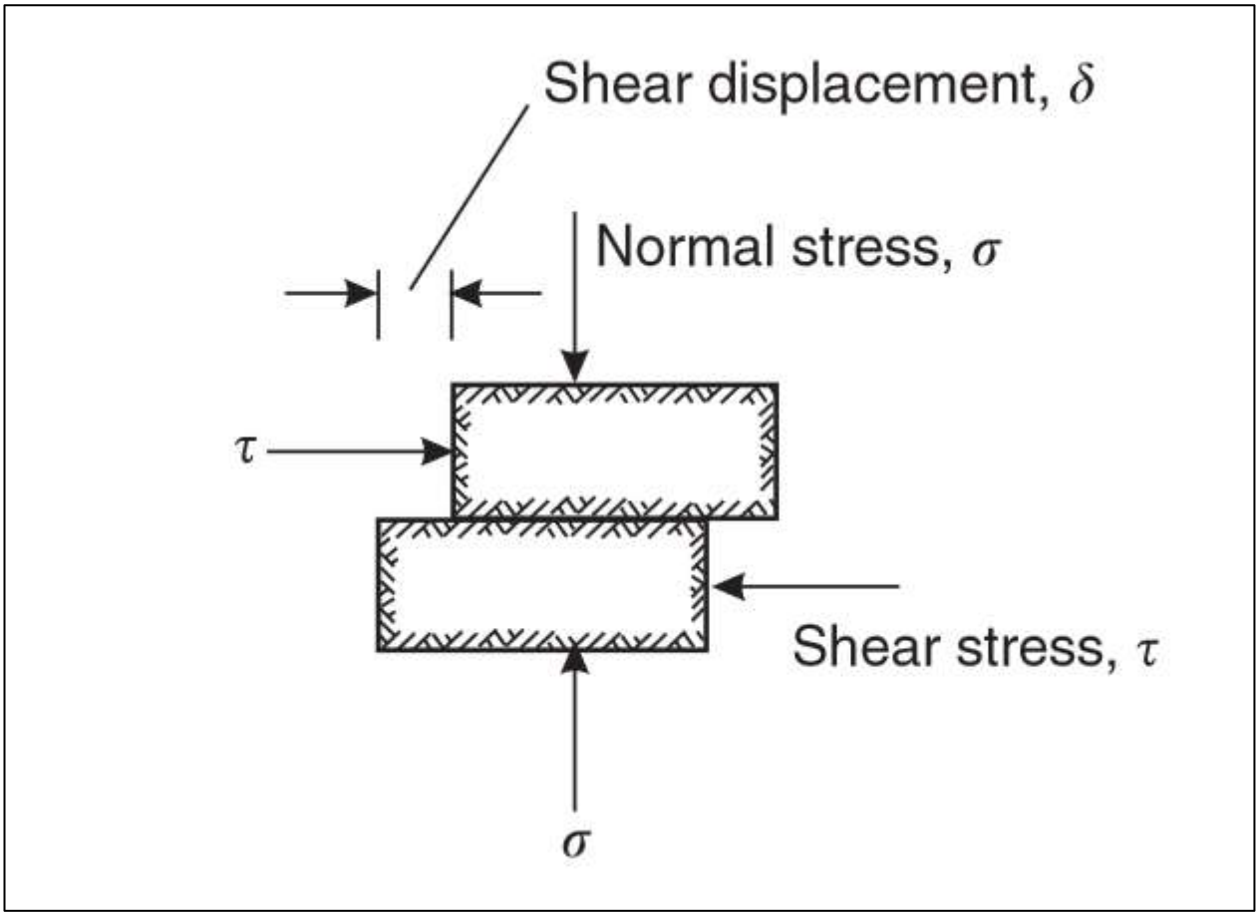


Figure 2-3: shear test of discontinuity

The graph between shear stress and shear displacement under the action of the uniform and lasting normal stress, for test conducted is as depicted below.

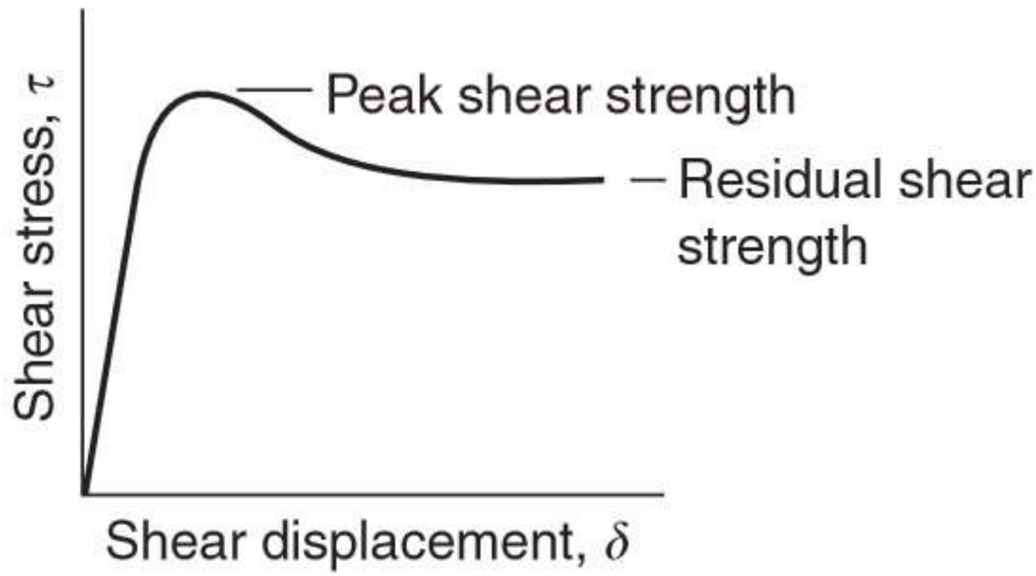


Figure 2-4: plot of shear displacement vs shear stress

Next to minute displacements, sample shows elastic behavior, due to that shear stress, whereby displacement will go further linearly along a linear path, it overcomes the resistance movement that has been caused by force ,a curve is established which is not linear in nature and hence comes to a point whereby it depicts the maximum shear strength of discontinuity. After that a continuing worth described as remaining or residual shear strength, which is necessarily needed to make happen the reducing displacement, causes the forth mentioned. When the maximum shear strength amounts for the conducted tests diverge from totally typical approach, the stress amounts are afterthought, a relationship evaluated thus is shown below.

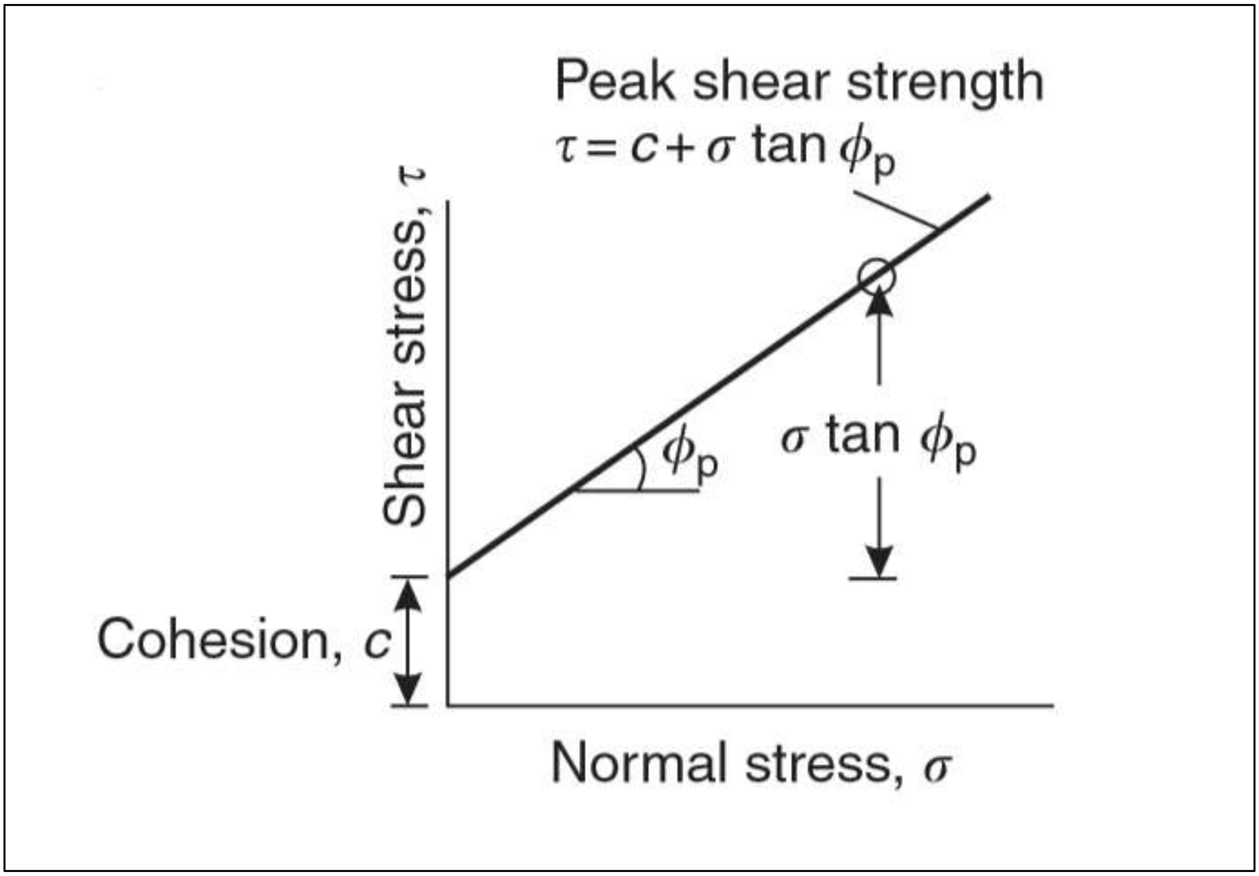


Figure 2-5: Mohr criterion graph of peak strength is shown above

The highlights of this plot are first, that it is around straight and the slant of the line is equivalent to the pinnacle grating point φp of the stone surface. Second, the block of the line with the shear pressure pivot speaks to the strong quality c of the establishing material. The frictional element increases with increasing normal stress. The peak shear strength in terms of equation is as follows:

$$\tau = c + \sigma \tan \varphi_p$$

In the event that the lingering shear pressure esteems at each connected typical pressure are plotted on the Mohr plot, the leftover shear quality line is gotten as appeared.

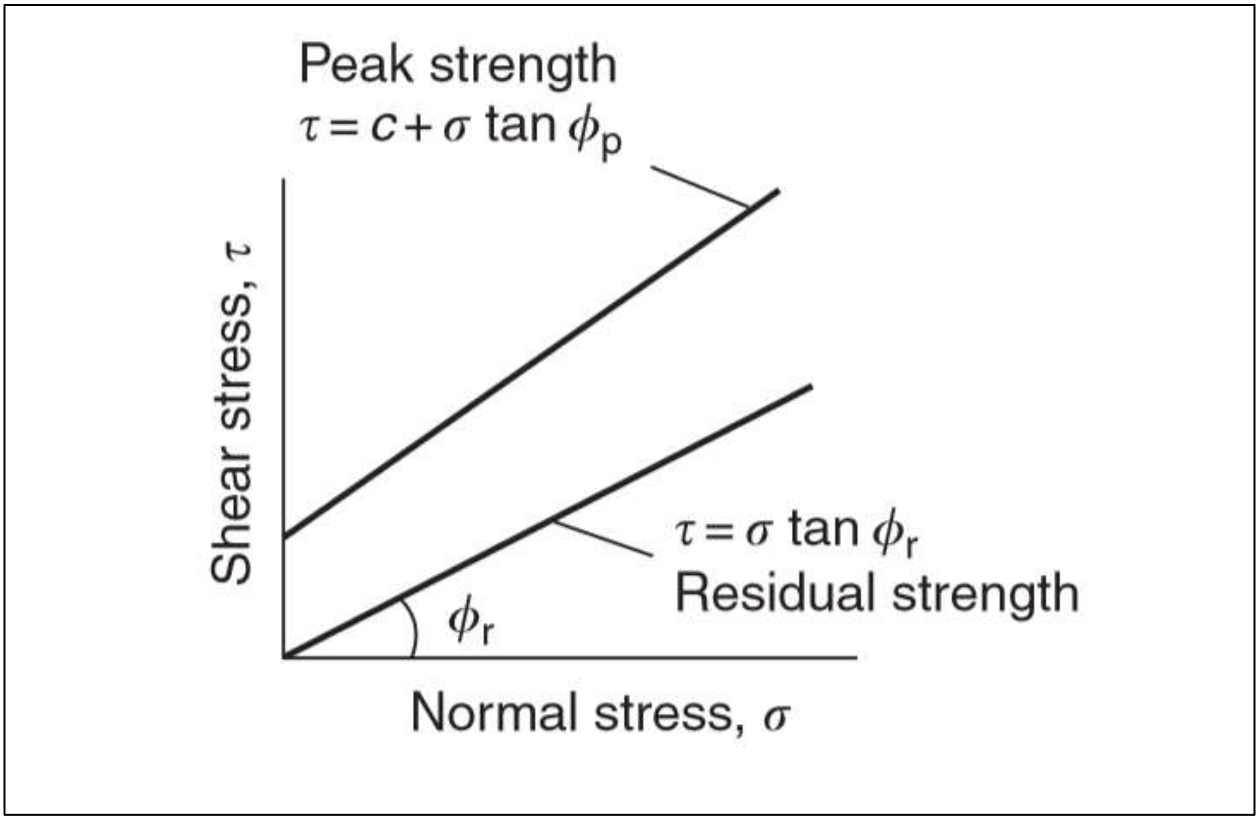


Figure 2-6: Mohr plot of peak and residual strength

The equation gives

$$\tau = \sigma \tan \varphi r$$

where φr is the leftover rubbing edge. For the remaining quality condition, the union is lost once dislodging has broken the solidifying activity; on the Mohr chart this is spoken to by the quality line going through the cause of the diagram. (Mah, 2004)

2.2.9 Laboratory Tests

2.2.9.1 Sonic Wave velocity test (PUNDIT)

It is a P-wave velocity test which has applications in civil, geotech and mining related projects and is carried out in field and in lab. Applications include underground blasting, tearing and opening. The rock mass deformation, stresses and damage zone limit can be predicted using seismological techniques.

Figure 2-7 PUNDIT

It is a common procedure for determining degree of rock weathering and classification. The main purpose of using Ultrasonic Pulse Velocity (UPV) and hardness number as a non-destructive method is to ascertain the elastic and strength of the rocks being studied. ASTM D2845 (1995) states that the ultrasonic pulse velocity is influenced by both the shape and size of the sample under study.

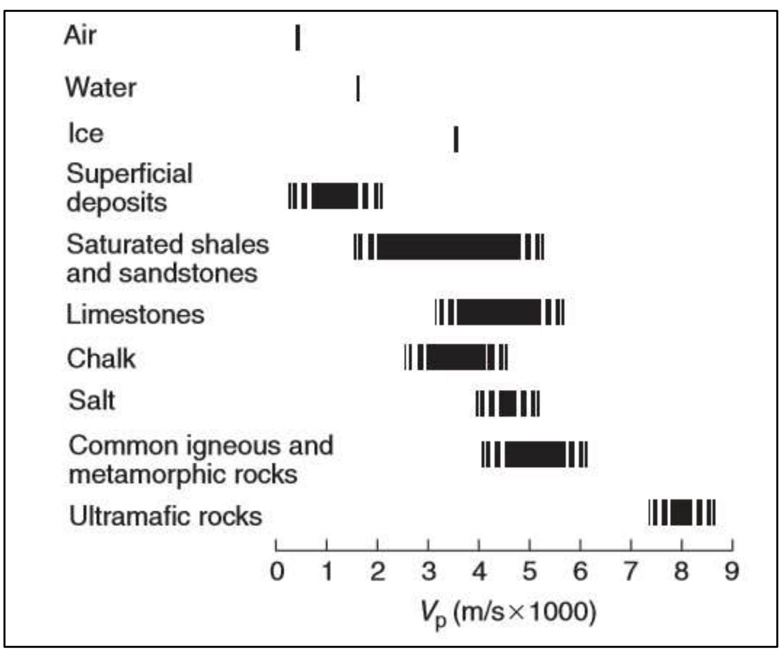


Figure 3 Range of Pundit

2.2.9.2 Brazilian Test

For the test, a sample with L/D ratio as 0.5 is made ready. The sample is placed in the compressive testing machine after which loading is applied at a rate of 200N/sec until there are signs of failure. The test can be either strain or stress controlled. The failure plane lies along the vertical diameter of the sample. For calculation of tensile strength, the equation is:

$$\sigma_t = \frac{2F}{\pi D L}$$

Where σ_t = tensile strength of sample, F = failure load, D=diameter of sample and L= length of sample.

Figure2-8 Brazilian test apparatus

2.2.10 Insitu Stresses

2.2.10.1 Current tectonic forces.

The earth's crust is under duress of tectonic stresses because of disturbance of tectonic plates, as it has been throughout earth's history. These forces and their accompanying stresses may not cause fracture to the rock mass if it is strong but if the stresses exceed the limit, the rock will be subject to fracture causing jointing, faulting and folding. These are established in periods of high stress, during large structural deformations of lithosphere happenings. Throughout the world, there are regions where the tectonic plates are in restrictive balance however rock mass has high occurring horizontal stresses. The maximum horizontal stress will occur horizontally, not in vertical direction, a general rule which is considered.

2.2.10.2 Preceding tectonic forces

Remaining or lingering forces from past structural occasions may exist too: when a split rock mass is liable to pressure and afterward emptied, stresses will be left in the stone mass.

Discontinuities

The Mohr chart demonstrates the impact of geography on shear quality, relative quality parameters for the 3 sorts of brokenness on two examples of rock mass. The contact edge is spoken to by the incline of each line and the attachment is appeared by the block with the shear pressure pivot.

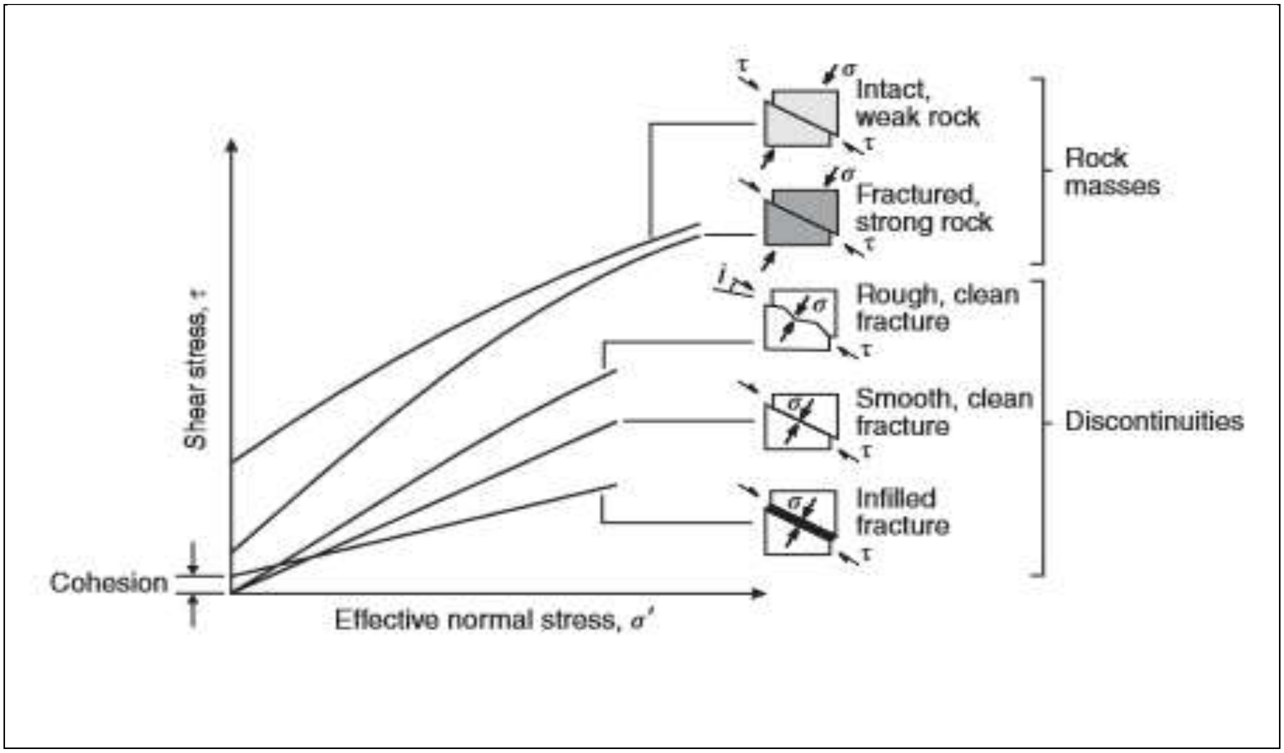


Figure 2-9 Connections among shear and ordinary weights on sliding surface for five distinctive topographical conditions (Transportation Research Board, 1996).

Curve 1 Infilled discontinuity: The infilling friction angle (φinf) is probably going to be small if the infilling is of fragile clay or fault groove, but there is a chance that there may exist some cohesion if infilling is not disturbed. On the other hand, if infilling is a strong calcite for example, the cohesive strength may be very great as calcite produces a healed surface.

Curve#2 even discontinuity: A smooth, dirt free discontinuity will have no cohesion, and critical friction will be equivalent to that of the stone surfaces (φr). The grain size in a straight line relates to abrasion angle of rock and is generally smaller in fine grained rocks as compared to their coarse-grained counterparts.

Curve #3 jagged discontinuity: Zero union will be shown by unsoiled, rugged irregularities surfaces and the grinding point will be compromised of two parts. The main part is the stone material erosion edge (φr), and second is identified with the harshness (severities) of the surface and the proportion between the stone quality and the ordinary pressure. As the typical pressure builds, the ill tempers are logically sheared off and the all out contact point gets littler.

Curve#4 Fractured/split rock mass:. There is stumpy confinement of the fractured rock under low normal stresses and the individual fragments of the fractured rock may move and rotate. The friction angle will be greater but the cohesion is low. At higher normal stresses, the rock pieces start getting crushed leading to a decrease in the friction angle. The grade of fracture, strength of whole rock determines the shape of the strength envelope.

Curve 5 Weak intact rock: The Rock accordingly appeared in figure underneath is made out of fine-grained material with a low grating edge. Nonetheless, in light of the fact that there exist no discontinuities in the stone, the attachment can be higher when contrasted with that of a solid unblemished shake that is intently broken.

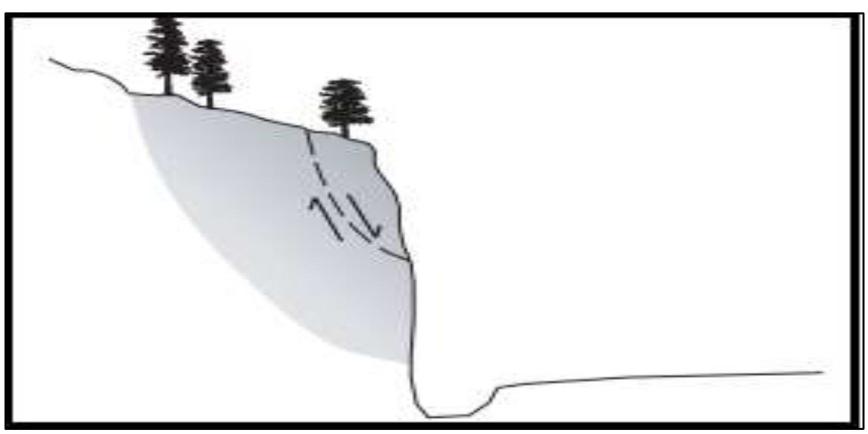


Figure 2-10 No discontinuities, shallow failure mode

2.2.11 Strength Tests

2.2.11.1 Bearing Capacity Tests

Bearing Capacity test is the ability of the rock to resist stress before failure. In this check load to rock is utilized through plate in upward direction. Plate size is 300 mm to 450 mm in diameter but may additionally be various for weak rock. Load application on plate must be concentric because eccentric loading may cause the plate to tilt and result in uneven settlement. Bearing capability of rock depends on a number elements such as fault zones, composition of rock mass, porosity, ambient pressure, degree of soaking and water strain in rock mass. Bearing capacity may also be determined using Terzaghi formula:

$$\mathbf{q_f = CN_c + qN_q + 0.5\rho b N_y}$$

Where $\mathrm{N_c}$, $\mathrm{N_q}$ and $\mathrm{N_y}$ are bearing capacity coefficients, depending upon φ of rock mass.

ρ = unit weight of rock mass,

b = width of footing,

C = cohesion and

$\mathrm{q_f}$ = surcharge load at footing level

2.2.11.2 Compressive Strength Test

If planes of weaknesses do no longer exist in the rock mass, compressive force may be determined via lab testing. But subsistence of imperceptible planes of weak point may also cause damage to essential structures in lengthy run. The Austrian School suggests that move sectional location of specimen have to contain 100-200 planes of weakness.

In addition Hoek (1966) suggests that if compressive strength of sample is to be within 10% of in-situ value, the amount of specimen need to be 50-100 instances the spacing between discontinuities. Consequently size of specimen for compression take a look at has to be massive and case might also be circular, rectangular or square prism. Load is put on via set of hydraulic jacks. Displacements are calculated using dial gauges or mechanical pressure gauges. Load is taken on up to failure value. Compressive strength is determined by means of dividing the left behind load by means of vicinity of specimen.

Figure 2-11 UCS testing

2.3 Rock Slope Stability

2.3.1 Stress Analysis using numerical techniques

Numerical fashions are arithmetical displays making use of numerical process to decide performance of model. It hence depicts mechanical behavior in return of rock mass that are related to stresses which are in situ in nature, boundary prerequisites, prompted modifications like slope excavation. Mathematical techniques by which rock mass is separated into various regions and each region is allocated a material mannequin and attributes. Numerical modeling techniques are appreciably used to resolve complicated slope problems. These models are utilized to model the rock slope as appropriately soil slope with defined and complex conditions. Fault, bedding planes and joints and spacing are modelled.

2.3.1.1 Boundary Methods

Considering these, excavation boundary is separated into factors whereas indoors of rock mass, like an infinite continuum, is shown arithmetically. This makes the ongoing procedure much easier as little effort is needed owing to the fact of fewer elements used. These models can be grouped as follows:

- Indirect approach or fabricated stress method, where the acting stresses are determined by stresses applied or theoretical in nature.
- Direct method, where displacements are intended straight away for boundary conditions.
- Displacement discontinuity method, wherein effects of extended slit in elastic continuum, is set apart.

2.3.1.2 Domain Methods

In these methods inside of the rock mass, separated, in easy factors of geometric shapes and assumed properties. Rock mass could be divided into various parts, but for the mentioned behavior, a struggle is required to shape the mesh, which will make it happen. They include:

- Finite issue and finite distinction methods, relate to the stipulations at nodal factors within the rock inside finite closed location fashioned via these points. Physical trouble is numerically modeled into factors through division of entire hassle region. This approach is nicely acceptable to clear up problems containing nonlinear or heterogeneous properties. However, they are no longer desirable for solving infinite boundaries.

- Distinct element method, individual rock is modeled as a unique element. The drawback of this method is it uses express solution practices, as it is highly nonlinear in nature.

2.3.1.3 Hybrid Approaches

Great rewards and removal of unwanted properties is the hallmark of hybrid approach. Help was taken by Lorig and Brady (1984) of a fusion model. It consisted of distinct models and boundary models for near and boundary elements respectively. Purpose behind this half breed approach is that most extreme nonlinearity happens near removal limit while the rock mass at some separation carries on in a versatile manner (Hoek E. , 2000). Finite element groundwater and equilibrium and limit equilibrium analysis is used by Geo-Slope 200 to analyze stress. Modern advances include coupled particle flow and finite difference analyses using FLAC3D and PFC3D (Itasca 1999). The most widely used discrete element codes for slope stability studies are UDEC and 3DEC (Mah, 2004).

2.3.1.4 Continuum versus Discontinum Models

Incline steadiness investigations constantly include discontinuities. On the off chance that rock mass can be named a proportional continuum, utilization of continuum models is prompted. Discontinuities are presented by consolidating interfaces between continuum bodies, for example, faults and bedding planes etcetera. This model is unfit to deal with exceptionally break shake mass; be that as it may, can recreate up to 10 non-crossing discontinuities. Limited distinction, limited component and limit component strategies depend on this displaying hypothesis. Complex conduct of inclines is displayed utilizing continuum codes. It is additionally utilized for reproduction of groundwater, pore weights and dynamic collaboration etcetera. PHASE, FLAC2D, FLAC3D, PLAXIS and VISAGE are based on continuum models.

2.3.1.5 2D investigation against 3D investigation

Slant reproduction can be accomplished in 2D and 3D utilizing numerical displaying. This kind of reproduction relies upon variables, for example, time required, basic parameter, PC setup, field conditions and design of PCs. Two-dimensional geometry accepts plane strain conditions for a unit cut through an interminably long incline, for example sweep of toe and peak are thought to be limitless. While, three-dimensional investigations are required when major land discontinuities don't strike inside 20-30°of the strike of slant. FLAC and UDEC are two-dimensional limited distinction programs grew explicitly for geo-mechanical examination. These codes can mimic shifting stacking and water conditions, and have a few pre characterized material models for speaking to shake mass continuum conduct. The two codes are one of a kind in their capacity to deal with exceptionally non-straight and precarious issues. The 3D correspondents of these codes are FLAC3D (Fast Lagrangian Analysis of Continua in 3D and 3DEC).

2.3.2 Strength Failure Criterion

Some of the strength failure criterions are listed in table 1:

Failure criteria	Method of derivation	What is derived
Hoek Brown Failure Criterion	Tri-axial lab tests or experiential observations	Rock mass failure
Mohr Coulomb Failure Criterion	Tri-axial lab tests	Rock mass failure
Coulomb's Shear Strength Criterion	From lab tests	Shear Failure
Griffiths Failure Criterion	Empirically with formulae	Crack initiation
Barton Brandis	From lab tests and discontinuity surface observations	Shear Failure

2.3.2.1 Hoek Brown Failure Criterion

Some initial challenges emerged on the grounds that numerous geotechnical issues, especially in regards to incline security examination, are all the more advantageously managed as far as shear and typical burdens as opposed to the main pressure connections of the first Hoek–Brown model, characterized by the equation:

$$\boldsymbol{\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a}$$

Where $\boldsymbol{\sigma_1}$ and $\boldsymbol{\sigma_3}$ are maximum and minimum effective stresses at failure, m_b= Hoek Brown constant , $\boldsymbol{s, a}$ =constants which depend upon rock mass characteristics and $\boldsymbol{\sigma_{ci}}$ is uni-axial compressive strength of intact rock pieces. Generally $\boldsymbol{s}$=**1** for intact rocks.

Geological Strength Index (GSI)

Geological strength index (GS)I gives a framework to evaluating the decrease in rock quality for various topographical conditions. Estimations made for GSI are identified with both the level of cracking and the state of break surfaces. The summed-up Hoek–Brown quality basis is communicated as far as the major and minor principal stresses, the equation could be tailored as

$$\boldsymbol{\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a}$$

where $\boldsymbol{m_b}$**=** condensed value of the material constant **m**$_i$=for intact rock and is as follows

$$\boldsymbol{m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)}$$

The quality of a jointed piece of rock relies upon the properties of the flawless rock pieces, just as the opportunity of the stone pieces to slide and pivot under various pressure conditions. This opportunity is constrained by the geometrical state of the flawless rocks and the state of the surfaces isolating the pieces. Precise shake pieces with spotless, harsh surfaces will result in a lot more grounded rock than one that contains adjusted particles encompassed by endured and modified material.

Hoek Brown Criterion's Limitations

- Hoek-Brown model is a shear-based measure.
- Only σ_1 andσ_1 are utilized and σ_2 is disregarded.
- It likewise just applies to the "focal" scope of shake masses, for example all around jointed shake masses in which the joints control conduct as opposed to the stone material or individual critical planes of shortcoming.
- In weak shake, disappointment will at first create as harm of the stone, trailed by spalling disappointment and after that eventually progress to shear disappointment.

2.3.2.2 Mohr-Coulomb Failure Criterion

The leftover quality of the stone as shear nonconformity continues, on the disappointment surface that has been made in the disappointment of the unblemished shake is generally assessed by Mohr Coulomb basis. The standard is generally composed as

$$\boldsymbol{\tau = C + \sigma_n \tan\varphi}$$

where C is the cohesion, $\sigma_n$is the normal stress and φ is angle of friction.

Mohr-Coulomb's Failure Criterion Restrictions

- Most likely mistaken outcomes are given when the non-conformity isn't shear as real shear break happens at pinnacle quality
- Relapse is initiated along a sliding surface within the slope
- Cohesion and friction are considered to be in unison
- solitary σ_1 and σ_3 are utilized and σ_2 is disregarded

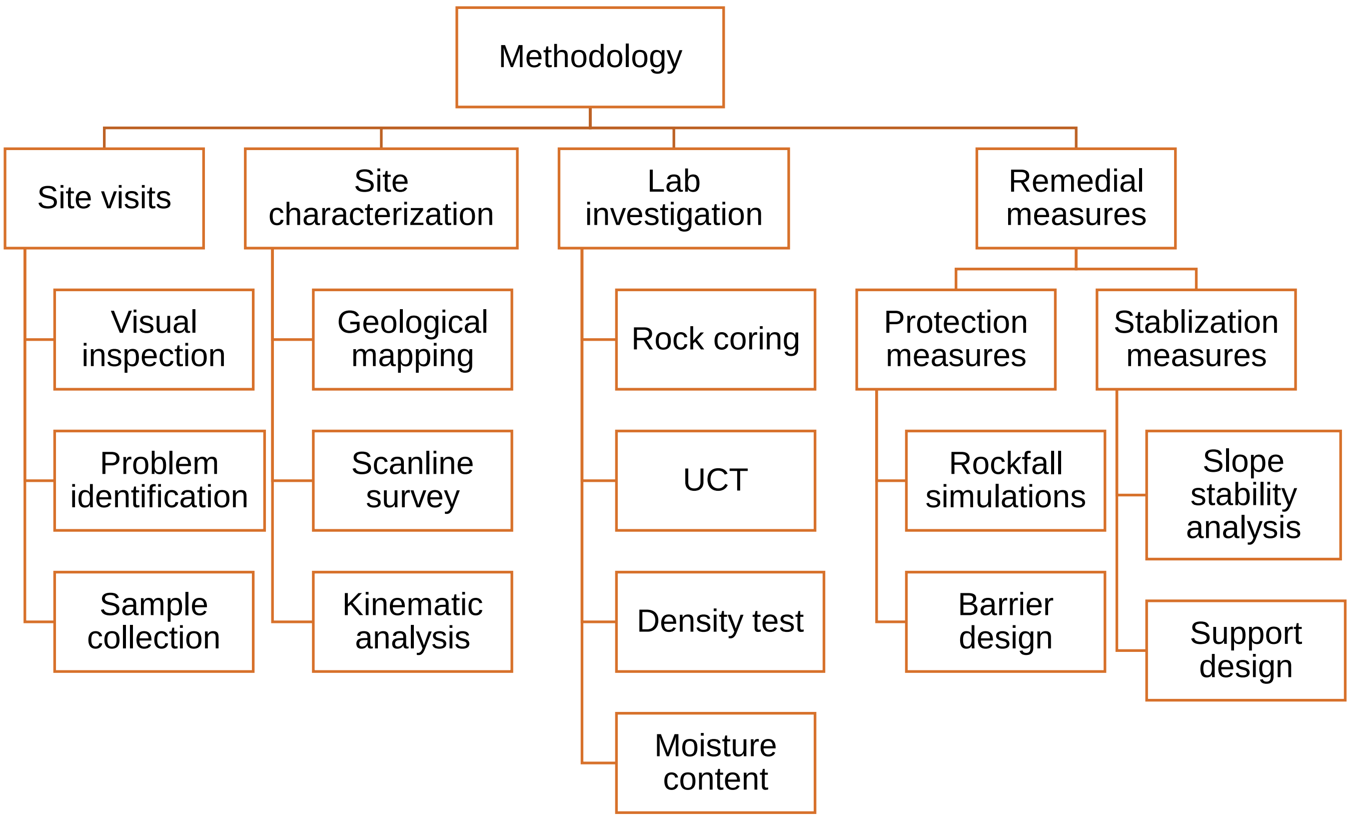
2.4 Scan line Survey

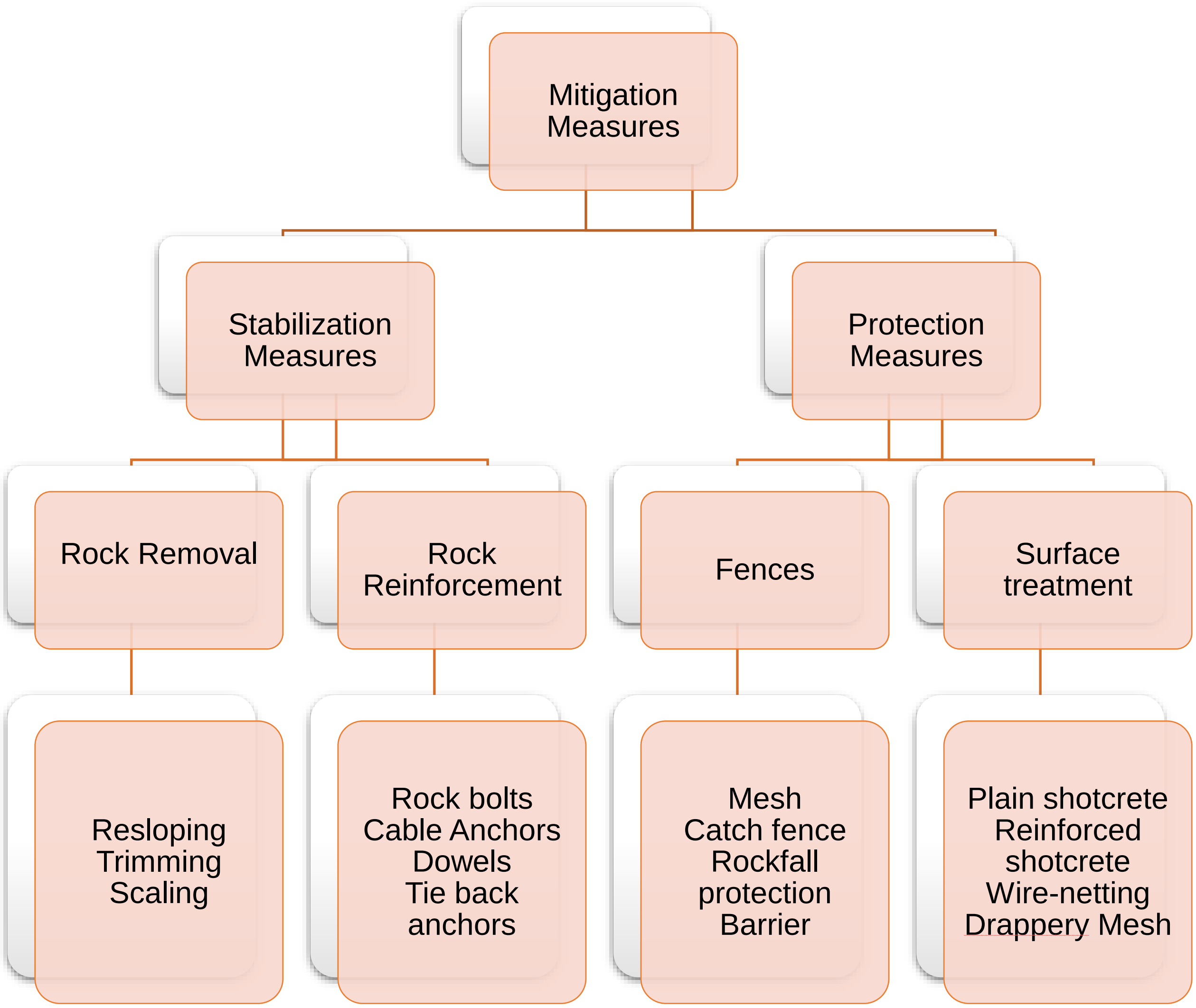
Scan line survey is a survey technique in which over a rock surface that is outcropped a line is drawn and all of the discontinuities that are intersecting the surface are first measured and then described. For a rock mass a discontinuity geometry is characterized by the mean density, number of discontinuity sets and distributions for orientation, location, spacing/ fracture intercept and size. The investigation on site of rock deal with certain crucial elements that need addressing, information that is required to characterize the system of rock and the built-in uncertainty that is linked with this info. In this way, quantifying the content of measurements taken on-site and subsequently creating a database is crucial for decision making and assessment of risk on projects of rock engineering design.

It is a technique that is widely applied to tackle geological problems that are complex by simulating different geological scenarios. Mathematical models are used to show physical conditions of different geological scenarios by the use of equations and numbers. Some of the equations like partial differential equations which are very difficult to solve in a direct manner can be solved using numerical modelling methods like the finite difference method to find the solution to these equations. In these models, numerical experiments can be performed which yield results which we can interpret in context of processes that are geological. Through these experiments both quantitative and qualitative of different geological processes, through these experiments can be developed.

CHAPTER – 3
RESEARCH METHODOLOGY

3.1 General




Mitigation Measures
Stabilization Measures
Protection Measures
Rock Removal
Rock Reinforcement
Fences
Surface treatment
Resloping
Trimming
Scaling
Rock bolts
Cable Anchors
Dowels
Tie back anchors
Mesh
Catch fence
Rockfall protection
Barrier
Plain shotcrete
Reinforced shotcrete
Wire-netting
Drappery Mesh

3.2 Laboratory Testing

3.2.1 Rock Coring

In this process we recover cylindrical cores of rock by rotating a hollow steel tube (core barrel) that is equipped with a coring bit. We than collect the drilled core in the core barrel. The cores need to be handled carefully so that their properties are not changed due to mechanical damage or changes in temperature, moisture or other environmental factors. Rock coring is carried out to record the existing conditions below the surface at the location of boreholes. The rock cores provide indication about the physical, geological and engineering nature of the subsurface, this is then used in the designing and construction of engineering structures. The core samples are preserved using specific procedures for a specified time. The period of storing the sample is dependent upon the nature of the structure

Figure 4-1 Rock coring

Figure 3-2 Berzelian Rock Cut

3.2.2 Density of Rock Mass

Density of rock slope is calculated in two steps. First, the density of intact rock sample is calculated. Then empirical formulae is used to determine the density of the rock mass

3.2.3 Rock Moisture Content

First the mass of the sample is determined. Then the sample is put in an oven at 110 $\pm$ 5°C till the time we get a constant mass. The mass that is lost, during drying, is considered water. The moisture content is calculated by the ratio of water mass to the mass of dry sample expressed in percentage. For rock materials, the moisture content holds significance in establishing a correlation between rock behavior and its index properties.

3.2.4 Uniaxial Compression Test

The test is performed by putting the sample specimen in the loading machine between its two plates, the vertical and lateral deformations are measured by attaching the strain gauges. The rate of loading for the stress control loading arrangement should be 0.5 – 1 MPa/second. After this the Elastic constants are found by the drawing of stress strain graph and empirical formulae.

Figure 3-3 UCT on Rock Core

Figure 3-4 Strain gauges on rock core

3.2.5 Poisson Ratio and Modulus Test

Young's modulus of elasticity and Poisson's ratio of rock specimen are measured from uniaxial compression test, if the stress strain behavior till peak is captured. The elastic modulus of the specimen is given by the slope of the stress-strain curve.

Young's modulus can be calculated using any one of several methods plotted by stress - strain curve. Tangent young's modulus is measured at stress level. Secant young's modulus is calculated from zero stress to some fixed percentage of the ultimate strength taken normally at 50%. The ratio of the total diametric strain to the total axial strain at any stress level is the Poisson ratio.

3.2.6 Schmidt/ Rebound Hammer test

In Schmidt test, the concrete surface is pressed by the plunger of the rebound hammer – the concrete surface is hit by a spring-controlled mass with a constant energy. The spring controlled mass then rebounds. The surface hardness is measured by the extent of the rebound on a graduated scale. The measured value is the Rebound Index.

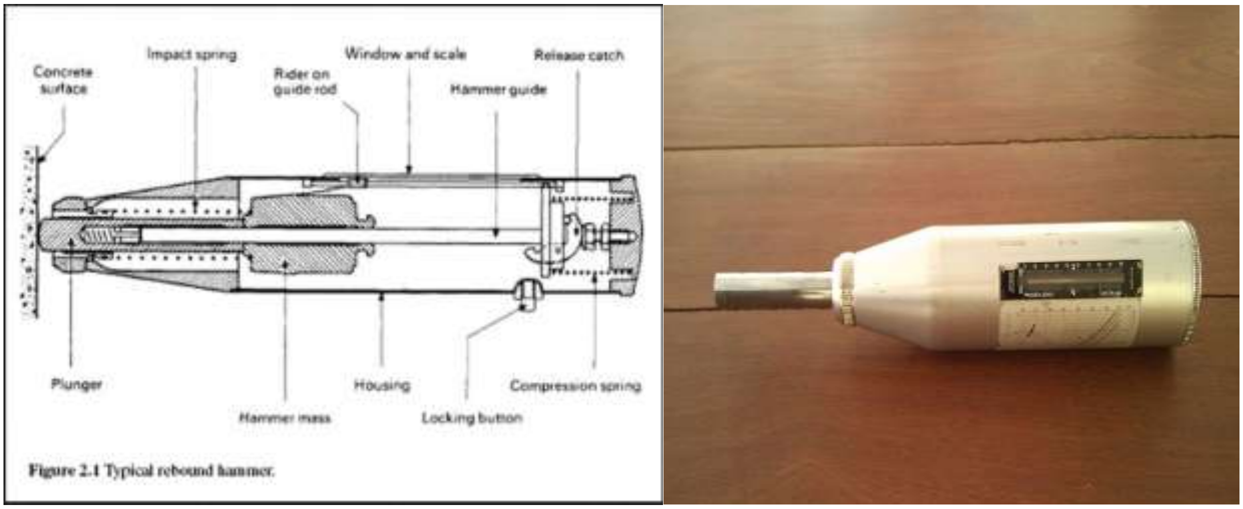


Figure 3-5 Schmidt Hammer

Figure 3-6 Schmidt Hammer test

3.2.7 System 8000

This system is made for dynamic and static test and for the measurement applications. Scanners of the system may independently operate, or for a maximum number of 128 channels, up to sixteen scanners can be used at the same time. The scanners may independently operate.

A twenty-four-bit digital signal processor processes each channel's data and finite impulsive response (FIR) filters perform the filtering. This gives excellent stability, noise rejection and unsurpassed accuracy of the measurement.

Figure 3-7 System 8000

3.2.8 Strain gauge

It is a sensor in which there is a variation in resistance with the force applied. It converts pressure, force, weight, tension, etc into electrical resistance which can be measured then. External forces applied onto a stationary object result in stress and strain. Stress is the internal resisting force of the object while strain is the deformation and displacement that occurs.

Figure 3-8 Strain Gauge

3.2.9 Load cell

A load cell is an electrical device that creates an electrical signal whose magnitude is proportional to the applied force. Strain gauge load cells have very good values of resonance, are very stiff and in application they have long life cycles. Their working principle is that deformation in strain gauge when there is appropriate deformation in the material of the load cell. Strain gauge deformation changes its electrical resistance proportional to the strain. This resistance change in strain gauge causes a change in the electrical value that is fine-tuned to the load that placed on the load cell.

Figure 3-9 Load Cell

3.2.10 Ultrasonic pulse velocity test

Ultrasonic pulse velocity test gives a nondestructive method to characterize the properties of geological core sample that is subjected to high pressures and temperature in a triaxial cell. It gives crucial data about the specimen rock structure, elastic properties, deformational stress and other properties. In these tests we propagate two shear waves that are orthogonal and an ultrasonic wave along the longitudinal axis of the specimen, then as the wave passes through the specimen measures the wave velocity to calculate dynamic elastic attributes including bulk modulus, young's modulus and Poisson's ratio.

Figure 3-10 PUNDIT

3.3 Estimation of Geotechnical Parameters for Analysis and Design

Rock mass density(w), internal friction (φ), Poisson ratio(ϑ), unit weight (γ), elastic modulus of rock mass (E) etc are the geotechnical parameters needed for analyzing slope stability and designing prestressed structures. The estimation/ determination of these parameters is done with the use of excel sheets.

3.3.1 Rock Mass Density

There is a significant difference of density among various rock types because of the differences in porosity and mineralogy. The interpreting of subsurface rock type and geologic structure can be done using the knowledge of underground rock density distribution. To assess the rock densities a useful way is to create a histogram using statistical range of the available data set. The representation of the value and its variation can be expressed as follows: (1) Mean, (2) Mode, (3) Median, (4) Standard deviation.

3.3.2 Internal Friction

The angle of internal friction is the measure of shear stress withstanding ability of a unit of rock or soil. Angle (φ) is measured between the normal force and its resultant force which is attained when the failure occurs in response to the shearing stress. Coefficient of the sliding friction is its tangent. We determine its value experimentally

3.3.3 Poisson ratio

Poisson's ratio is ratio between transverse contraction strain and the longitudinal extension strain applied in the direction of stretching force. The compressive deformation is considered to be negative and the tensile deformation positive. Poisson's ratio definition has a negative sign so that normal materials have positive ratio.

$n = - e_{trans} / e_{longitudinal}$

3.3.4 Unit Weight

The unit weight is the total weight of a substance in a single unit of volume. Although similar to density or specific gravity, they are different because weight is dependent on the gravitational acceleration. Unit Weight can easily be calculated if the object density and value for the gravitational acceleration are known.

3.3.5 Elastic Modulus of Rock Mass

In case of underground excavations, the elastic modulus of rock mass is among the parameters that best represent the mechanical behavior of a rock and of a rock mass. This is the reason why most boundary element and numerical finite element analysis for studying displacement and stress distribution around underground excavations are mostly based on this parameter. The elastic modulus is therefore the basis of many geomechanical analysis.

3.4 Analysis to Evaluate the Instability Mechanisms

Many modern numerical tools are available to determine rock slope instability mechanisms such as Slide, Roc Fall, FLAC, UDEC. Slide and RocFall would be used for for the project.

3.4.1 RocFall

It is a two-Dimensional simulation software to predict the behavior of rockfall on slopes and to design the rock fall barriers. RocFall les its users toggle between Design mode and the Results mode. After the computational of results, they will only be deleted and recalculated if the model has been changed. RocFall allows to toggle between the two modes without having to recalculate the results as long as the model is unchanged. Data collectors can be edited and added without having to recalculate results.

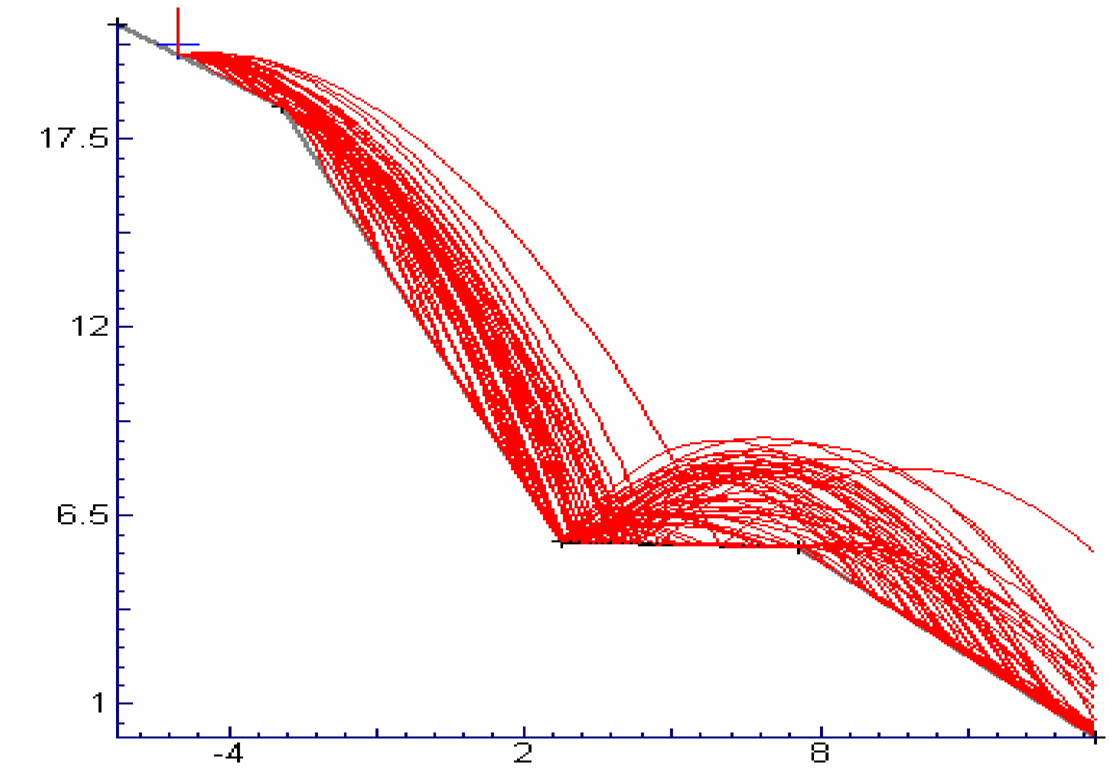


Figure 311 Rockfall Prediction

3.4.2 Slide

It is a Two-Dimensional limit equilibrium slope stability software that evaluates the probability of failure or the safety factor, of failure surfaces in rock or soil slopes. It analyzes the slip surface stability by using either non-vertical or the vertical slice limit methods of equilibrium. It also has ground water seepage analysis of the finite element in-built for both transient and steady state conditions.

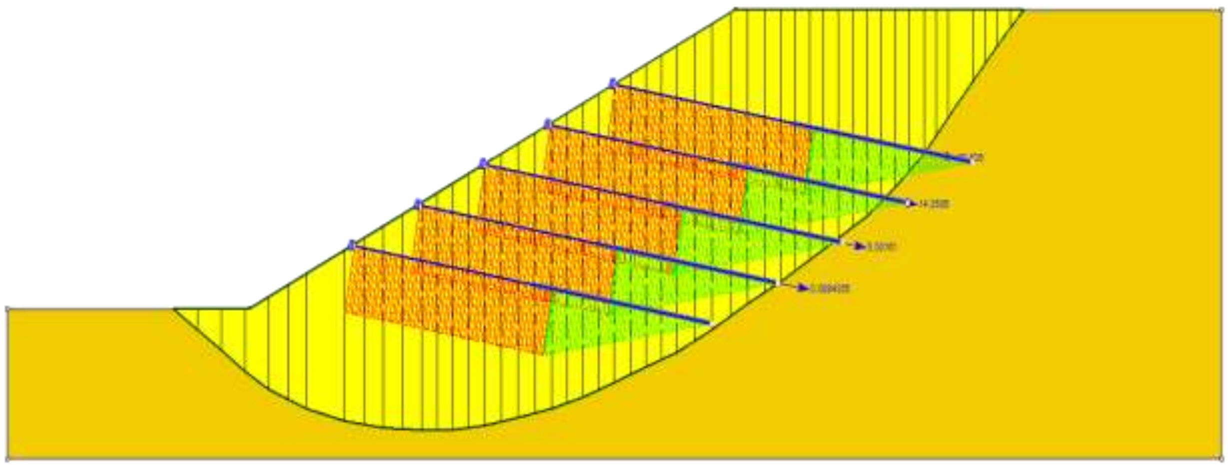


Figure 3-12SLIDE

3.5 Slope Stabilization Options

Efficiency of conventional slope stabilization options like slope flattening, surface/ subsurface water control, structural and mechanical stabilization will be determined on the basis of construction time requirement, cost effectiveness, working space requirements to install structural system and the estimated system life span.

3.6 Conclusion

Chapter covers methodology of the research work. In the start of this chapter, literature review overview was present after which the organization for site characterization which covers laboratory tests are covered. Different options were conferred for the slope stabilization and discussion was carried out on the mechanism of instability of rock slope. Geostructurual model salient and its numerical validation was highlighted that paved way towards the guidelines of construction.

Chapter – 4
Site Characterization

4.1 General

Site characterization in rock mechanics during the initial field studies of the project continuing with a variety of processes including rock coring, laboratory testing, aerial photography and geological mapping, permeability and testing of the mechanics of rocks in the area (insitu testing) etc. The purpose of the characterization of the site is to develop an understanding of the geology, geometry and properties of the material present at the candidate site. It also helps to identify the geological issues that could cause problems for the proposed installation. During Site characterization it is not possible to access all issues relating to the geology material and geological properties. Hence this process should ideally continue during entirety of the construction and design processes.

4.2 Phases in investigation

- Collection of available data/information
- Preliminary reconnaissance or a site visit
- Detailed soil exploration/sample collection (scanline survey)
- Lab Testing
- Development of geological profile (geological mapping)

4.3 History of Project Site (RD-37)

Swat Motorway is a planned 81 km long Motorway in Khyber Pakhtunkhwa district of Pakistan with its north end connecting Chakdra in Lower Dir district with Nowshera. It is being constructed by Frontier works organization with Asian Development Bank and China providing principal technical and financial assistance to the Government of Khyber Pakhtunkhwa. RD 37 site has unstabliized rock cut slope due to blasting of local rock formation needed for the pavement of motorway according to design. Hence RD 37 needs protection and stabilization measures to avoid landslide and rockfall.

4.4 Preliminary site visit/reconnaissance

- Site visits were carried out and different rock types were observed which were:
  - Marble
  - Green Schist
  - Graphitic Schist
  - Calcareous Phyllite
  - Coal Bed
- Rock mass was observed to be weak especially green schist, graphitic schist and coal bed.
- Vegetation was observed on the top of site.
- Preliminary benching was done on RD 37 site but was unstable and evident by rock fall in drainage channel and Road.
- Schmidt hammer used on site gave very weak strength. Preliminary UCS of schist was very low i.e. 12MPa.

4.5 Scanline Survey

4.5.1 Introduction

Rock stability is controlled by shape, type and number of discontinuities. Rock may fail in many ways depending on the geometry of discontinuities. It may fail in form of Toppling failure, Plane failure or Wedge failure. Kinematic analysis using stereometric projection, give the geometry of discontinuities whose result can be used to predict which type of failure is most possible. For stereonetic analysis, scanline survey is essential for determining those required parameters. They include:

- Type of discontinuities
- Persistance
- Aperture
- Nature of infilling
- Spacing

- Roughness
- Water condition
- Lithology

Scanline for slope stability at Swat Motorway was carried out from RD 36+770 to 37+200 km at left side of the box cut by Geological department of Military College of engineering and a report has been prepared in this regard.

The following parameters were observed in this regard:

RD 36+904 to 37+200	
Persistence	Moderate to high
Aperture	Narrow to tight
Infilling	Surface stain, clay, Quartz
Spacing	Wide to close
Roughness	Smooth, rough to slightly rough
Water condition	Wet to dry

Using Roclab and evaluating properties for numerical analysis we obtained the following results:

Rock Type		Schist	Marble
Hoek Brown	Intact UCS(Mpa)	15	110
Classification	GSI	35	40
	Mi	10	9
	D	1	1
	Intect Modulus Ei(Mpa)	10125 Mpa	93500
Hoek Brown	Mb	0.096	0.124
Criteria	S	1.97e-5	4.54e-5

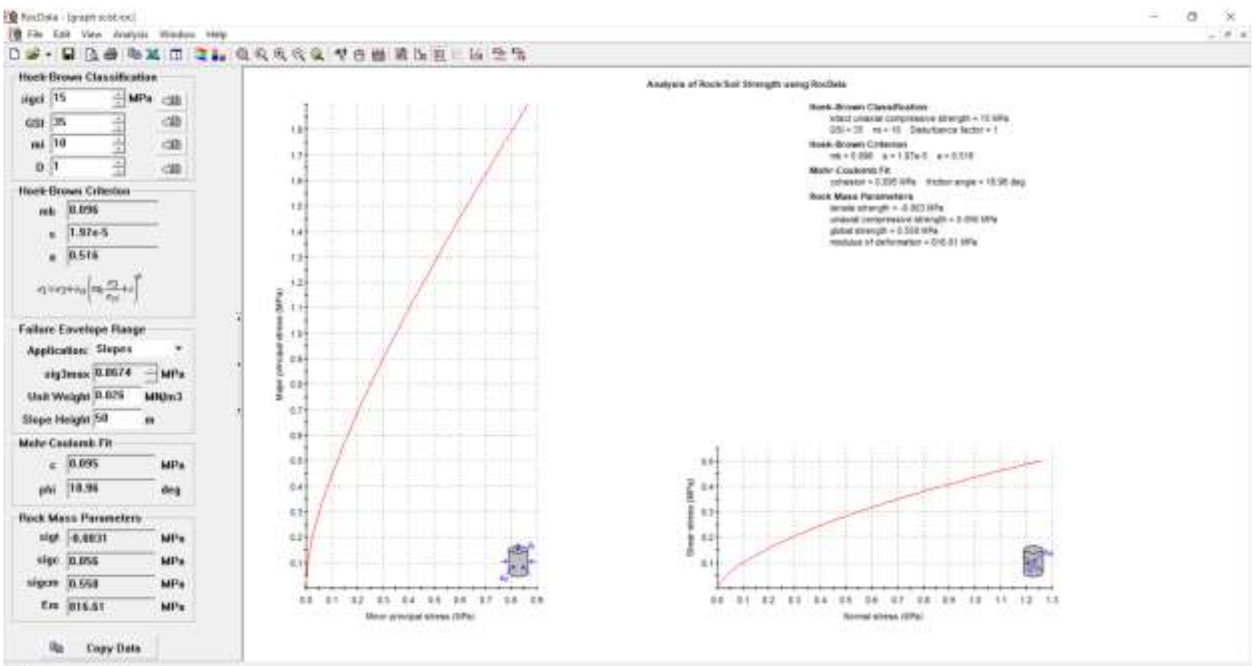


Figure 4-1 Rocdata for Green Scist

Roc Data for Green Schist

Hoek-Brown Classification		
Sigci	15	MPa
GSI	35	
Mi	10	
D	1	
Hoek-Brown Criterion		
Mb	0.096301	
S	1.97E-05	
A	0.51595	
Failure Envelope Range		
Application	Slopes	
sig3max	0.867448	MPa
Unit Weight	0.026	MN/m3
Slope Height	50	m
Mohr-Coulomb Fit		
C	0.094579	MPa
Phi	18.9596	degrees
Rock Mass Parameters		
Sigt	-0.00307	MPa
Sigc	0.056056	MPa
Sigcm	0.558363	MPa
Em	816.612	MPa

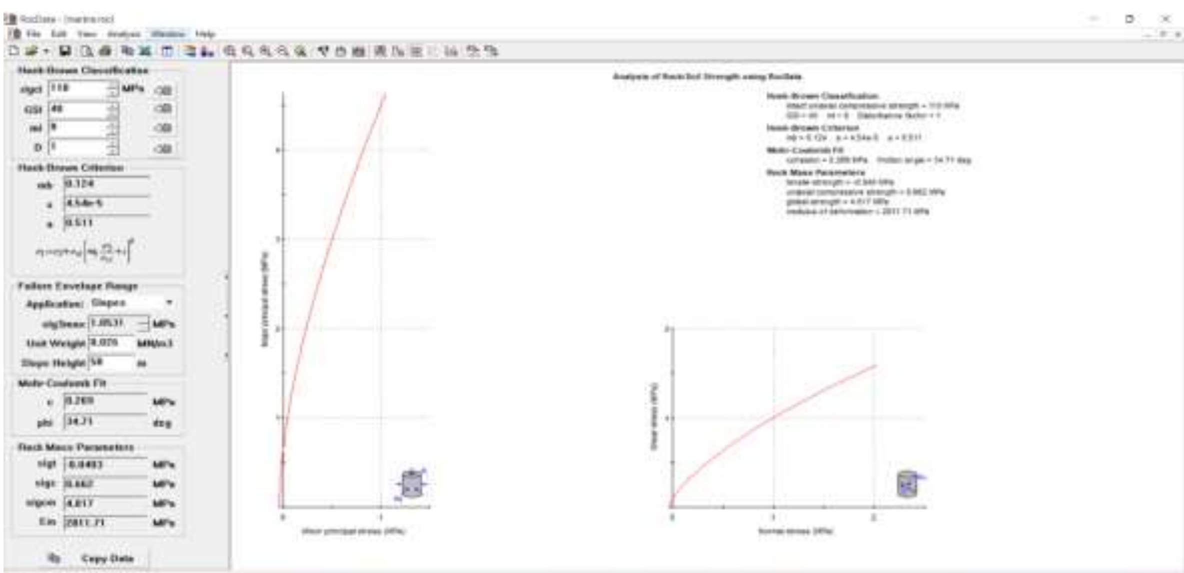


Figure 4-2 Rocdata for Marble

Rocdata for marble:

Hoek-Brown Classification		
Sigci	110	MPa
GSI	40	
Mi	9	
D	1	
Hoek-Brown Criterion		
Mb	0.123874	
S	4.54E-05	
A	0.511368	
Failure Envelope Range		
Application	Slopes	
sig3max	1.0531	MPa
Unit Weight	0.026	MN/m3
Slope Height	50	m
Mohr-Coulomb Fit		
C	0.269129	MPa
Phi	34.7063	degrees
Rock Mass Parameters		
Sigt	-0.04032	MPa
Sigc	0.661527	MPa
Sigcm	4.81664	MPa
Em	2811.71	MPa

4.5.2 Geology

Geology is mainly composed of Schistose Marble and Schist and Phyllite. Coal seams were observed at a point. The strata is highly folded, jointed and foliated. Strength of Marble lies in medium strong condition and Schist/Phyllite ranges from very weak to weak. Average joints were filled with quartz in marble. Clay infilling and surface staining was found in Schist.

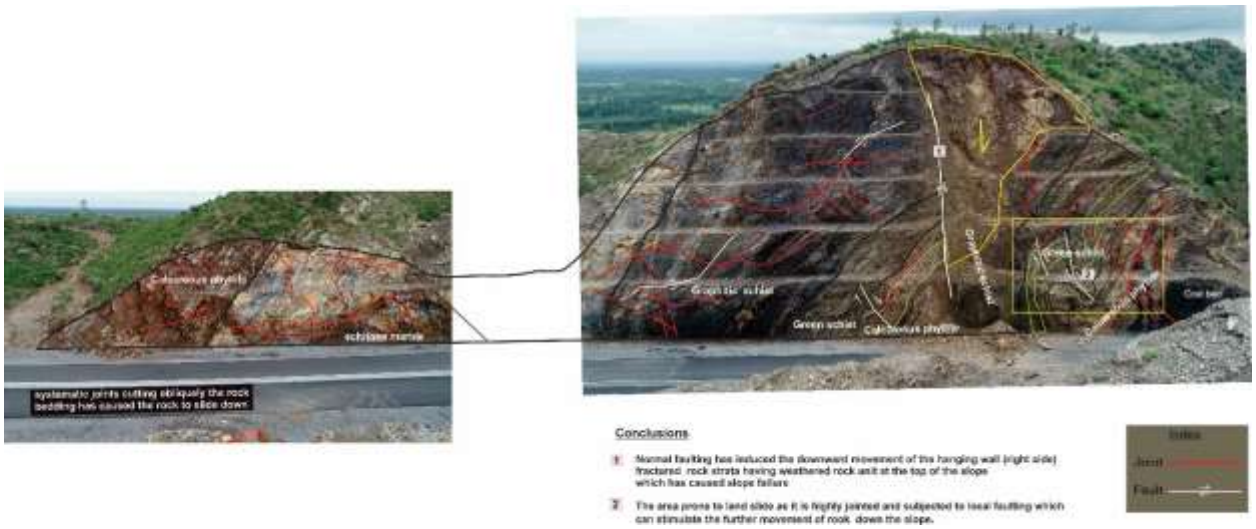


Figure 4-3 Geology of site (Geological Dept MCE)

Scanline Data for RD 37. (Geological Dept. MCE)

Location	Dip	Dip Direction	Type(Joint, Fault, Bedding Plane)	Rock Type(Marble, Schist)
37+001	82	65	J	M
1	48	332	J	M
1	60	135	F	S+M
1	35	85	J	S+M
6	70	143	F	M
6	18	128	J	M
6	84	251	J	M
6	85	266	J	M
6	80	145	J	M
11	50	95	J	M
11	70	68	J	M
11	19	300	J	M
15	75	145	B	M
16	40	98	J	M
16	82	42	J	M
16	39	70	J	M
16	32	15	J	M
18	69	140	B	M
23	65	238	J	M
23	28	85	J	M

23	88	280	J	M
24	84	266	J	M
25	55	82	J	M
28	28	118	J	M
28	50	50	J	M
28	85	263	J	M
28	55	10	J	M
28	70	140	B	M
31	68	80	F	S
31	48	70	J	S
32	74	350	F	S
32	65	136	B	M
35	70	255	J	M
37	45	18	J	M
37	40	40	J	M
37	65	138	F	S
39	64	248	J	S
40	75	250	J	S
45	65	345	F	S
45	80	265	J	S
46	48	15	J	S
46	75	261	J	S
48	62	359	F	S
51	80	246	J	S
51	56	55	F	S
58	82	106	J	S+M
58	45	141	B	S+M
58	54	138	B	S+M
62	70	50	J	S+M
64	18	120	J	S+M
64	65	136	F	S+M
64	63	65	J	S
64	54	130	J	S
65	66	20	J	S
66	66	144	J	S
68	65	252	J	S
74	68	139	F	S
76	87	86	J	S
78	60	140	F	S
81	38	80	J	S
84	34	85	J	S
85	85	155	J	S
88	55	278	J	S

88	80	255	J	S
88	63	140	B	S
89	78	266	J	S+M
91	84	257	J	S+M
91	55	142	B	S+M
91	80	260	J	S+M
95	56	148	F	S+M
98	45	20	J	M
98	15	60	J	M
102	60	140	B	M
102	75	340	FAULT	M
118	82	341	F	COAL
118	62	336	F	S
118	63	148	J	S
119	52	60	J	S
119	55	328	F	S
127	60	65	J	S
127	50	36	J	S
130	80	166	B	S
130	45	4	J	S
130	82	135	F	S
131	53	58	J	S
131	45	316	J	S
131	76	260	J	S
135	63	55	J	S
135	65	130	F	S
141	68	50	J	S+M
147	66	25	J	S+M
147	54	138	B	S+M
148	45	350	J	S+M
149	64	136	J	S
149	78	55	J	M
151	83	258	J	M
151	65	24	J	M
153	50	148	B	M
157	80	78	J	M
157	68	296	J	M
157	35	41	J	COAL
157	65	330	F	S
160	60	131	B	S
160	70	325	J	S
171	60	140	F	S
172	66	320	J	S

177	84	318	J	S
178	35	25	J	S
181	65	358	J	S
182	75	20	J	S
184	62	141	B	S
185	48	325	J	S
185	52	278	J	S
191	48	131	F	S
191	60	338	J	S
191	40	105	J	S
194	78	266	J	SS
194	61	15	J	S
194	55	142	B	S
201	82	257	J	S

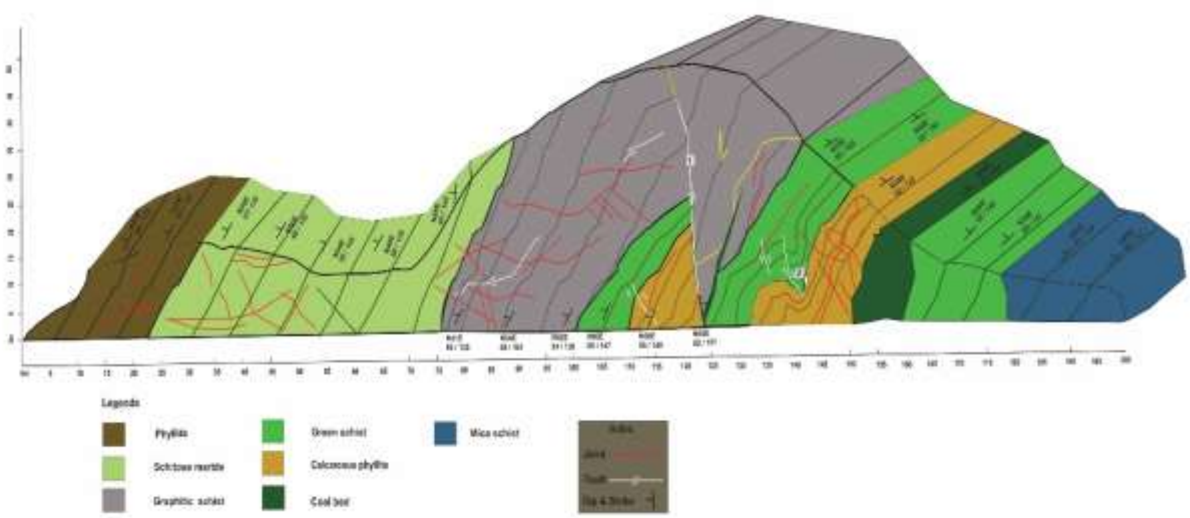


Figure 4-4 Geological mapping (Geological Dept MCE)

4.5.3 Stereonetic Analysis

Stereonetic analysis was run using DIPS software for complete section, Schist only data and Marble only data from RD 37+001 to RD 37+200. Furthermore, analysis was run for 0.25 slope and 0.5 slope.

Description	Plane Failure Probability **%**				
	Schist+Marble		Schist		Marble
37+001-37+200					
Site slope	16.67%		13.11%		20%
0.25 slope	15.83%		13.11%		17.5%
0.5 slope	10%		8.2%		15%

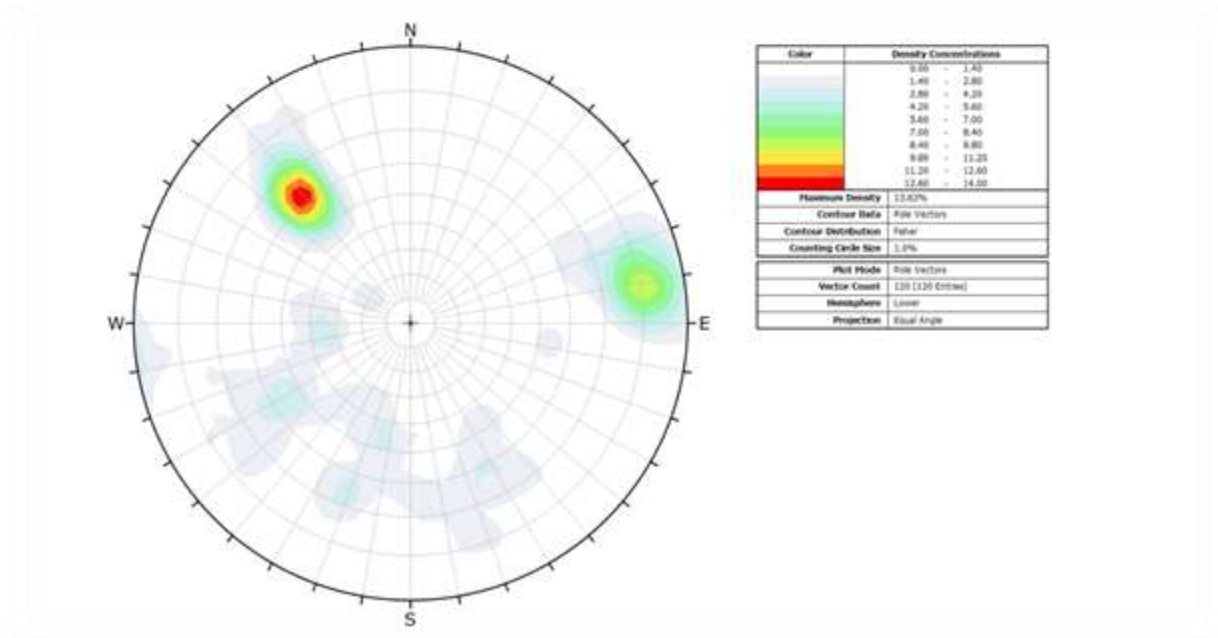


Figure 4-5 Contour Plot complete section

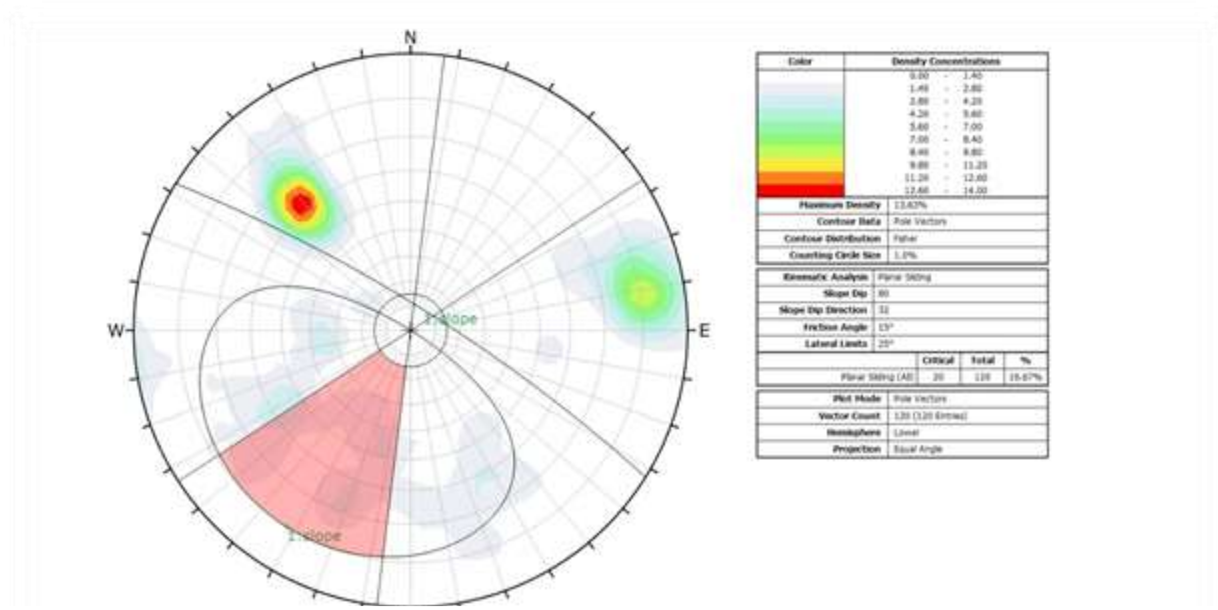


Figure 4-6 Kinematic analysis complete section

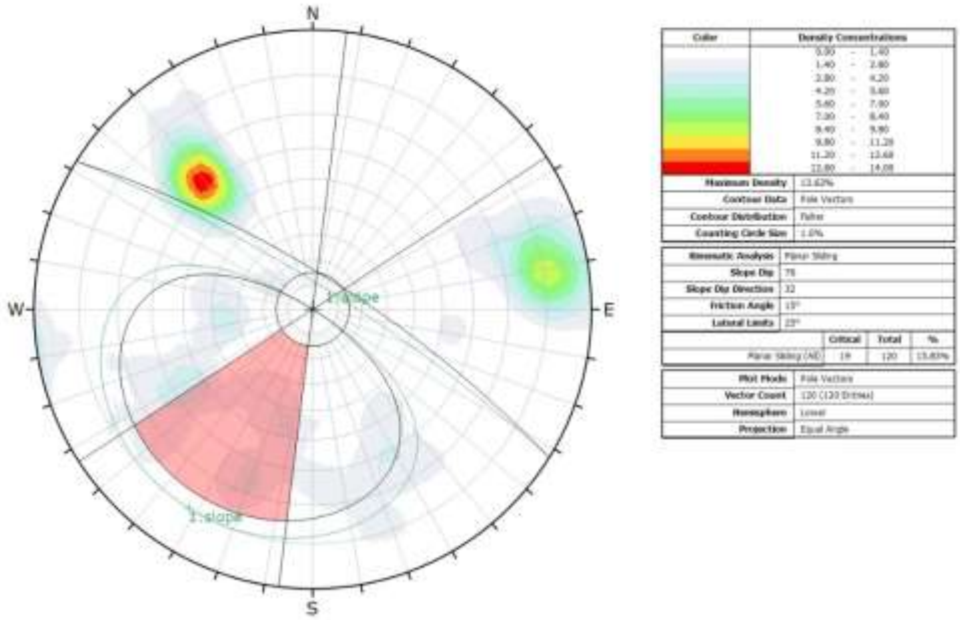


Figure 4-7 Kinematic analysis at 1:0.25 slope

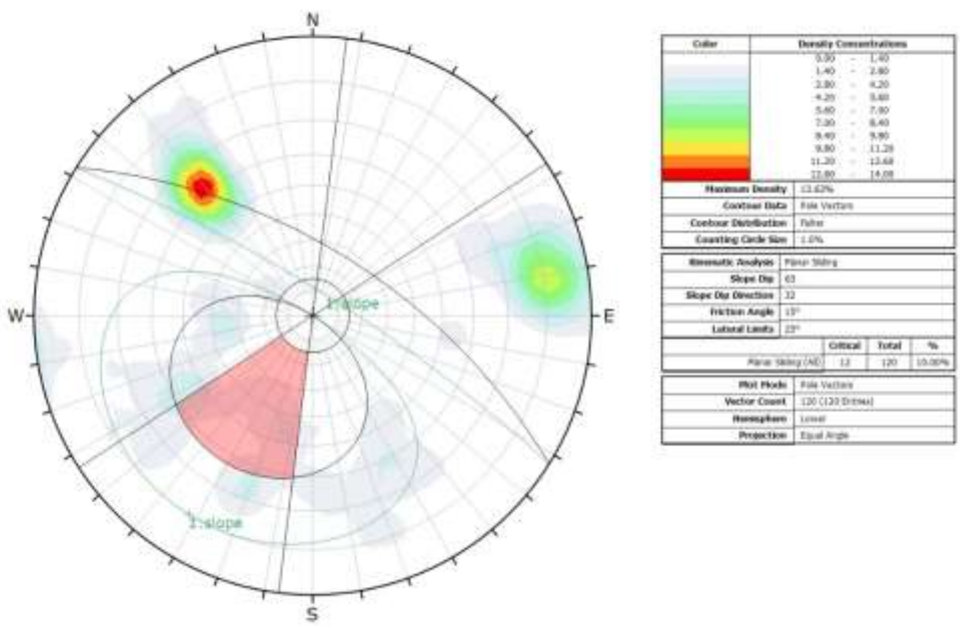


Figure 4-8 Kinematic Analysis at 1:0.5 slope

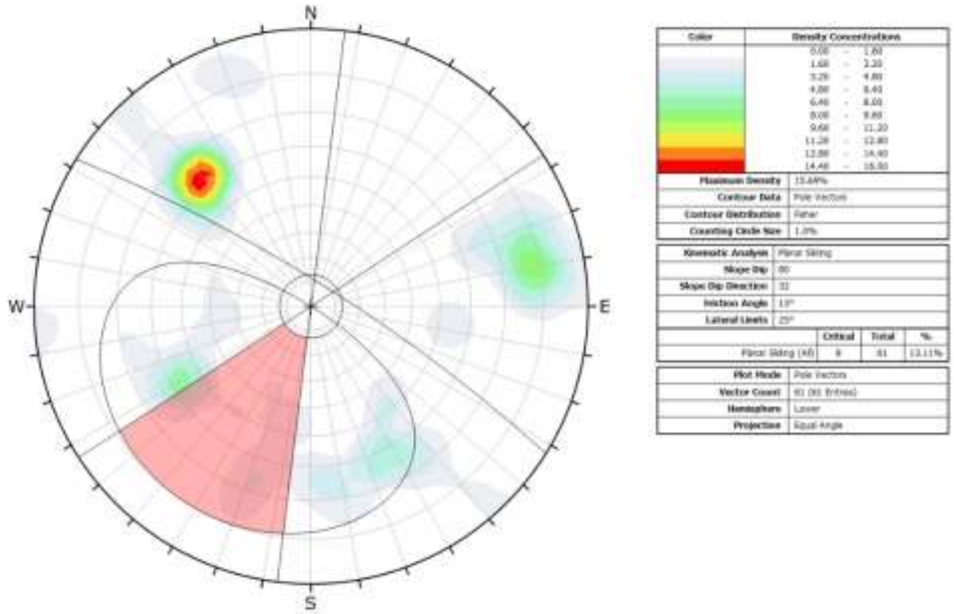


Figure 4-9 Contour plot complete section - schist

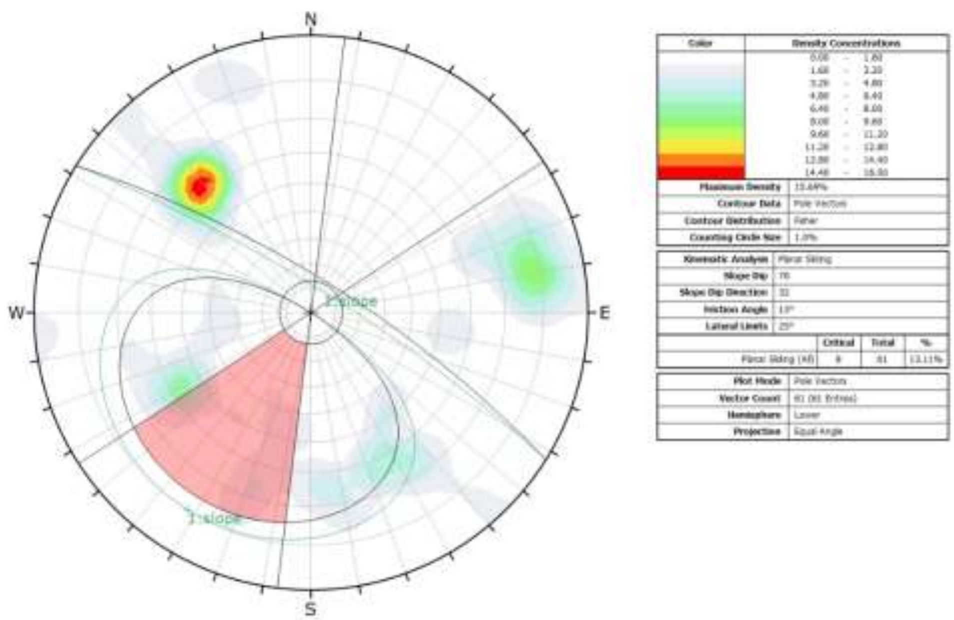


Figure 4-10 Kinematic Analysis at 1:0.25 slope - schist

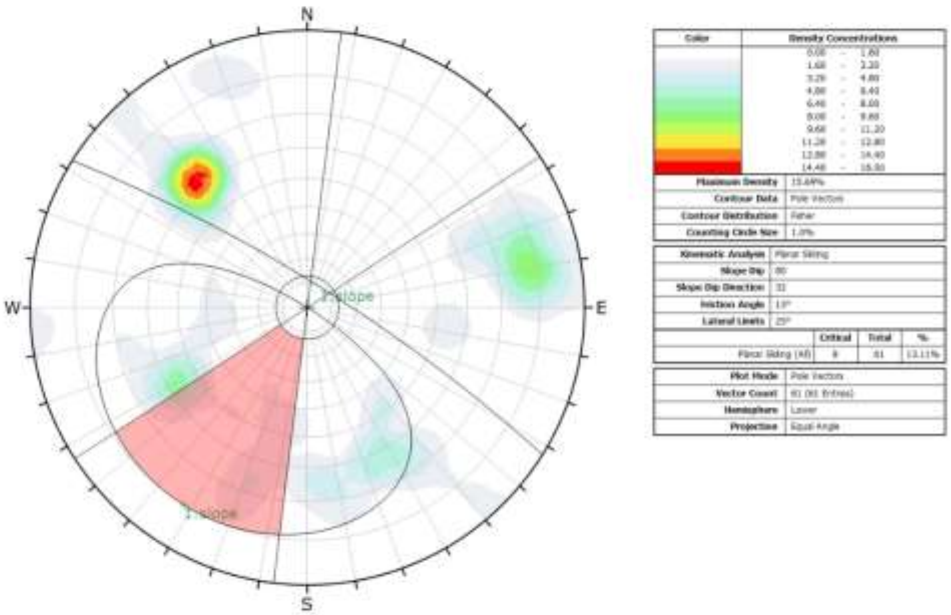


Figure 4-11 Kinematic Analysis complete section - schist

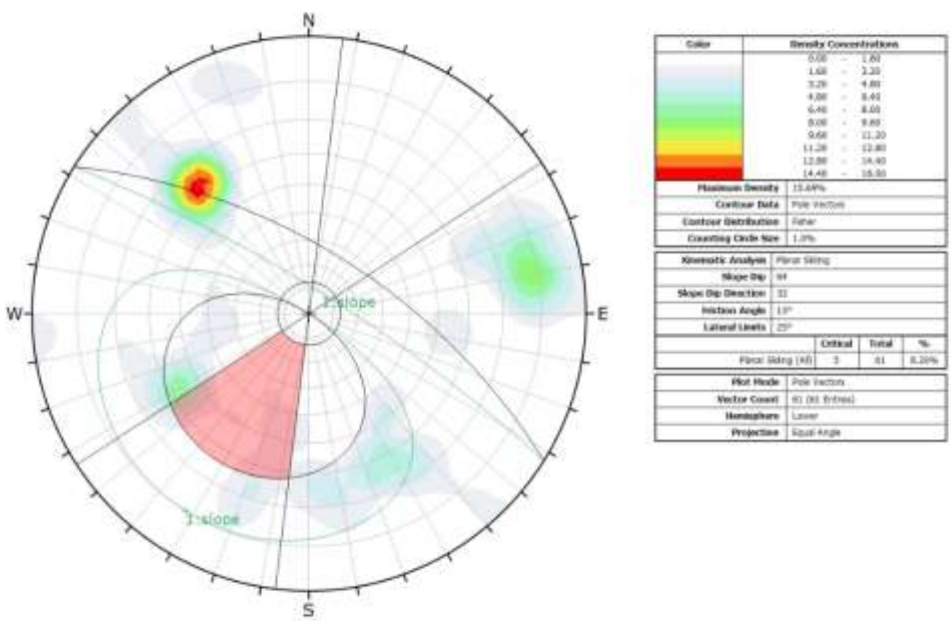


Figure 4-12 Kinematic Analysis at 1:0.5 slope schist

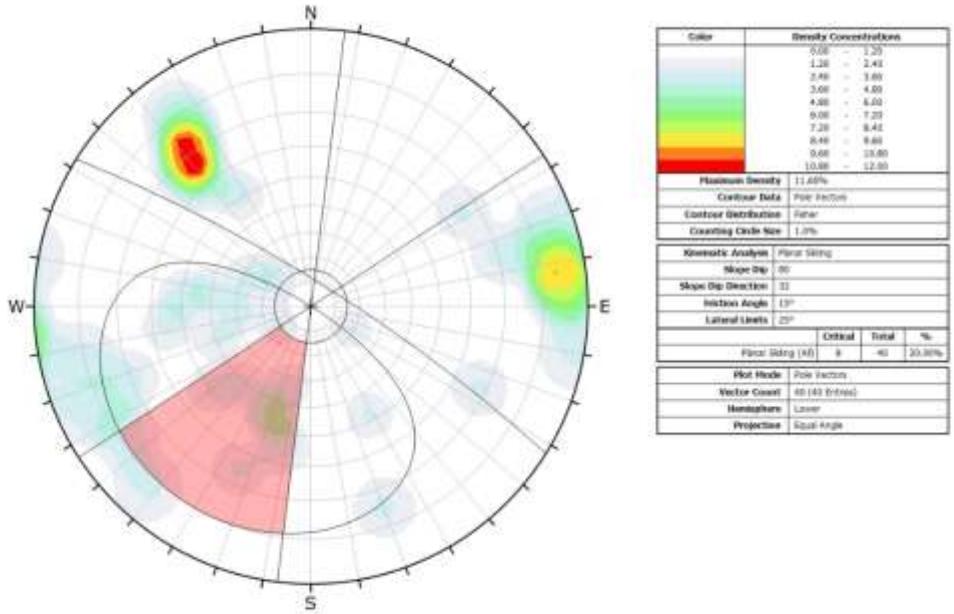


Figure 4-13 Contour plot complete section - marble

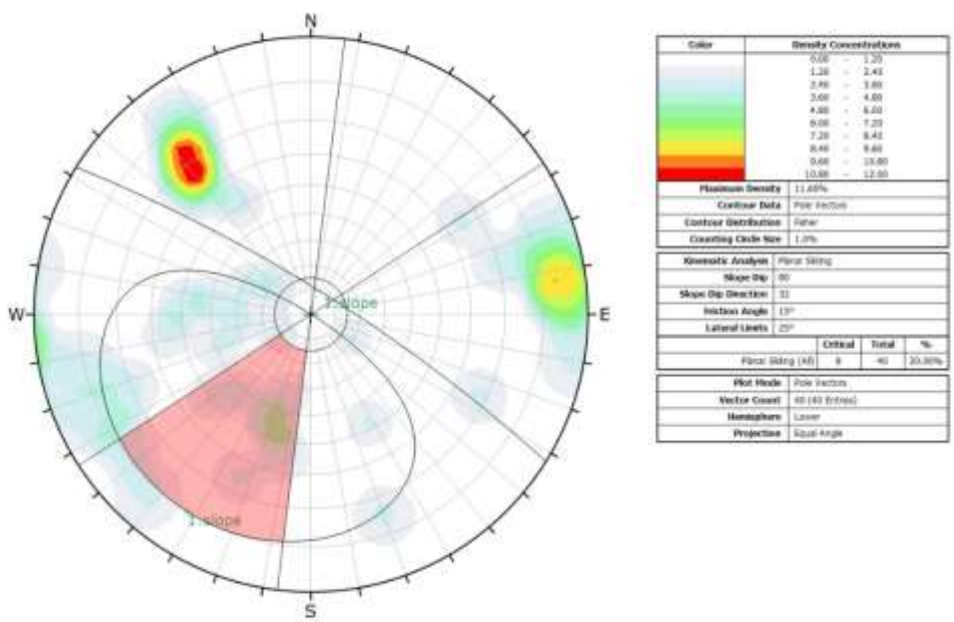


Figure 4-14 Kinematic analysis complete section - marble

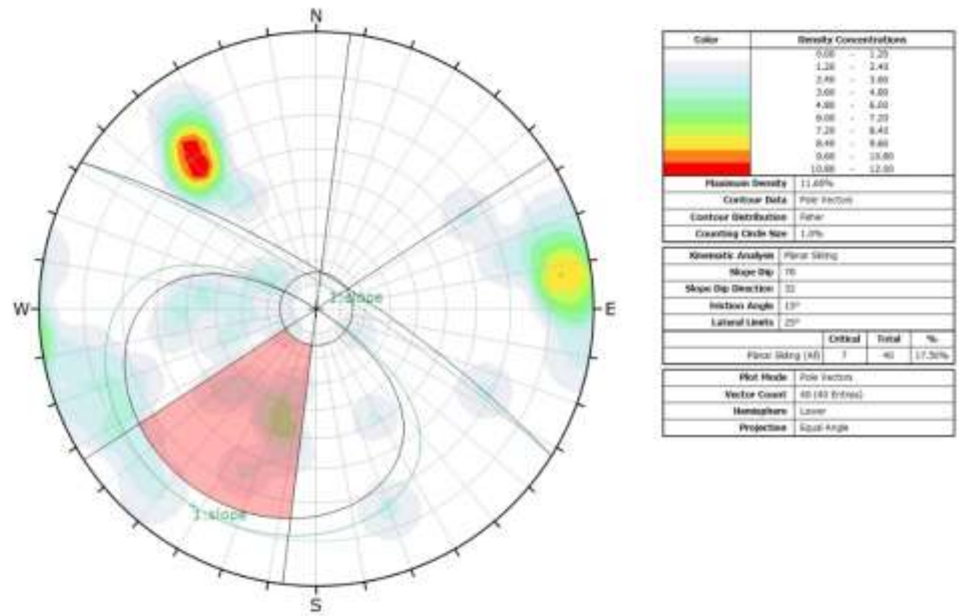


Figure 4-15 Kinematic Analysis at 1:0.25 slope marble

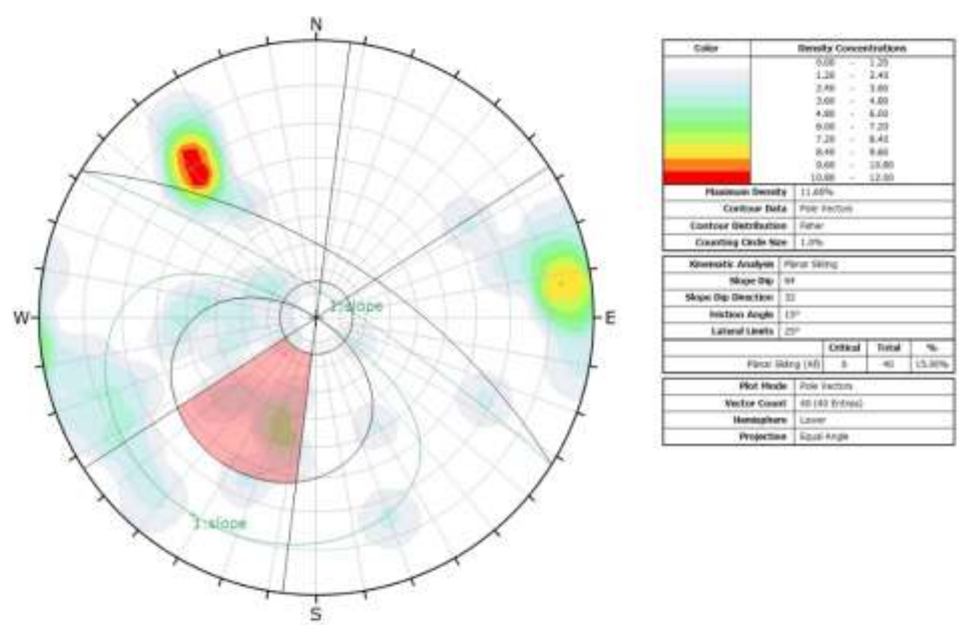


Figure 4-5 Kinematic Analysis at 1:0.5 slope marble

4.6 Earthquake Analysis/Seismicity

RD 37 lies in Zone “2B” with reference to seismic zoning map of Pakistan. Zoning is based on PGA ie Peak Ground acceleration values. 0.16-0.24g is the range of peak horizontal ground acceleration.This PGA value can result in landsliding at site.

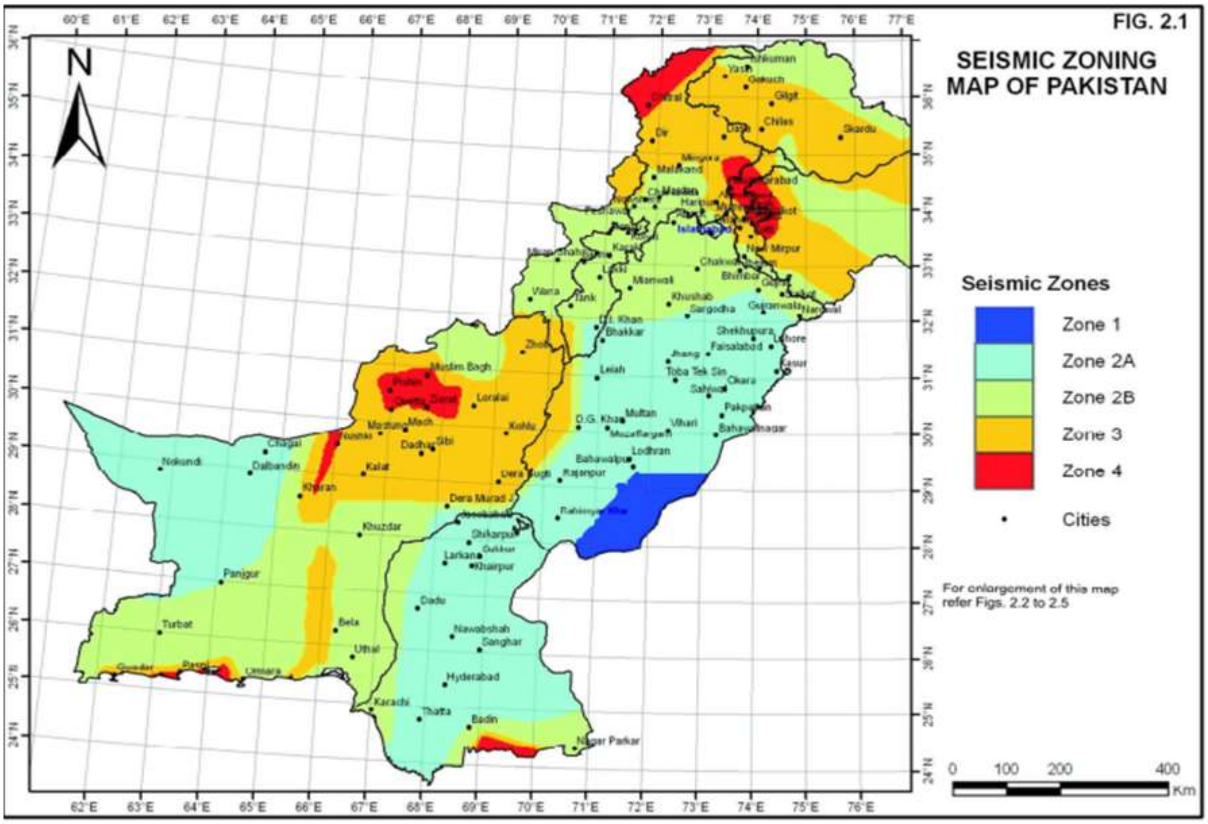


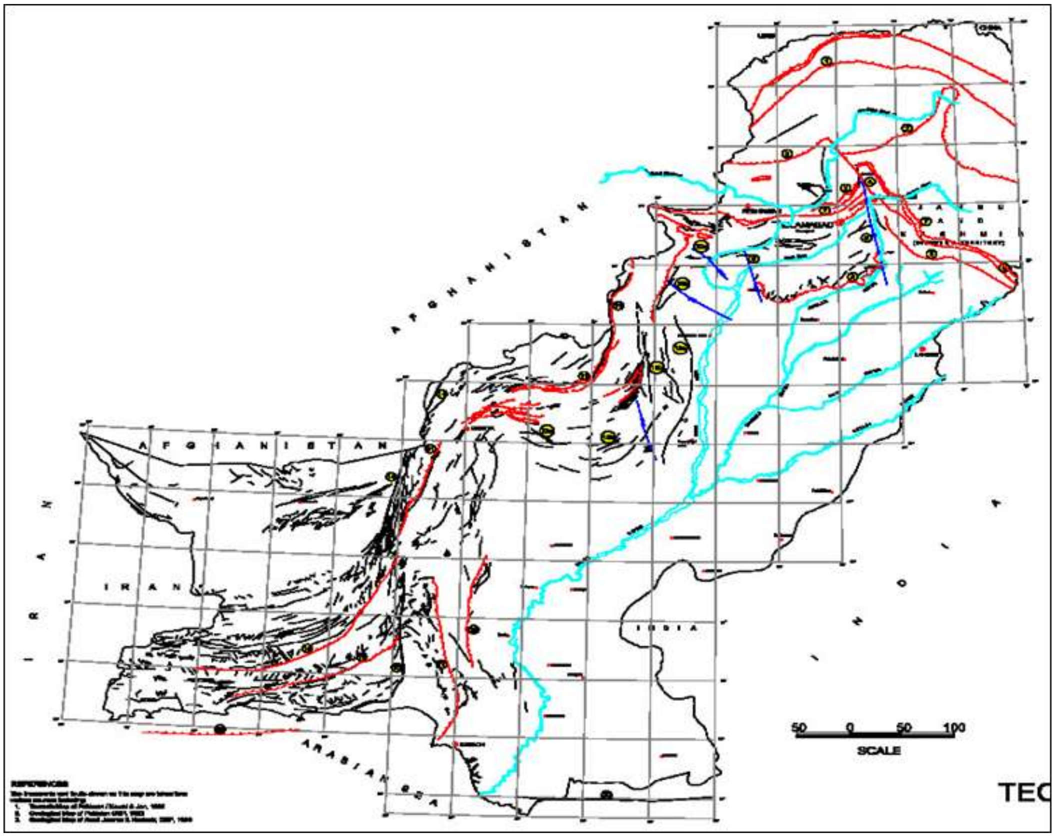
Figure 4-6 Seismic Map of Pakistan

Seismic Zone	Peak Horizontal Ground Acceleration
1	0.05-0.08g
2A	0.08-0.16g
2B	0.16-0.24g
3	0.24-0.32g
4	> 0.32g

4.7 History of earthquakes

From the above figure we can conclude the past trends of earthquake that had been occurred in the past. The earthquake intensity of swat region varies between 4.5 to 6.5

on Richter scale. These lateral movements of earth crust cause the rockfall in this region from the mountains. However, the frequency in this region is not much high but due to the strategic importance of the region any natural calamity is vulnerable. The last earthquake in swat valley was recorded on 2nd April 2018 of magnitude on 5.5 on Richter scale. Other these earthquake mild lateral movements of ground that cannot be recorded on the Richter scale causes the landslide in the region.



4.8 Weather conditions

There are four climatic zones into which the world has been divided. (James Peterson, 2010).

Zone	Altitude
Tropical	0-23.5°
Subtropics	23.5°-40°
Temperature	40°-60°
Cold	60°-90°

4.9 Climatic zones

Pakistan has been divided into 5 zones i.e A,B,C,D, and E. swat valley lies the zone A due to its climatic conditions. During winters it experience long lasting rainfall as compared to the summer season. The peak temperature during summer has been recorded as 35 Celsius.

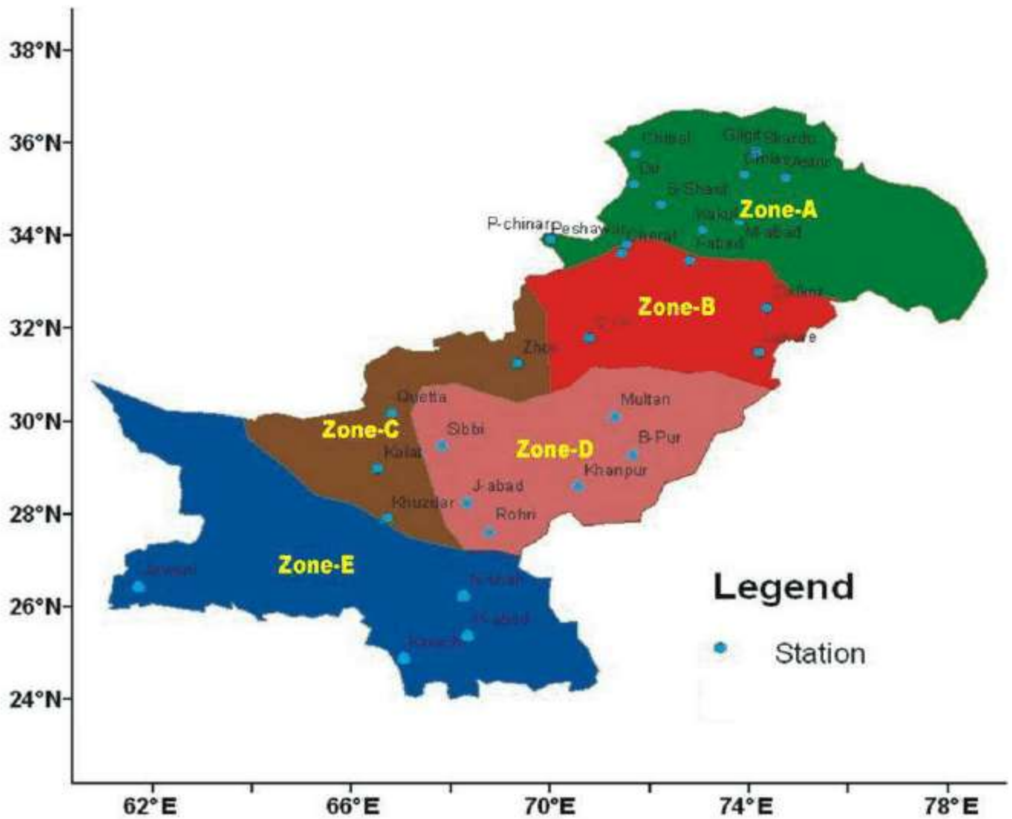


Figure 4-20 climatic zones of Pakistan

4.10 Temperature

- June is the warmest month of the year. The temperature in June averages 33.2 °C.
- In January, the average temperature is 10.0 °C. It is the lowest average temperature of the whole year. (Climate Data, n.d.)

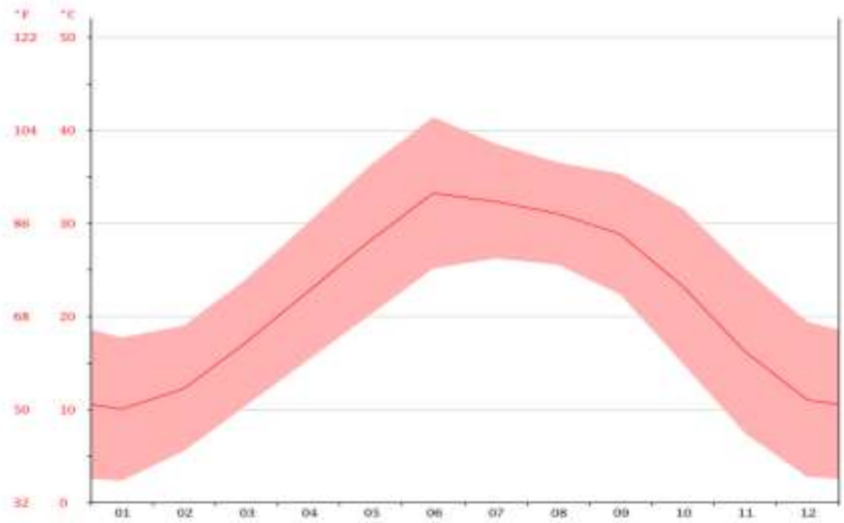


Figure 7 Temperature Data

4.11 Precipitation

In the above graph the precipitation data is also shown from this data we can conclude the peak precipitation in site region is about 122mm in August. From the chart shown above we can also conclude that in swat rainfall intensity is quite high. Due to the intensity the rocks are exposed to alternate wetting and drying which causes the rock to weathered quickly.

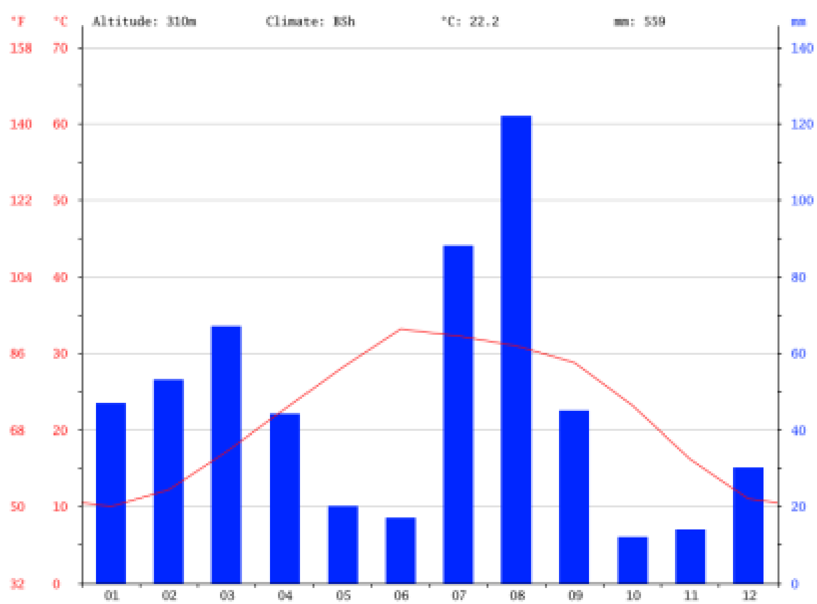


Figure 8 Precipitation data

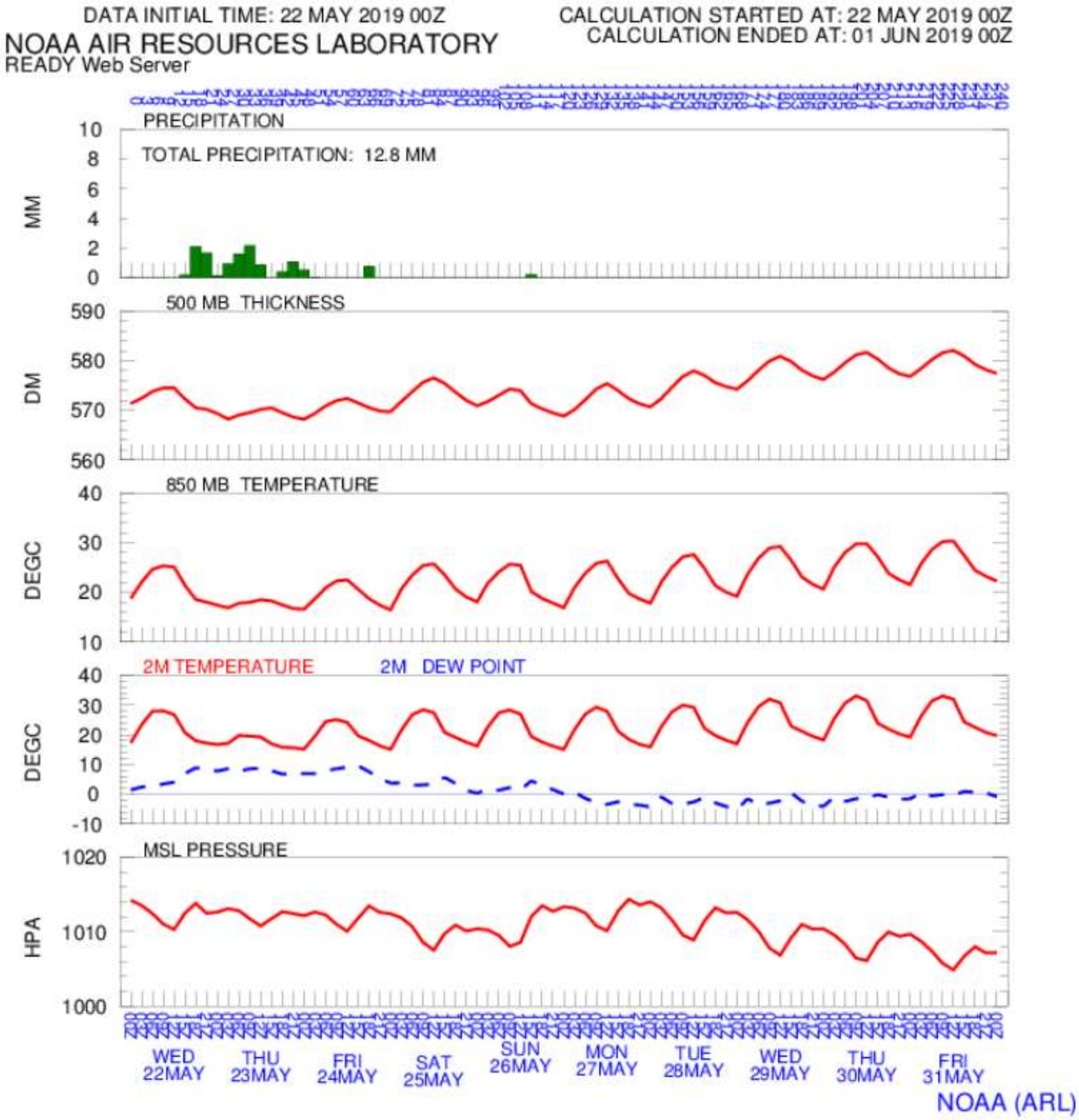


Figure 9 Precipitation Data

4.12 Lab Investigation

4.12.1 Rock coring

Rock cores are very essential to perform any rock test because the intact rock itself are very large to fit into any equipment so the cores from the intact rocks are extracted. These rock cores can be extracted on site or can be extracted in the laboratory. We extracted the core cores in the laboratory we collected the intact rock samples from the site and obtained the cores from these samples. The dimensions of the rock cores were 1.5” in dia and 3” in length. RD 37 contains 5 different types of rocks we obtained the 3 cores core for each rock type excluding coal (one of the rock on RD-37) because of the clayey nature of the coal cores couldn’t be extracted.

Figure 4-21 Rock Cores

Figure 4-22 Rock core extraction

4.12.2 Density Tests

Density of the rocks were calculated by performing different density tests on the rock samples. Flask was filled with water containing the rocks. Volume of the rocks were computed and then arithmetic calculations were performed to find out the rock density.

Rock Type	Density (g/cm^3)
Marble	2.61
Schist	2.75
Calcareous phyllite	2.73

4.12.3 Uniaxial compression test

Uniaxial compression test is one of the most important tests to find out the geo mechanical properties of the rock sample. UCT was performed on the rock core samples that were extracted from the rock samples in laboratory A rock core sample is cut to length and the ends are machined flat. The specimen is placed in a loading frame and, if required, heated to the desired test temperature. The specimen is placed in a triaxial compression machine which was digitized using load cell, data acquisition device (system 8000). Axial load is continuously increased on the specimen and digitized data was obtained for load, axial strain and lateral strain until failure. (ASTM, 2014) (ASTM, 2014)

Figure 4-23 Uniaxial compression test

Types and modes of failure of sample during testing indicate the degree of hardness & brittleness of rock. A number of characteristics can be interpreted about the sample tested. Under uniaxial compression, rock sample fails in a sudden manner & fracture planes are distinctive. For strong rock like gabbro (igneous), crushed material (dust) and fragmentations is very minimal.

For softer rock like shale (sedimentary rock), failure is gradual and fragmentation & powdered rock is more obvious. Fracture planes is less significant. Brittle & hard materials like rocks – the strain at failure is relatively smaller & the stress at failure is higher. Mode and shape of failure is shown in the figure below. UCS of Marble was calculated to be 110 MPa and that of Schist was found to be 15MPa.

Figure 4-24 Core under UCT

4.12.4 Schmidt / Rebound Hammer test

Test on surface hardness of rock sample using Schmidt hammer (L-type), a portable & simple equipment to handle. Sample can be core or block. Test is nondestructive & sample can be re-used. Index data obtained is rebound number (R). The stronger is the surface the higher is the R value. R is related to the surface strength (JCS) of rock sample tested:

Log10 JCS = 0.00088 (g) (R) + 1.01 (Broch & Franklin, 1972).

Rock Type	Rebound Value	Compressive strength Range (Mpa)
Marble	52	110
Schist	35	15
Calcareous Phyllite	36	17

*These values are not accurate as rebound hammer test does not give the accurate values.
*These values are calculated by the average of 5 test values of one rock sample.

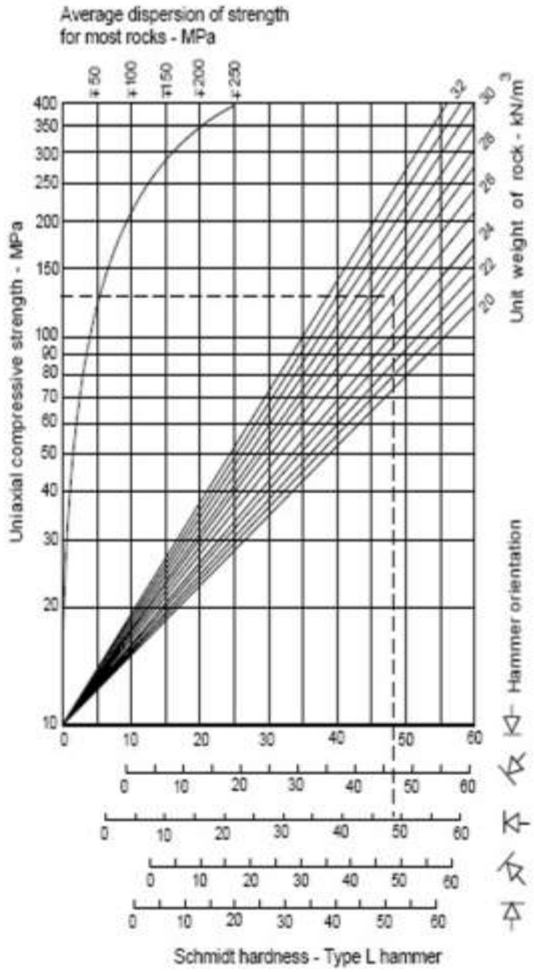


Figure 4-24 Schmidt Hammer testing

4.12.5 Young Modulus & Poisson Ratio

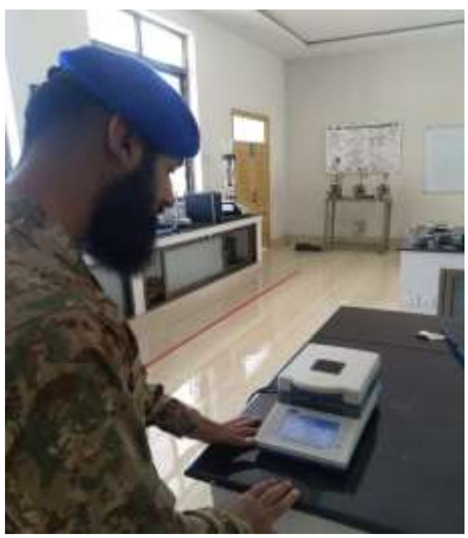
The resistance of rock core against the compression force applied on it is defined as young modulus. It tells us about how the rock will behave under certain compressive load. During the compression testing we applied the strain gauges un the rock core which gave us the strains in the rock core. We applied 2 strain gauges on one rock core. One strains gauge is vertical and one is radial. Both strains were recorded by data acquisition device i.e. system 8000. Then by dividing the radial strains by vertical strain we found out the poison ratio. Due faulty rock cores the results were not accurate so we didn't use the results of modulus from this test instead we performed sonic wave test.

Figure 4-25 Data acquisition device

Figure 4-26 Vertical and Horizontal strains Gauges

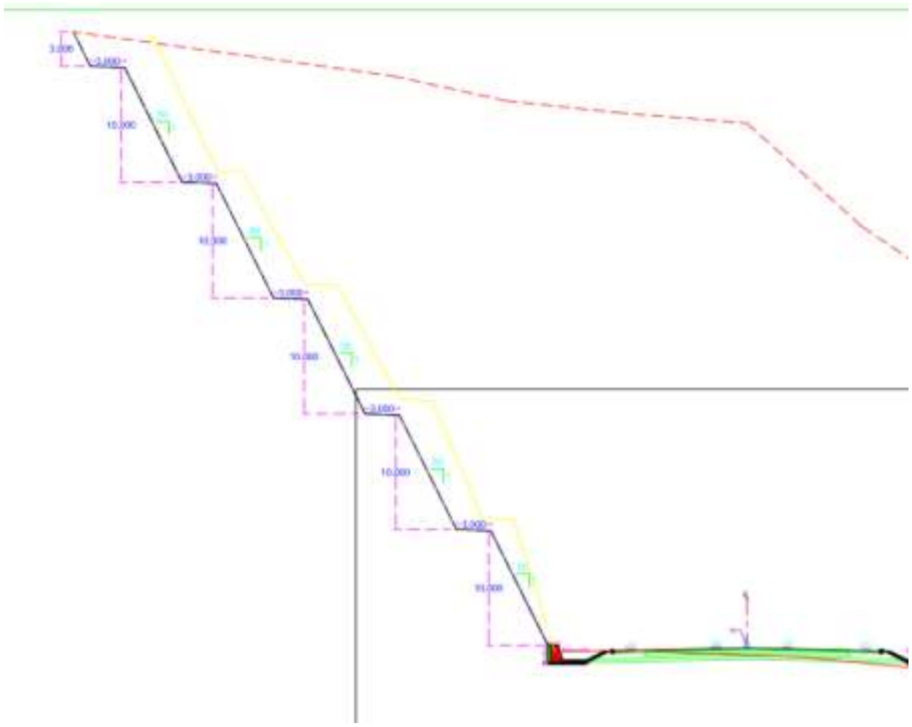
4.12.6 Moisture content test

Moisture content tells us about the percentage of moisture present in the rock. From the moisture content we can predict about the weathering behavior of the rock. We calculated the moisture content in the lab via using latest digital apparatus present in the soil lab. The moisture was found to be .14% to .16% almost for all types of rock.



4.13 Geotechnical profile

The geotechnical profile of any soil profile is given by the geological, hydrological, seismological studies, geological mapping, scanline survey, along with the field testing, lab investigation, and some other physical parameters. The geotechnical profile is given by the above-mentioned parameters.



4.14 Conclusion

The major causal factors that triggered the rockfall along swat motorway at site RD-37 were revealed from the following studies.

- Strata is highly folded, jointed and foliated (rock weathering)
- Open joints with seepage condition were observed which makes site highly susceptible to sliding
- Weak interbedded Schist in Marble also contribute to sliding
- Non engineering cut is made which is main reason to sliding
- Local fault was observed which has contributed to slide
- The temperature changes all over the year
- The change in weather condition all over the year
- Seismic activities happening in the area

CHAPTER - 5
SLOPE STABILITY ANALYSIS AND REMEDIAL MEASURES

5.1 General

When the downwards movement of strata under action of gravity due to gravity and shear stresses exceeds shear strength, slope failure occurs. A slope can be remedied through two major methods:

- Rock slope Protection
- Rock slope Stabilization

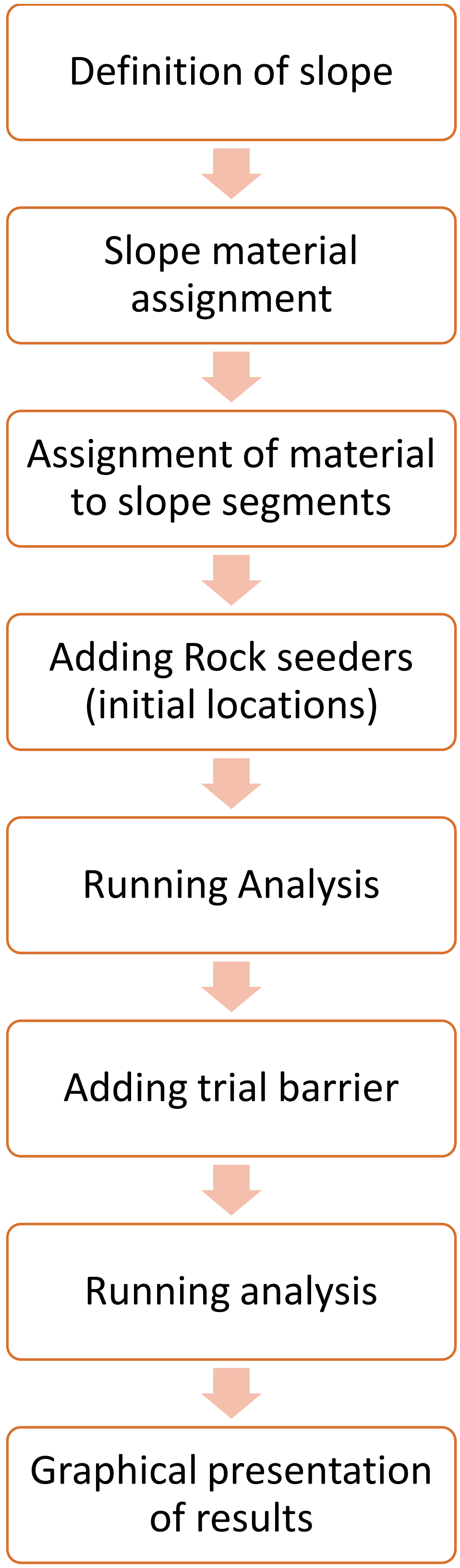
Slope cannot be stabilized without evaluating the geotechnical properties, site hydrology, geology and seismology. Required parameters have been calculated/estimated in previous chapter. This chapter will discuss rockfall barrier design (Protection measure) using Rocfall software and slope stability analysis in static conditions and under seismic loading using SLIDE software. Furthermore, we will suggest stabilization measures for the slope instability namely rock bolting through SLIDE.

5.2 Protection Measures

5.2.1 Rocscience ROCFALL

ROCFALL is a statistical analysis program that assists with the risk assessment of slopes i.e. the risk of rockfall at slopes. It determines energy, bounce height envelopes and velocity for the input slope. Remedial measures can also be determined through ROCFALL especially barrier design. It gives us the trajectory of the rockfall. It also gives us the impact location and kinetic energy of rocks striking barrier which helps us determine the strength of barrier, its size and location through trial and error.

5.2.2 Algorithm for Rockfall barrier design



5.2.3 Rockfall simulations and graphical intrepretations

All simulations were run on cross section of RD 37 + 80. The rockfall trajectory was calibrated with actual rockfall test conducted on ground. Only then has the barrier been designed. 100 cycles were performed for each rock.

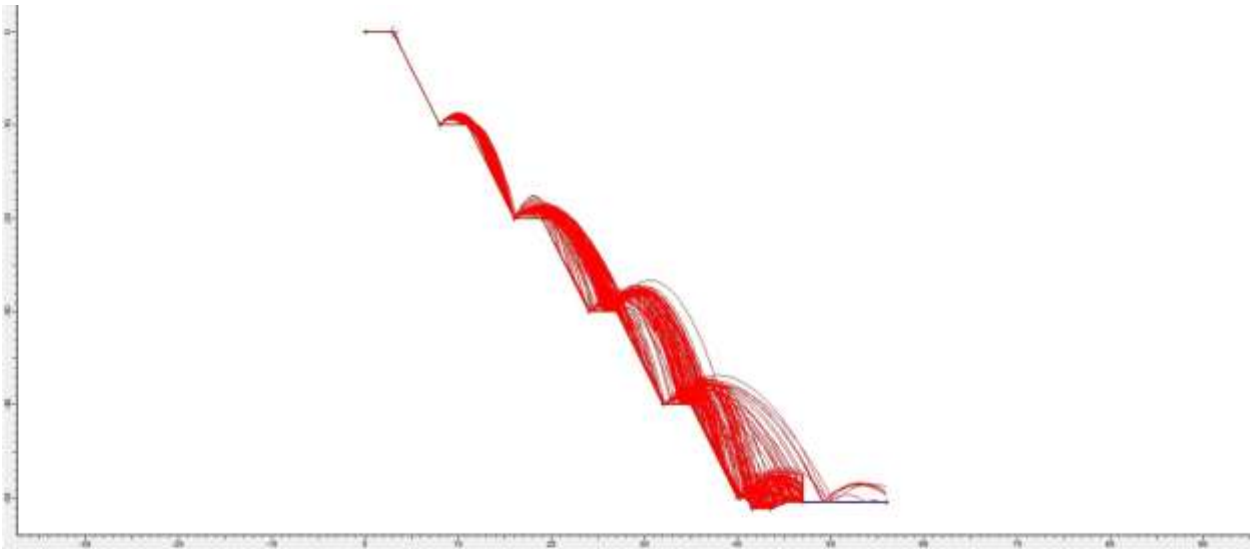


Figure 5-1 Rock 1 simulation

Rock 1 was 20 kg and fell from the fifth bench and some rocks crossed the barrier while majority were stopped by it.

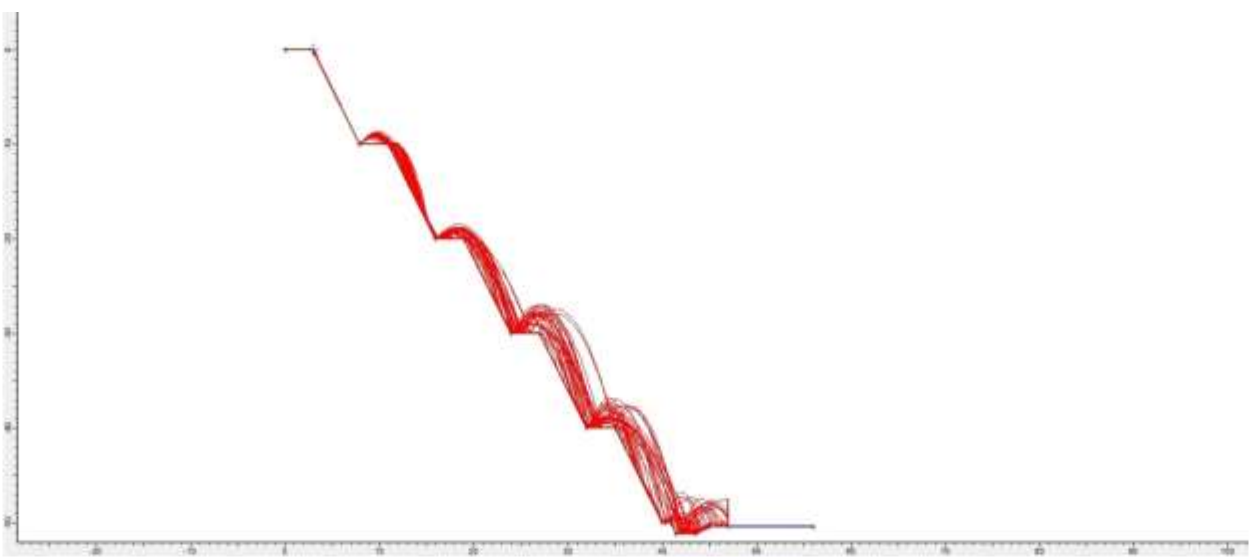


Figure 5-2 Rock 2 simulation

Rock 2 was 50 kg and fell from the fifth bench and fell just before the barrier with some striking it.

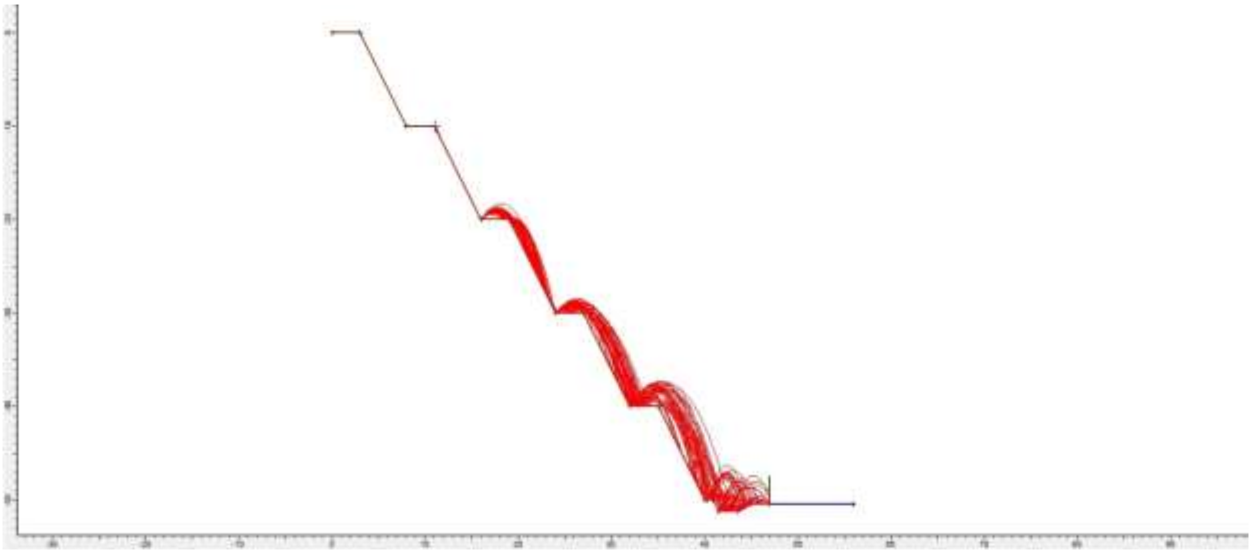


Figure 10 Rock 3 simulation

Rock 3 was 30 kg and was fell from the fourth bench. Only some rocks touched the barrier.

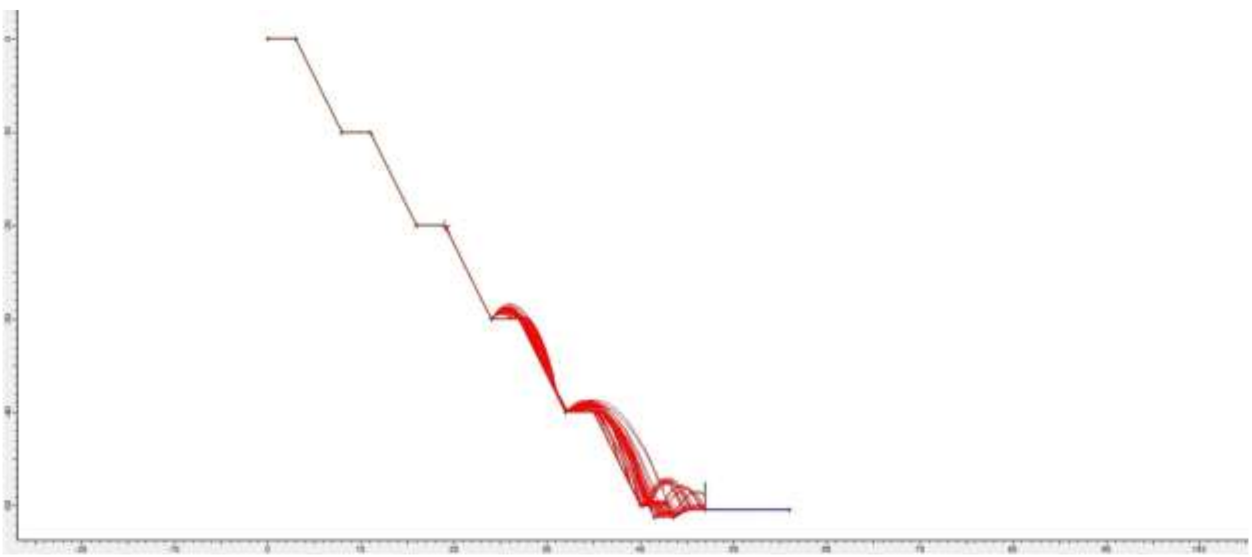


Figure 5-5 Rock 4 simulation

Rock 4 was 30 kg and all rocks simulated were stopped by barrier.

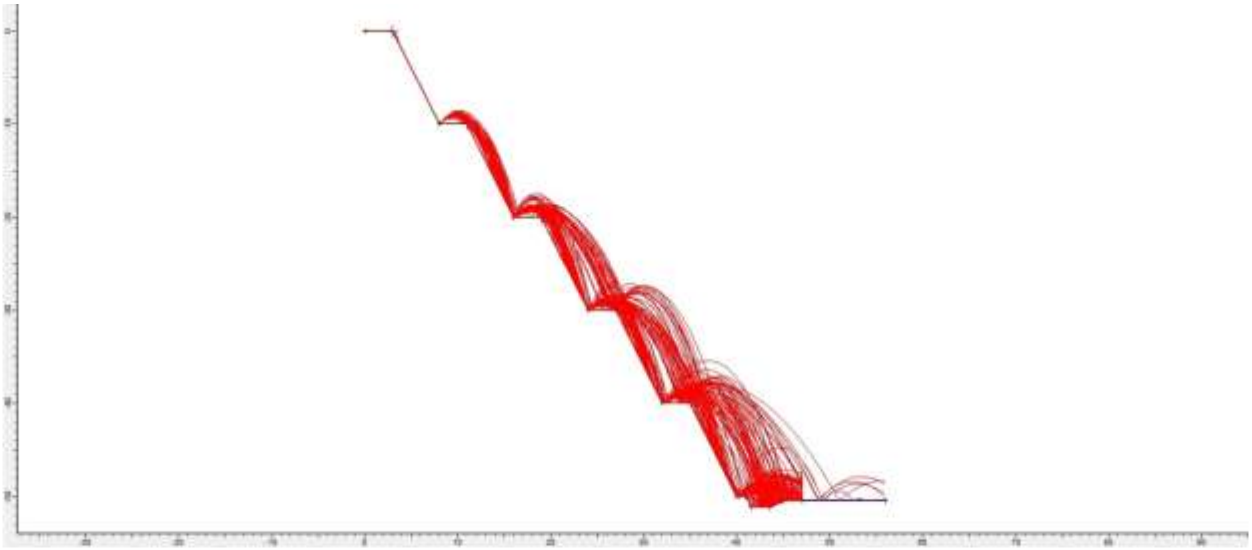


Figure 5-6 Complete simulation of all rocks

The following graphs are the cumulative analysis of all rocks simulated.

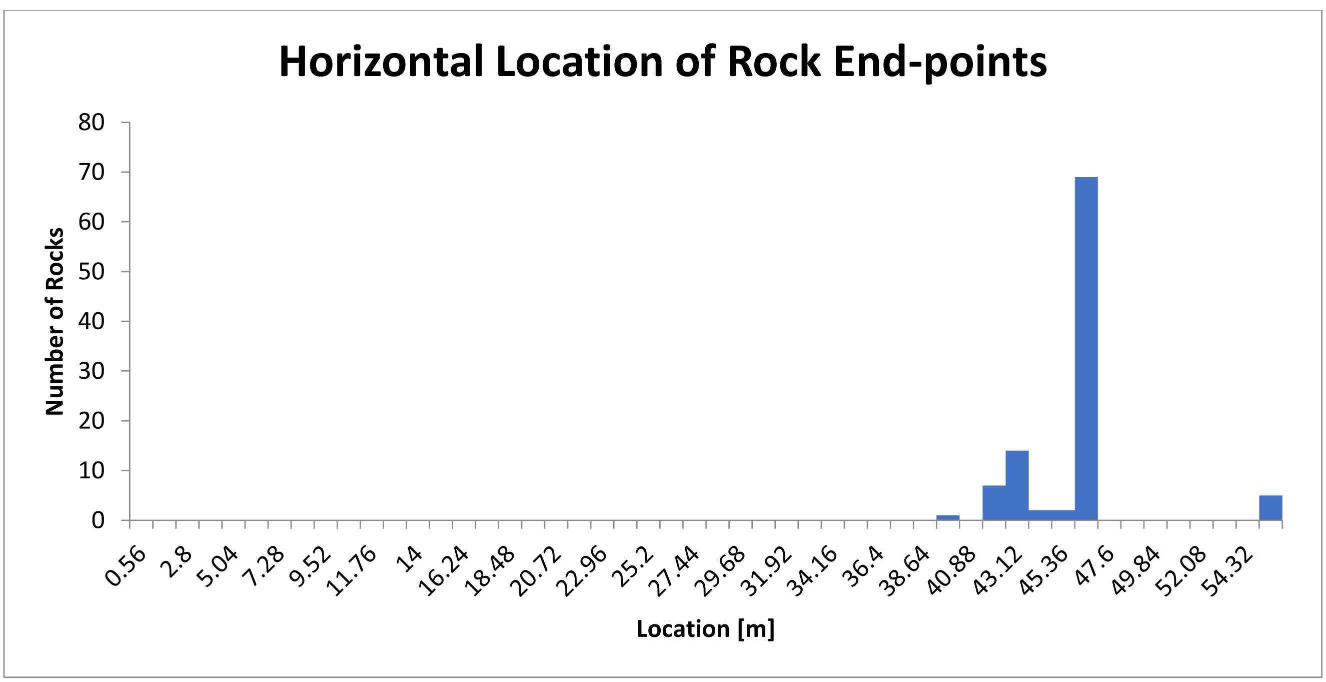


Figure 11 Horizontal location of Rock End-points

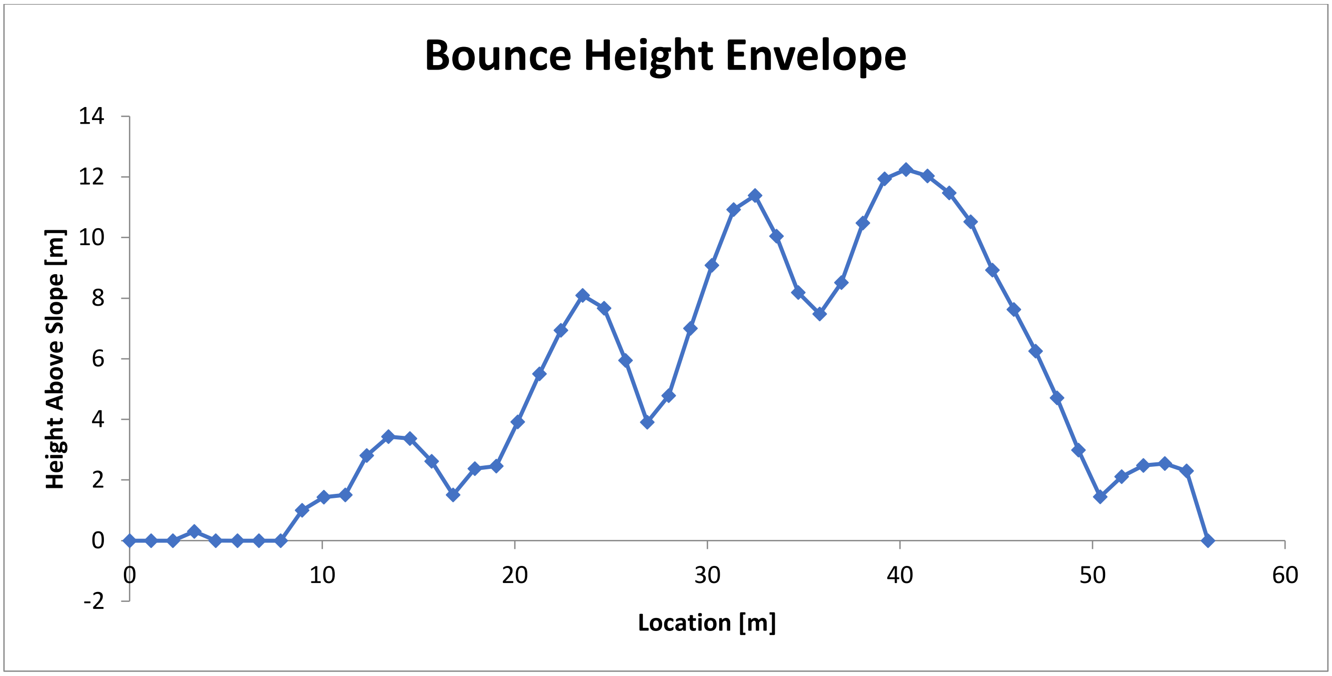


Figure 12 Bounce Height Envelope

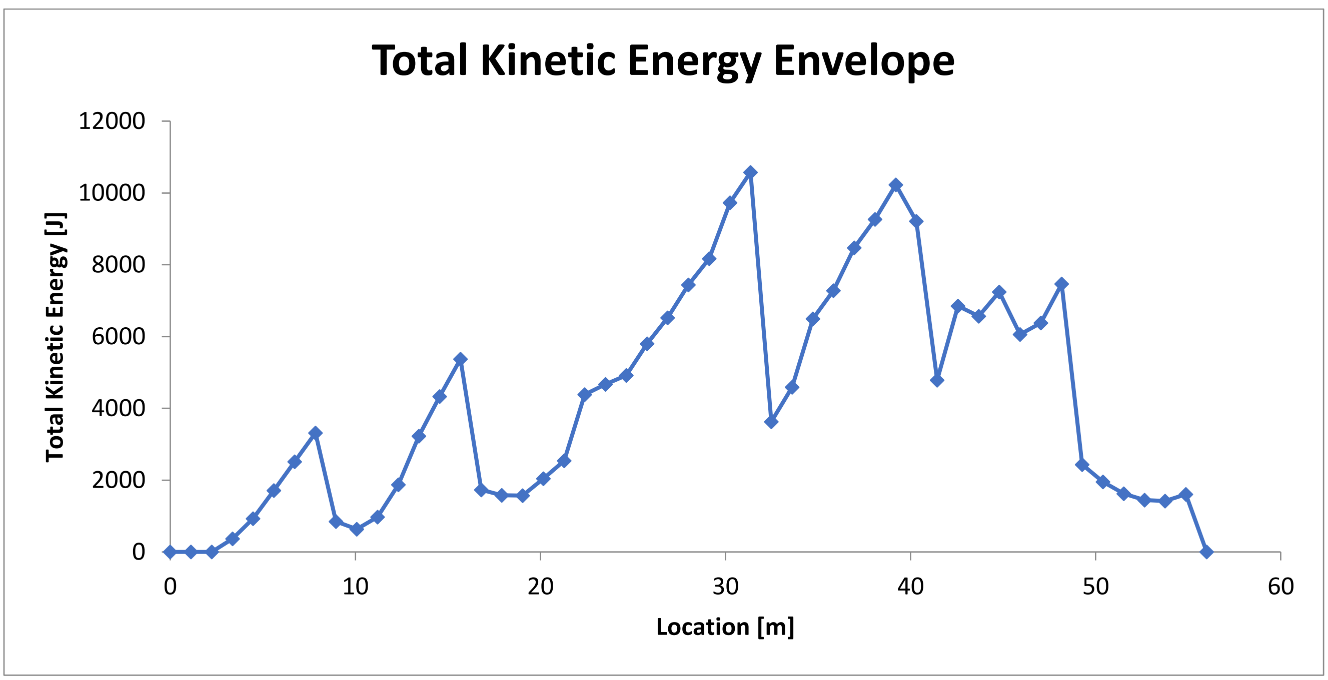


Figure 13 Total Kinetic Energy Envelope

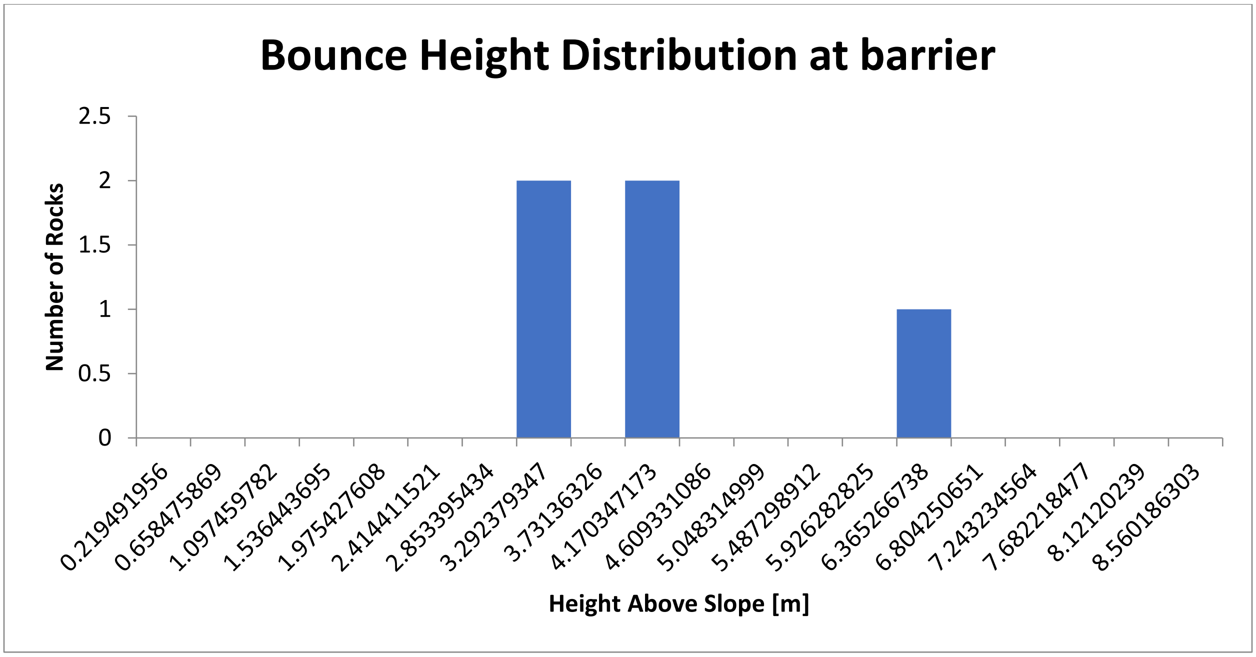


Figure 14 Bounce Height Distribution at Barrier

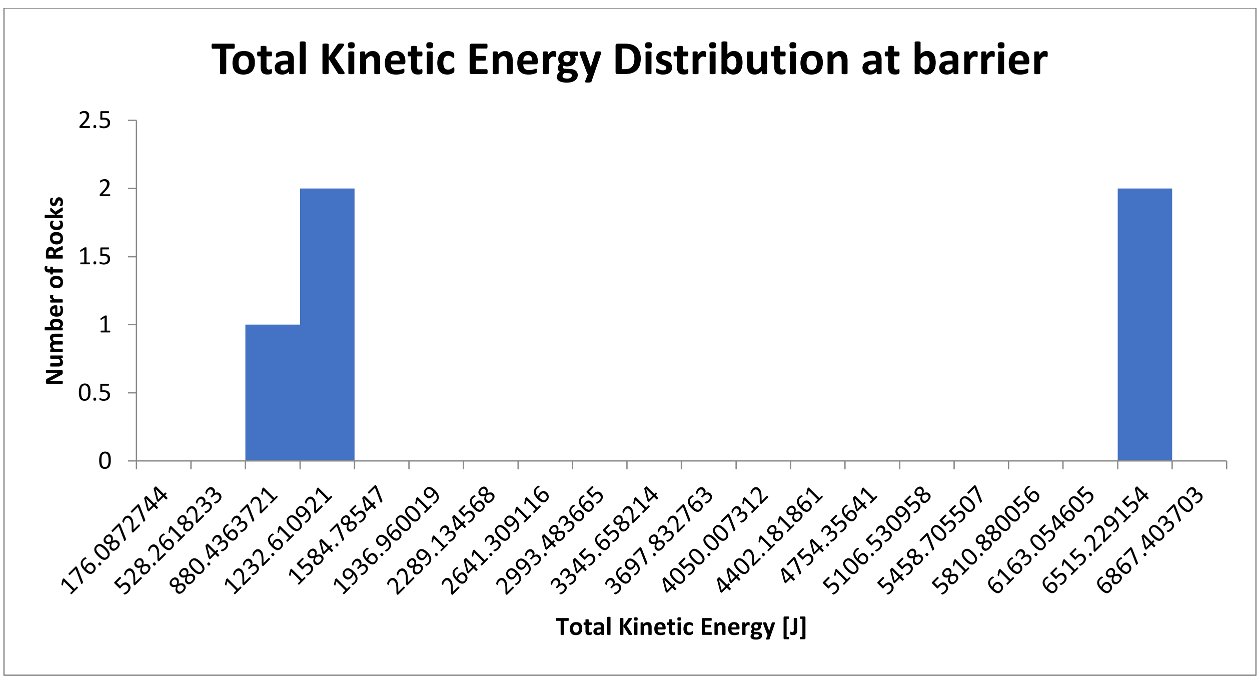


Figure 15 Total kinetic energy at barrier

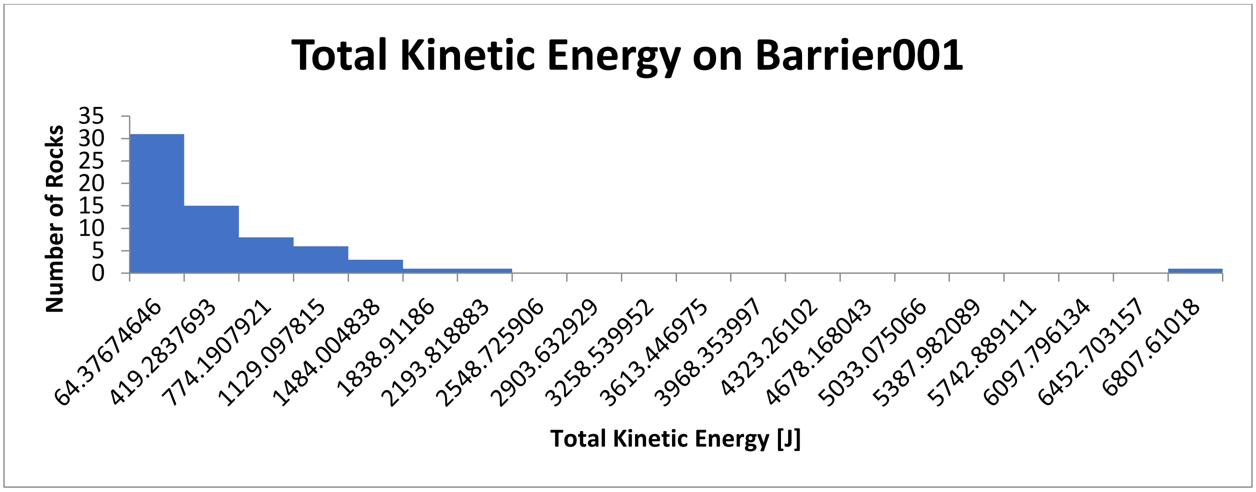


Figure 16 Total kinetic energy at barrier

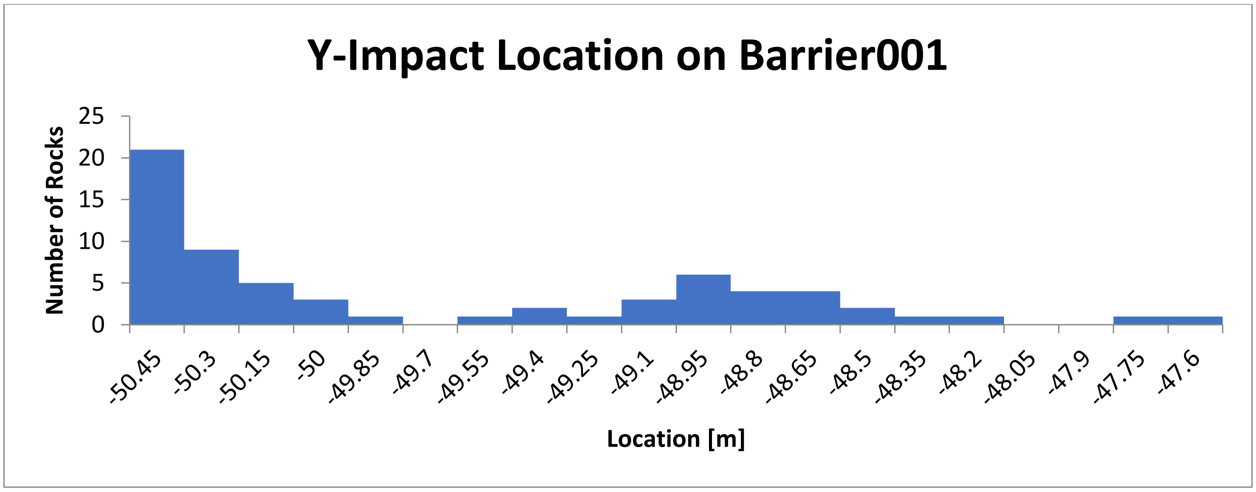


Figure 17 Y-Impact location at Barrier

X Coordinate	Y Coordinate	Translational Velocity [m/s]	Rotational Velocity [rad/s]	Total Energy [J]
47.00	-50.14	2.50	16.02	128.58
47.00	-50.43	5.69	48.86	213.91
47.00	-50.08	2.10	11.36	151.75
47.00	-50.36	8.64	46.00	1406.55
47.00	-48.62	3.37	22.36	238.26
47.00	-48.36	4.36	25.53	373.73
47.00	-50.43	2.13	13.28	92.11
47.00	-48.62	4.68	23.63	404.08
47.00	-48.89	5.52	33.15	607.28
47.00	-49.88	6.68	21.73	733.57
47.00	-48.35	5.05	23.96	821.46
47.00	-50.37	3.52	20.31	241.48
47.00	-50.12	6.34	22.59	671.52
47.00	-50.35	2.83	18.37	166.30
47.00	-50.33	3.22	18.29	200.89
47.00	-50.45	2.14	14.14	95.67
47.00	-50.20	2.67	11.18	218.00
47.00	-50.27	5.72	37.86	685.90
47.00	-49.90	6.14	23.73	642.23
47.00	-50.07	2.55	14.51	125.83
47.00	-48.99	16.38	37.93	7162.52
47.00	-50.31	6.57	43.29	902.73
47.00	-48.71	7.37	34.47	1737.49
47.00	-50.40	6.07	39.42	763.94
47.00	-50.28	5.88	38.45	719.90
47.00	-50.30	7.18	68.83	361.28

47.00	-49.72	8.66	42.90	2459.74
47.00	-50.29	4.55	30.10	434.01
47.00	-50.27	5.47	51.76	207.78
47.00	-50.29	2.59	16.49	137.49
47.00	-48.16	6.84	41.76	271.86
47.00	-48.51	7.34	60.51	348.98
47.00	-48.85	4.67	24.67	410.53
47.00	-50.36	8.53	42.54	1337.60
47.00	-48.87	7.48	57.71	352.32
47.00	-48.90	4.97	22.02	436.50
47.00	-50.44	5.23	43.76	178.66
47.00	-48.60	7.33	33.29	1694.82
47.00	-47.69	6.84	34.04	1538.72
47.00	-49.88	3.19	24.83	64.38
47.00	-49.33	5.61	22.04	538.75
47.00	-48.91	5.98	50.61	234.41
47.00	-50.06	8.33	35.33	2130.62
47.00	-49.07	5.92	15.17	949.89
47.00	-50.30	9.21	76.43	551.11
47.00	-49.07	7.57	38.96	1065.23
47.00	-50.42	2.00	13.20	83.44
47.00	-47.51	15.41	71.99	1300.55
47.00	-50.24	5.65	31.45	1112.12
47.00	-48.41	15.81	74.00	1368.68
47.00	-49.11	4.99	22.46	441.82
47.00	-50.22	3.29	29.07	72.55
47.00	-50.34	6.85	45.42	983.45
47.00	-50.45	5.28	33.08	566.52
47.00	-48.76	15.96	73.05	1390.20
47.00	-50.38	5.40	49.15	198.40

47.00	-50.34	9.05	74.44	530.00
47.00	-50.40	3.71	23.01	80.21
47.00	-50.24	5.15	28.63	924.28
47.00	-48.75	16.27	65.14	1415.79
47.00	-49.38	7.19	55.16	324.98
47.00	-50.30	4.94	47.14	170.36
47.00	-49.48	7.01	52.58	305.88
47.00	-48.70	7.31	56.06	335.42
47.00	-50.43	6.51	41.82	874.31
47.00	-48.87	5.44	26.07	536.19

5.2.4 Rockfall barrier preliminary design

Based on performed rockfall test, a 3m tall barrier is sufficient to stop 90% of rocks at RD 37 + 80. This barrier is designed to withstand energy of 2500J.

5.3 Stabilization Measures

5.3.1 Rocscience SLIDE

SLIDE is a two-dimensional program for slope stability that evaluates factor of safety, probability of failure ie circular and non-circular failure surfaces in soil and rock slopes. Complex models can be created and analyzed efficiently although it is quite a simple program to use. External loading including seismic loading, groundwater and a wide variety of supports (different kinds of rock bolts) can be modelled in a multitude of ways.

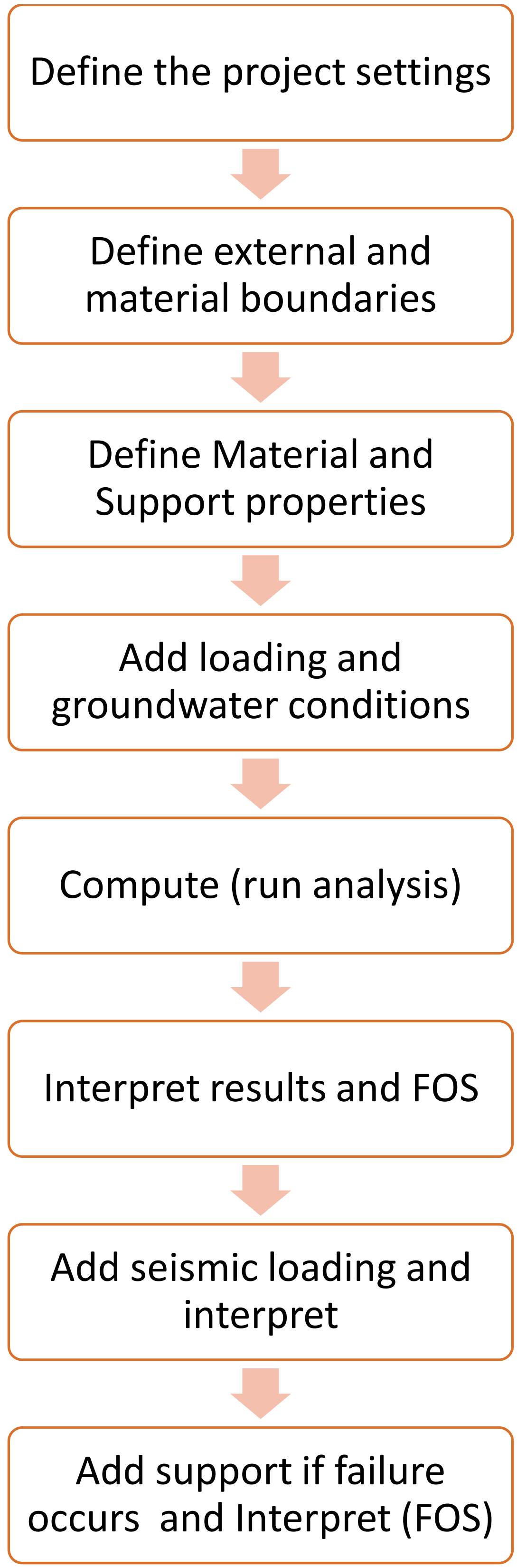
The stability of slip surfaces is analyzed through vertical slice limit equilibrium methods (e.g. Bishop, Janbu, Spencer, etc.). Individual slip surfaces can be analyzed, or search methods can be applied to locate the critical slip surface for a given slope. Deterministic (safety factor) or probabilistic (probability of failure) analyses can be carried out.

5.3.2 SLIDE features

- Critical surface search methods for circular or non-circular slip surfaces
- Analysis methods include Bishop, Janbu, Spencer, GLE / Morgenstern-Price

- Multiple materials
- Anisotropic, non-linear Mohr-Coulomb materials
- Probabilistic analysis – calculate probability of failure, reliability index (see below)
- Sensitivity Analysis
- Groundwater – piezo surfaces, Ru factors, pore pressure grids, finite element seepage analysis (see below), excess pore pressure (B-bar method)
- Finite element groundwater seepage for steady state or transient conditions
- Rapid drawdown analysis
- Tension crack (dry or water filled)
- External loading – line, distributed or seismic
- Support – soil nails, tiebacks, geotextiles, piles. Infinite strength (slip surface exclusion) zones
- Back analysis of required support force for a given safety factor
- View any or all surfaces generated by search
- Detailed analysis results can be plotted for individual slip surfaces

5.3.3 Algorithm for slope stability analysis



5.3.4 Interpreted Result and FOS

The acceptable FOS for static load is 1.5-1.7 and 1.2 for seismic loading.

RD 37 + 80

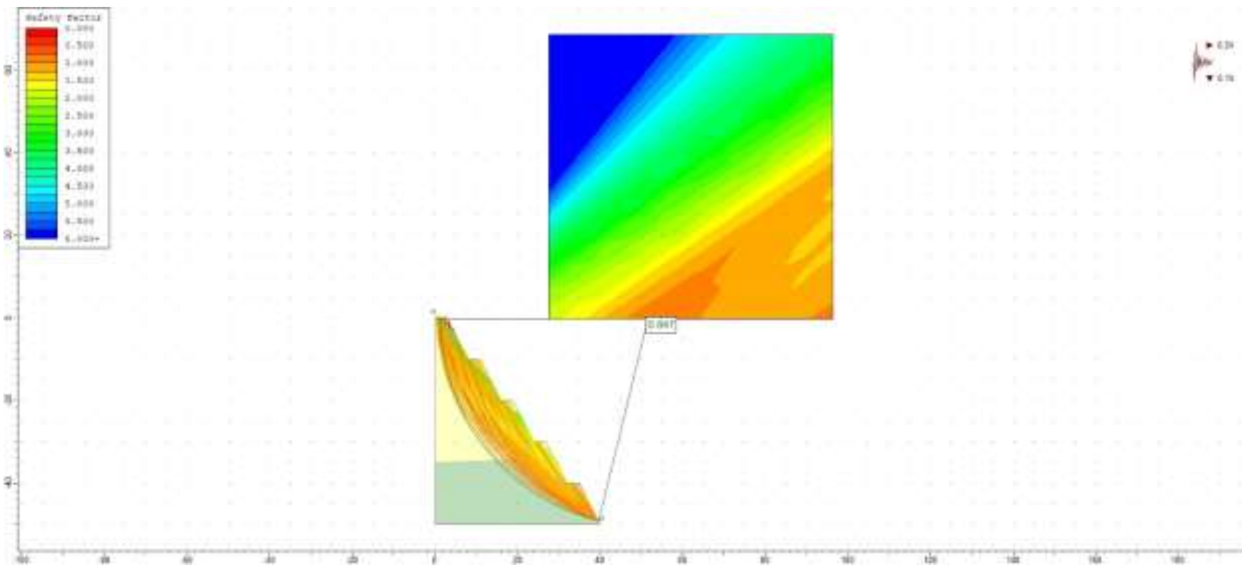


Figure 5-6 No reinforcement

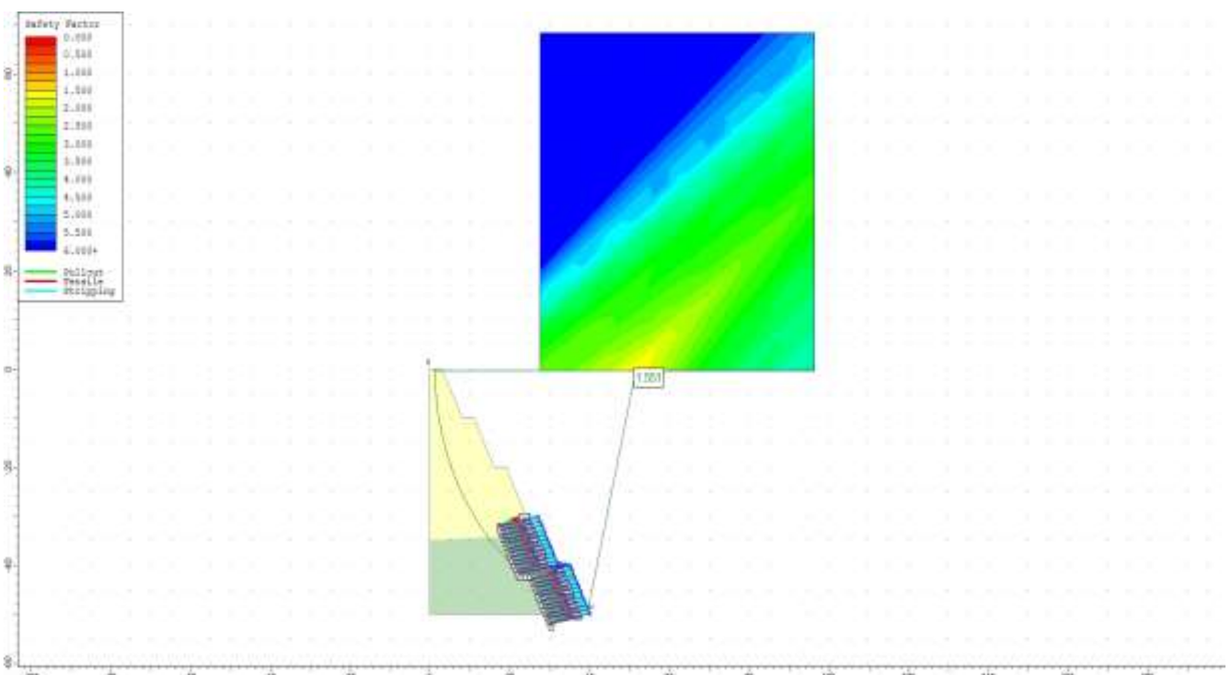


Figure 18 Reinforcement applied static loading

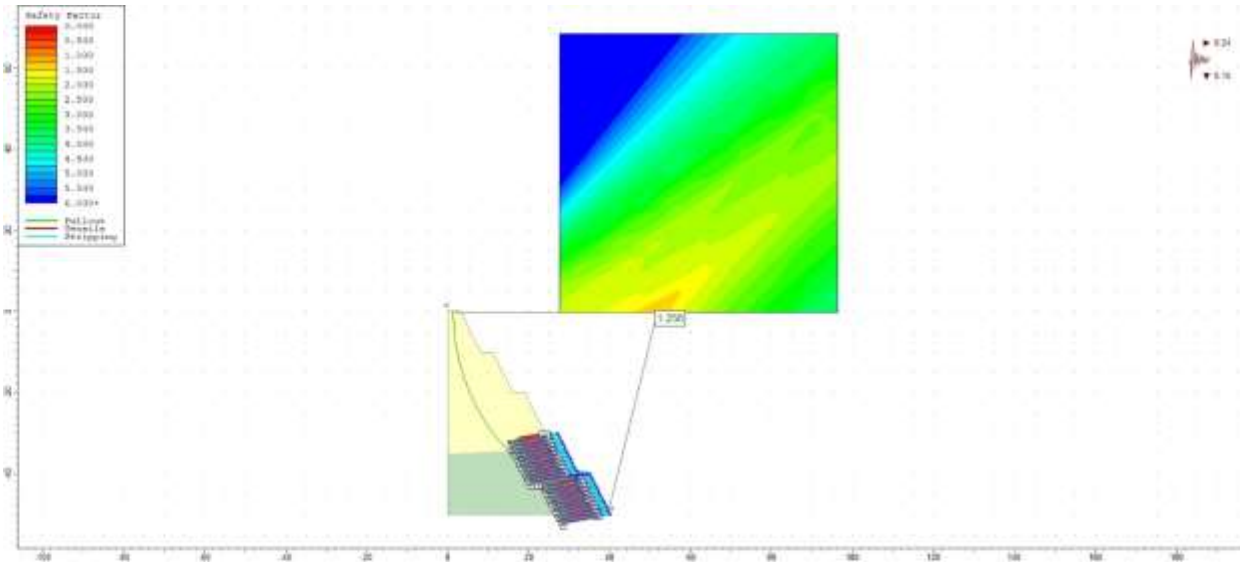


Figure 19 Reinforcement applied for seismic loading

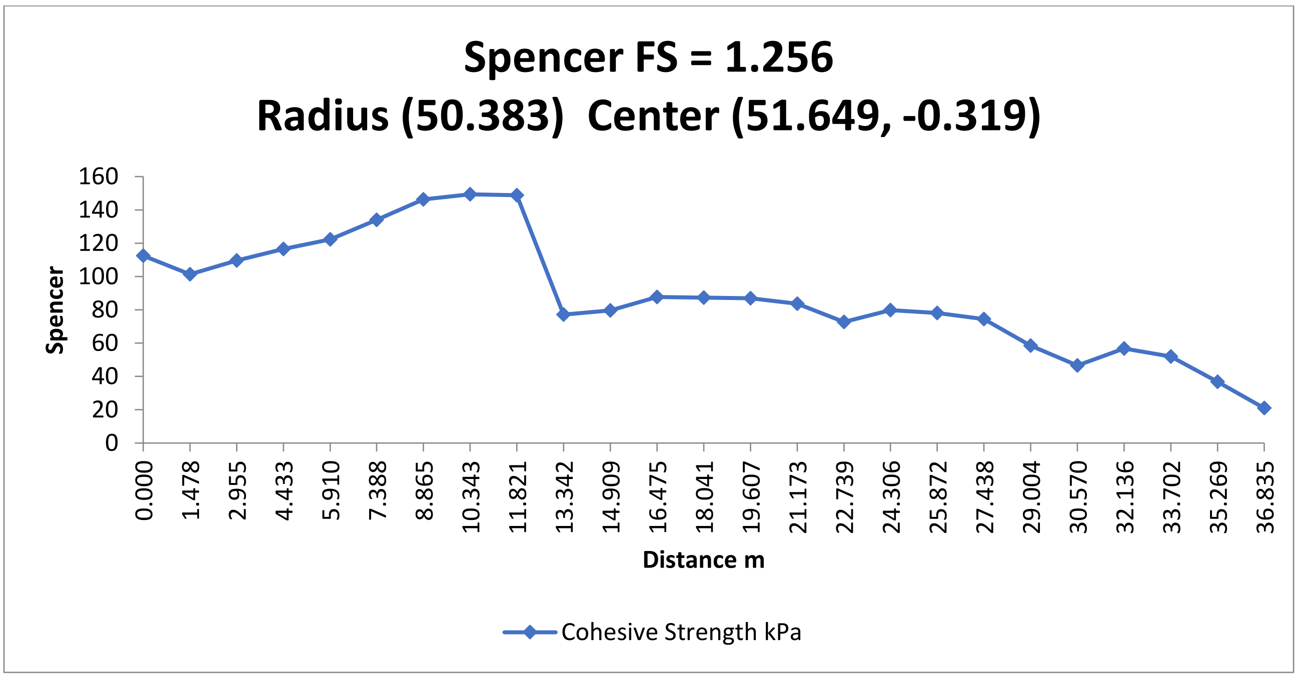
Support Properties

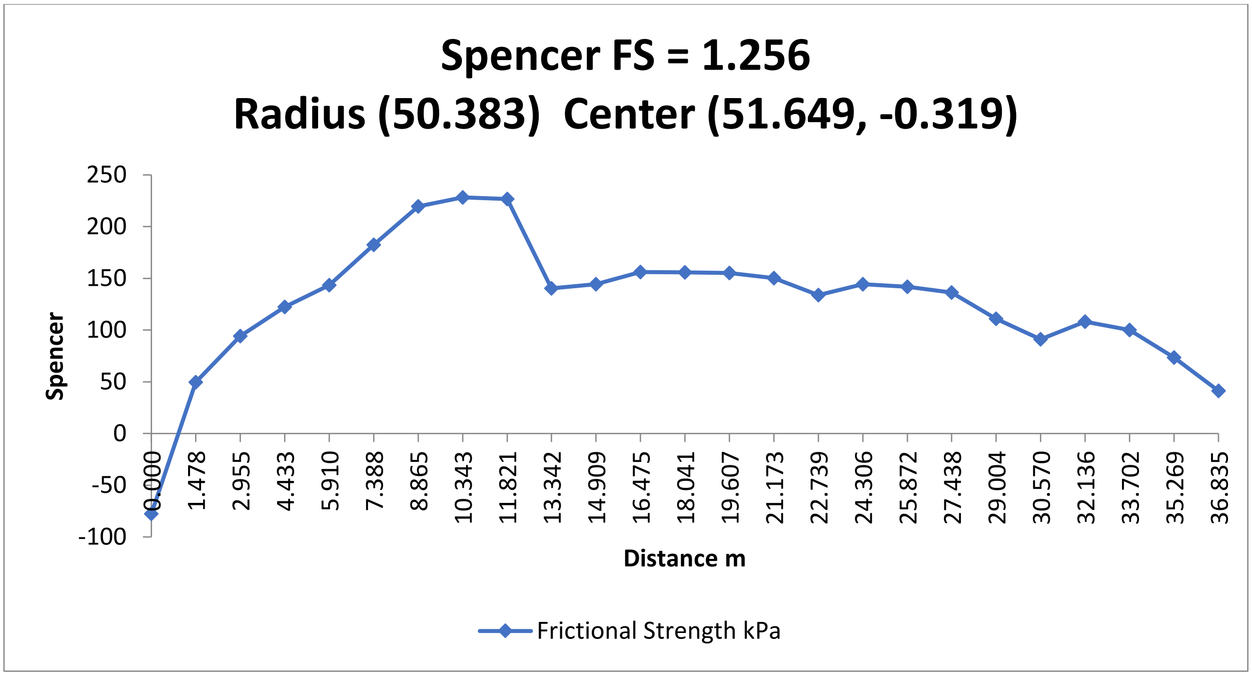
- Support Type: Grouted Tieback
- Force Application: Passive
- Out-of-Plane Spacing: 1 m
- Tensile Capacity: 350 kN
- Plate Capacity: 200 kN
- Bond length: 75 percent
- Bond Strength: 150 kN/m

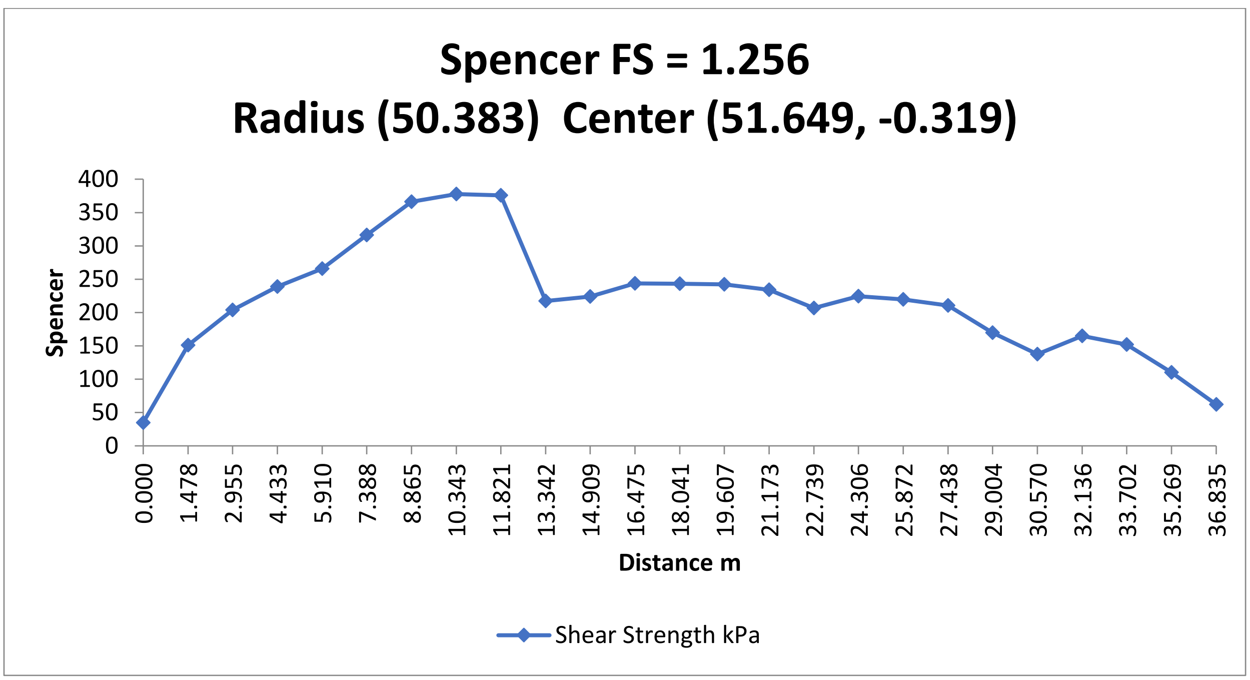
Slice	Width	Weight	Base	Base	Base	Shear	Shear	Base
	[m]	[kN]	Material	Cohesion	Friction Angle	Stress	Strength	Normal Stress
				[kPa]		[kPa]	[kPa]	[kPa]
1	1.47756	251.504	Marble	112.534	68.7733	27.8168	34.9401	-30.138
2	1.47756	547.031	Marble	101.315	56.6087	120.141	150.906	32.688
3	1.47756	600.897	Marble	109.657	53.4587	162.401	203.988	69.906 9

4	1.47756	616.005	Marble	116.462	51.7184	190.152	238.846	96.589
5	1.47756	621.325	Marble	122.182	50.5124	211.449	265.597	118.17
6	1.47756	699.205	Marble	133.908	48.4706	251.759	316.229	161.47
7	1.47756	774.441	Marble	146.31	46.7073	291.327	365.93	206.90
8	1.47756	754.556	Marble	149.341	46.3202	300.658	377.65	218.02
9	1.47756	707.474	Marble	148.808	46.3872	299.026	375.6	216.06
10	1.56616	691.317	graphytic schist	77.0329	22.868	172.987	217.285	332.54
11	1.56616	699.729	graphytic schist	79.717	22.4836	178.374	224.052	348.73
12	1.56616	756.081	graphytic schist	87.6089	21.4395	194.066	243.762	397.65
13	1.56616	723.978	graphytic schist	87.4053	21.4649	193.664	243.257	396.36
14	1.56616	642.304	graphytic schist	86.9278	21.5249	192.72	242.072	393.35
15	1.56616	556.786	graphytic schist	83.7275	21.9378	186.375	234.101	373.35
16	1.56616	530.02	graphytic schist	72.7922	23.5101	164.418	206.522	307.40
17	1.56616	568.381	graphytic schist	79.7815	22.4745	178.503	224.214	349.13
18	1.56616	530.853	graphytic schist	78.026	22.7239	174.983	219.793	338.50
19	1.56616	433.733	graphytic schist	74.3713	23.2658	167.617	210.54	316.70
20	1.56616	336.116	graphytic schist	58.5481	26.0505	135.037	169.617	227.21
21	1.56616	283.901	graphytic schist	46.6646	28.8096	109.622	137.694	165.51

22	1.56616	307.641	graphytic schist	56.8382	26.4046	131.438	165.096	218.04
23	1.56616	272.33	graphytic schist	51.9645	27.4875	121.078	152.084	192.42
24	1.56616	165.061	graphytic schist	36.8454	31.7969	87.7482	110.219	118.35
25	1.56616	55.3907	graphytic schist	20.9508	39.429	49.5417	62.2284	50.200




Spencer FS = 1.256
Radius (50.383) Center (51.649, -0.319)
250
200
150
100
50
0
-50
-100
Spencer
0.000
1.478
2.955
4.433
5.910
7.388
8.865
10.343
11.821
13.342
14.909
16.475
18.041
19.607
21.173
22.739
24.306
25.872
27.438
29.004
30.570
32.136
33.702
35.269
36.835
Distance m
Frictional Strength kPa


Spencer FS = 1.256
Radius (50.383) Center (51.649, -0.319)
400
350
300
250
200
150
100
50
0
Spencer
0.000
1.478
2.955
4.433
5.910
7.388
8.865
10.343
11.821
13.342
14.909
16.475
18.041
19.607
21.173
22.739
24.306
25.872
27.438
29.004
30.570
32.136
33.702
35.269
36.835
Distance m
Shear Strength kPa

RD 37 + 130

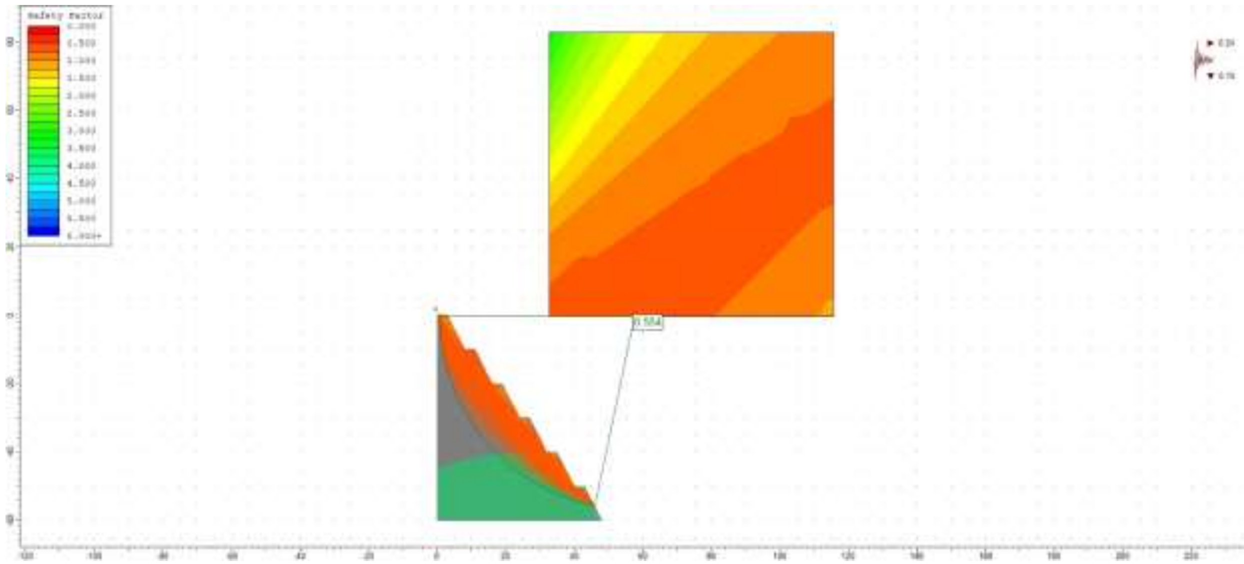


Figure 20 No reinforcement applied

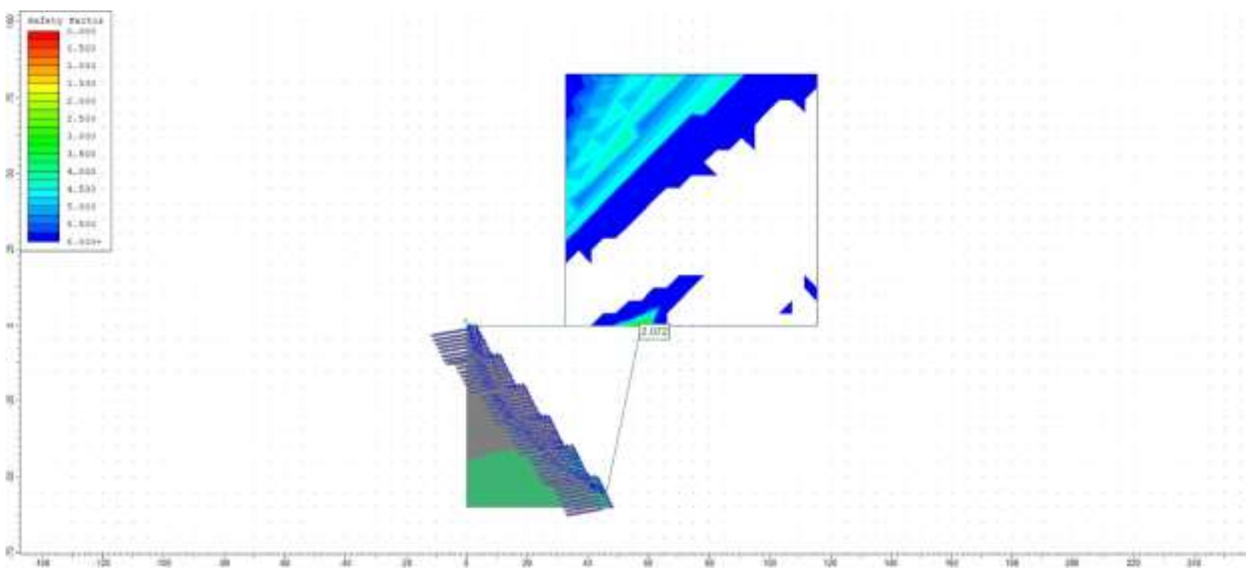


Figure 5-21 Reinforcement applied for static loading

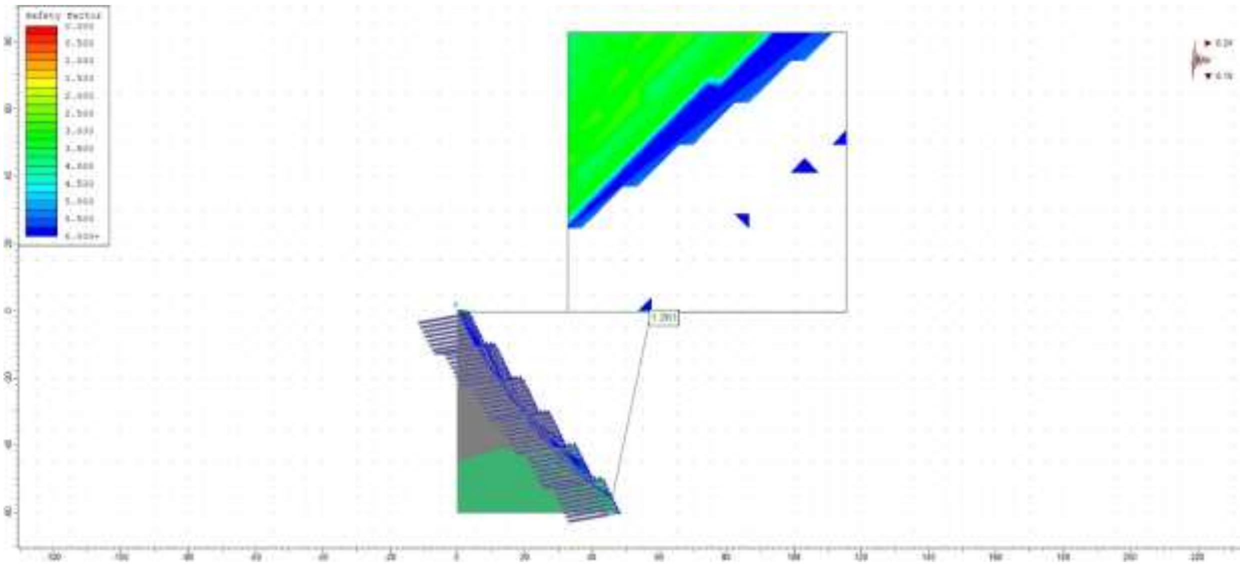


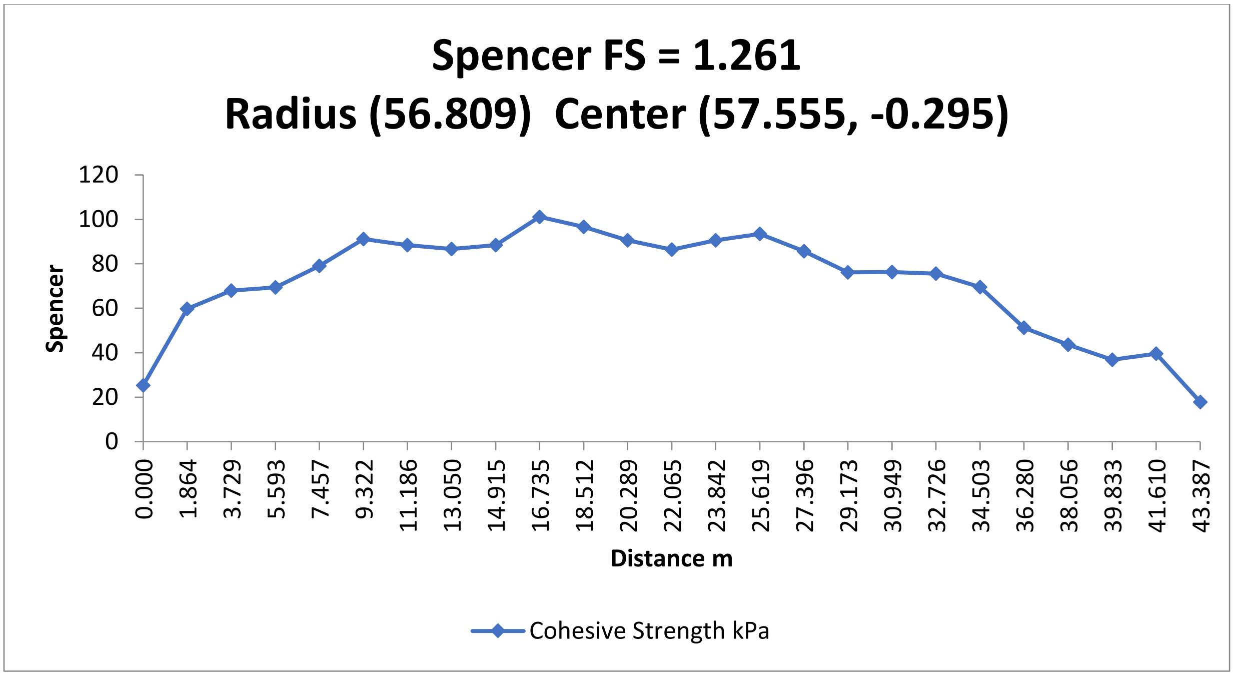
Figure 22 Reinforcement applied seismic loading

Support Properties

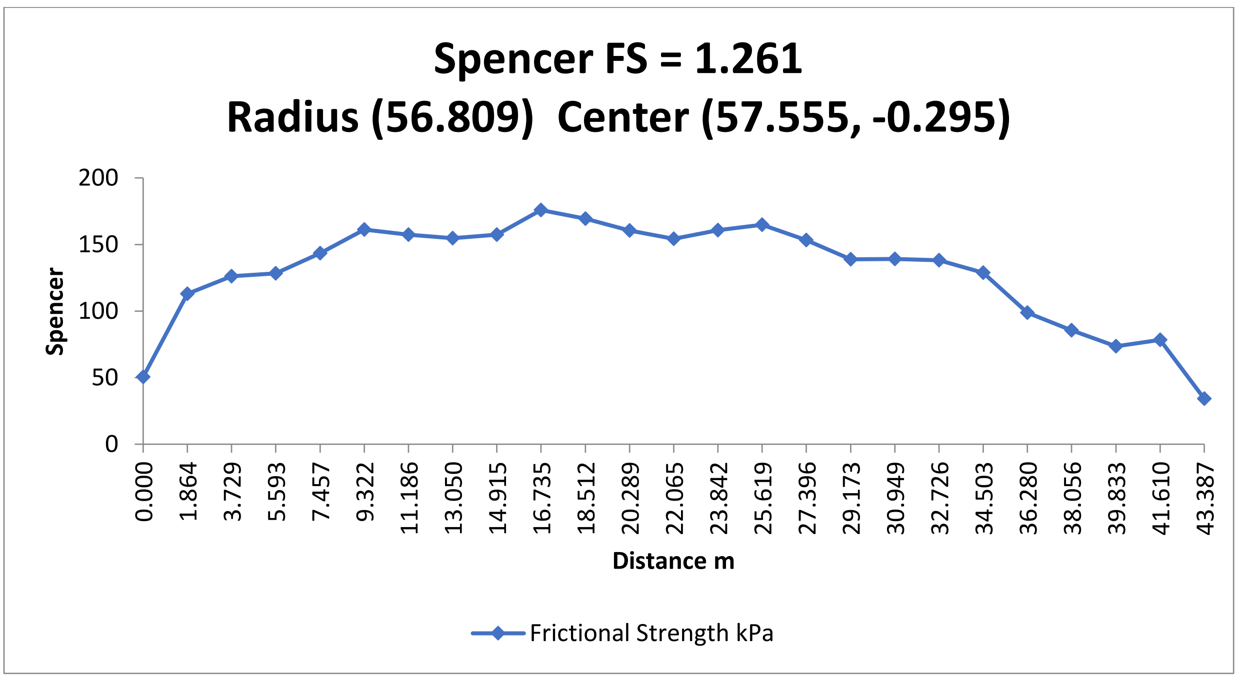
- Support Type: Grouted Tieback
- Force Application: Active
- Out-of-Plane Spacing: 1 m
- Tensile Capacity: 350 kN
- Plate Capacity: 200 kN
- Bond length: 75 percent
- Bond Strength: 150 kN/m

	Width	Weight	Base	Base	Base	Shear	Shear	Base
	[m]	[kN]	Material	Cohesion	Friction Angle	Stress	Strength	Normal Stress
				[kPa]		[kPa]	[kPa]	[kPa]
1	1.86431	364.142	graphatic schist	25.2356	36.8267	60.1061	75.7752	67.492
2	1.86431	798.165	graphatic schist	59.6981	25.8191	136.946	172.646	233.44
3	1.86431	867.269	graphatic schist	67.9605	24.2993	153.997	194.142	279.47
4	1.86431	878.948	graphatic schist	69.3528	24.0651	156.837	197.723	287.44
5	1.86431	967.201	graphatic schist	79.0917	22.5717	176.474	222.479	344.94
6	1.86431	1083.07	graphatic schist	91.0782	21.0168	200.166	252.347	419.75
7	1.86431	1047.25	graphatic schist	88.4679	21.3329	195.045	245.892	403.08
8	1.86431	976.025	graphatic schist	86.6592	21.5589	191.486	241.404	391.66
9	1.86431	954.699	graphatic schist	88.3816	21.3435	194.876	245.679	402.54
10	1.77677	988.522	green schist	101.138	19.8968	219.716	276.994	485.88
11	1.77677	955.799	green schist	96.6183	20.3818	210.966	265.963	455.77
12	1.77677	860.483	green schist	90.5813	21.0761	199.193	251.12	416.56
13	1.77677	770.116	green schist	86.3375	21.5996	190.851	240.605	389.64

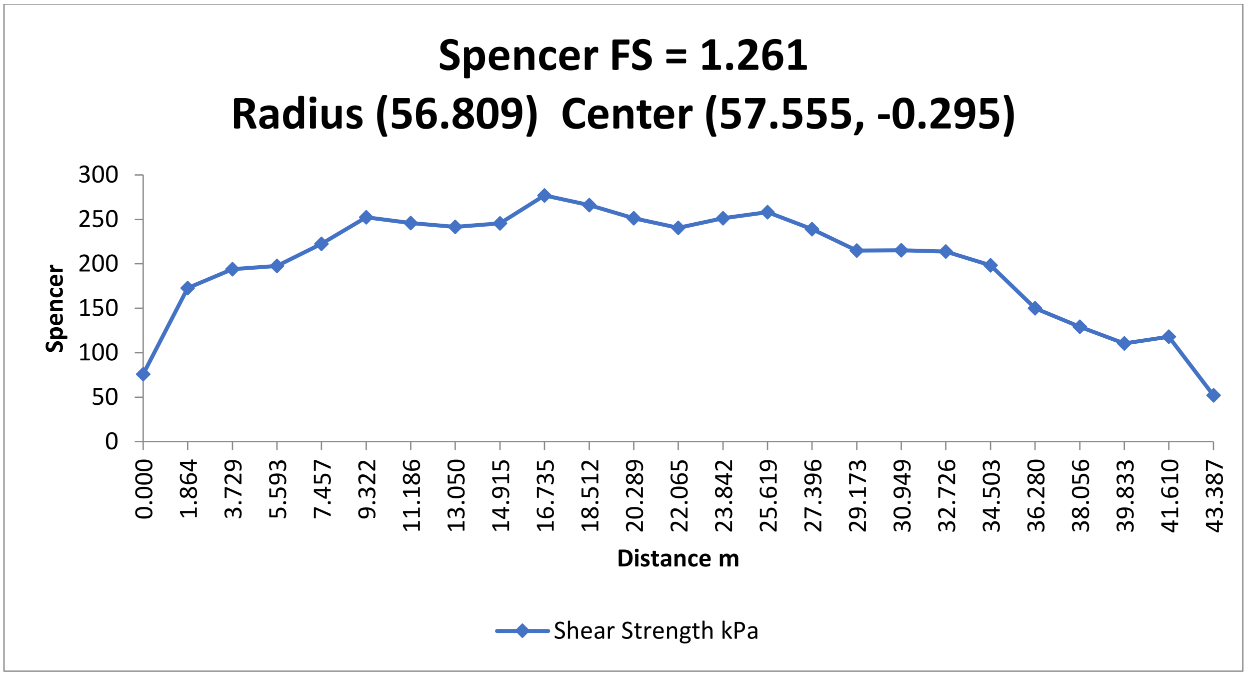
14	1.77677	794.492	green schist	90.6371	21.0694	199.302	251.258	416.92
15	1.77677	811.798	green schist	93.4952	20.734	204.888	258.3	435.36
16	1.77677	706.337	green schist	85.6919	21.6821	189.578	238.999	385.59
17	1.77677	587.824	green schist	76.1417	22.9993	170.566	215.031	327.21
18	1.77677	525.333	green schist	76.2788	22.979	170.841	215.378	328.03
19	1.77677	559.698	green schist	75.6093	23.0786	169.496	213.681	324.04
20	1.77677	487.775	green schist	69.5929	24.0253	157.326	198.339	288.82
21	1.77677	355.369	green schist	51.1706	27.6753	118.938	149.944	188.33
22	1.77677	229.953	green schist	43.6152	29.6525	102.551	129.285	150.48
23	1.77677	220.998	green schist	36.8858	31.7828	87.5189	110.334	118.54
24	1.77677	207.728	green schist	39.5386	30.893	93.5009	117.876	130.98
25	1.77677	72.8293	green schist	17.8581	41.7544	41.2553	52.0101	38.258



Spencer FS = 1.261
Radius (56.809) Center (57.555, -0.295)
Spencer
120
100
80
60
40
20
0
0.000
1.864
3.729
5.593
7.457
9.322
11.186
13.050
14.915
16.735
18.512
20.289
22.065
23.842
25.619
27.396
29.173
30.949
32.726
34.503
36.280
38.056
39.833
41.610
43.387
Distance m
Cohesive Strength kPa



Spencer FS = 1.261
Radius (56.809) Center (57.555, -0.295)
Spencer
200
150
100
50
0
0.000
1.864
3.729
5.593
7.457
9.322
11.186
13.050
14.915
16.735
18.512
20.289
22.065
23.842
25.619
27.396
29.173
30.949
32.726
34.503
36.280
38.056
39.833
41.610
43.387
Distance m
Frictional Strength kPa



RD 37 + 170

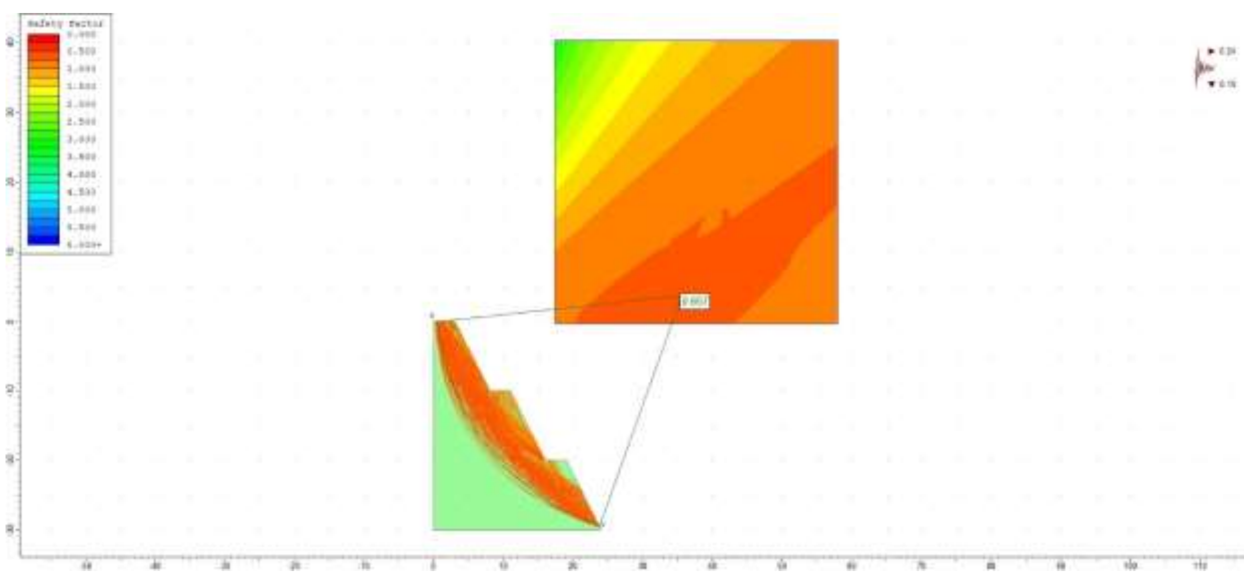


Figure 5-12 No reinforcement applied

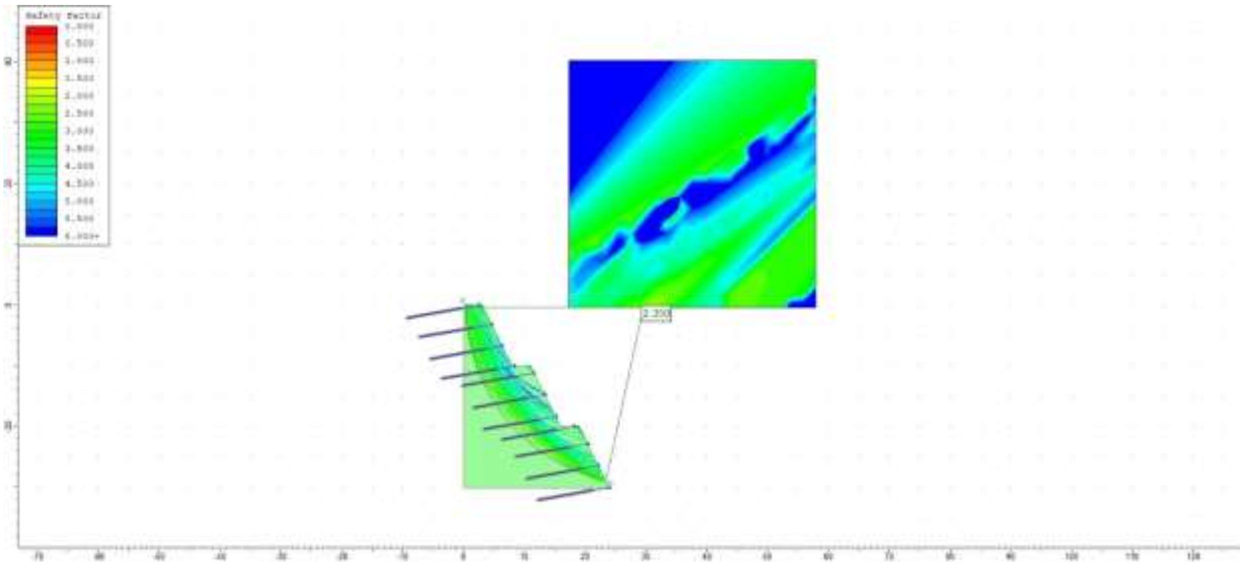


Figure 5-13 Reinforcement applied static loading

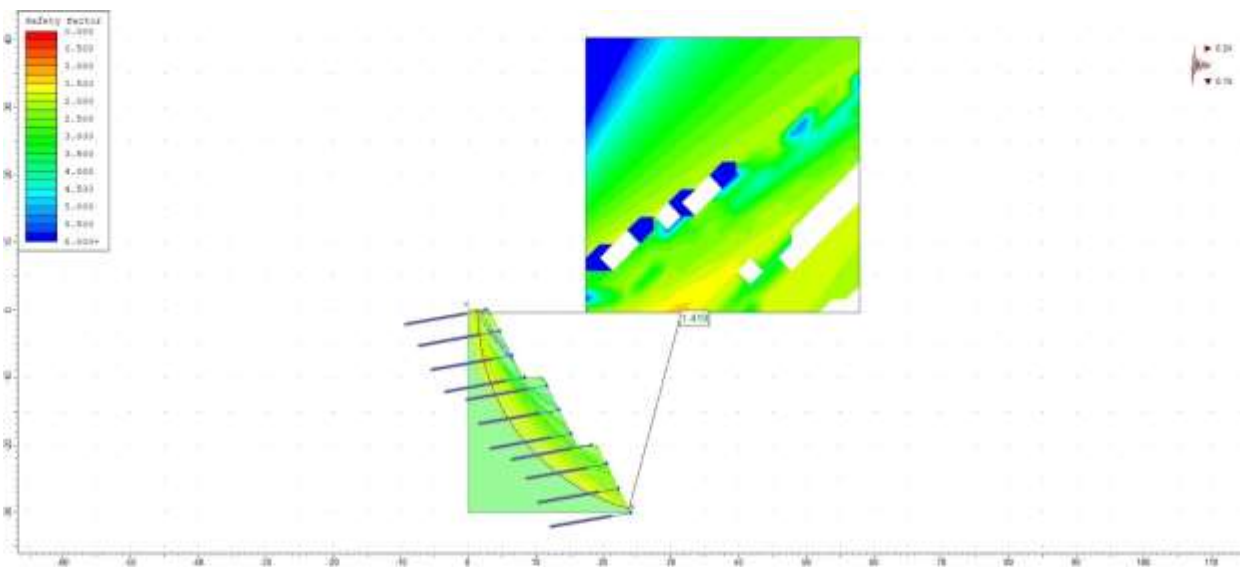


Figure 23 Reinforcement applied seismic loading

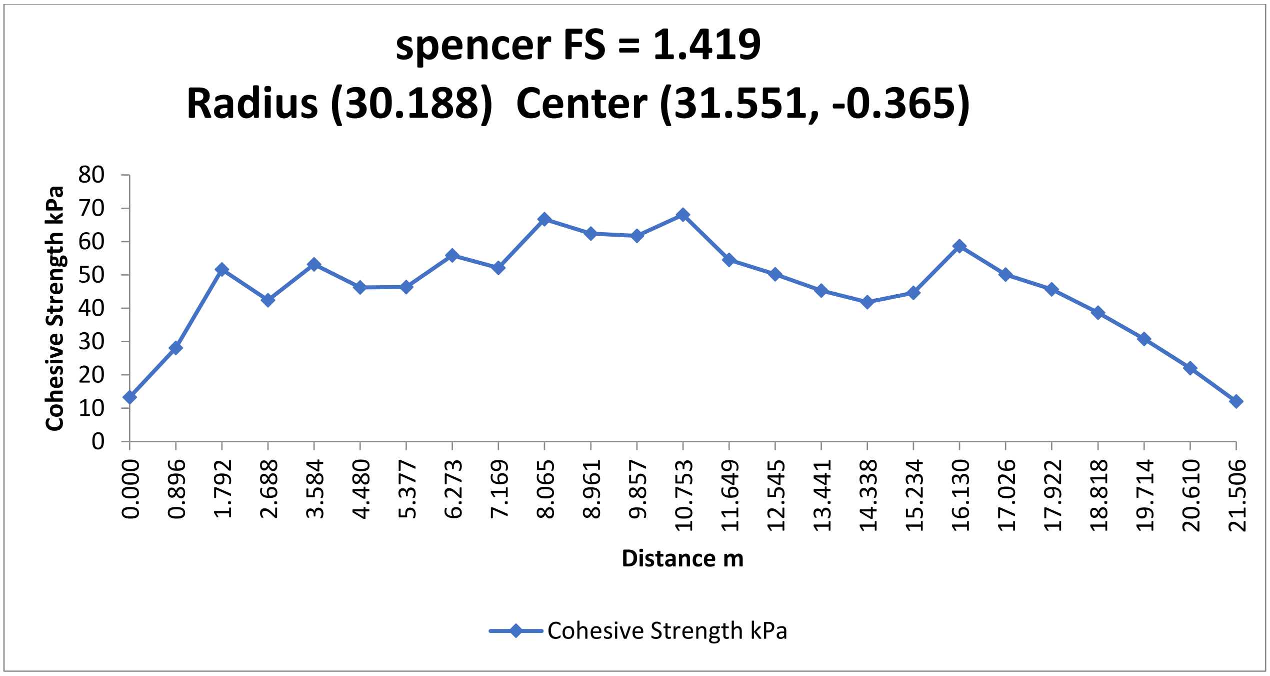
Support Properties

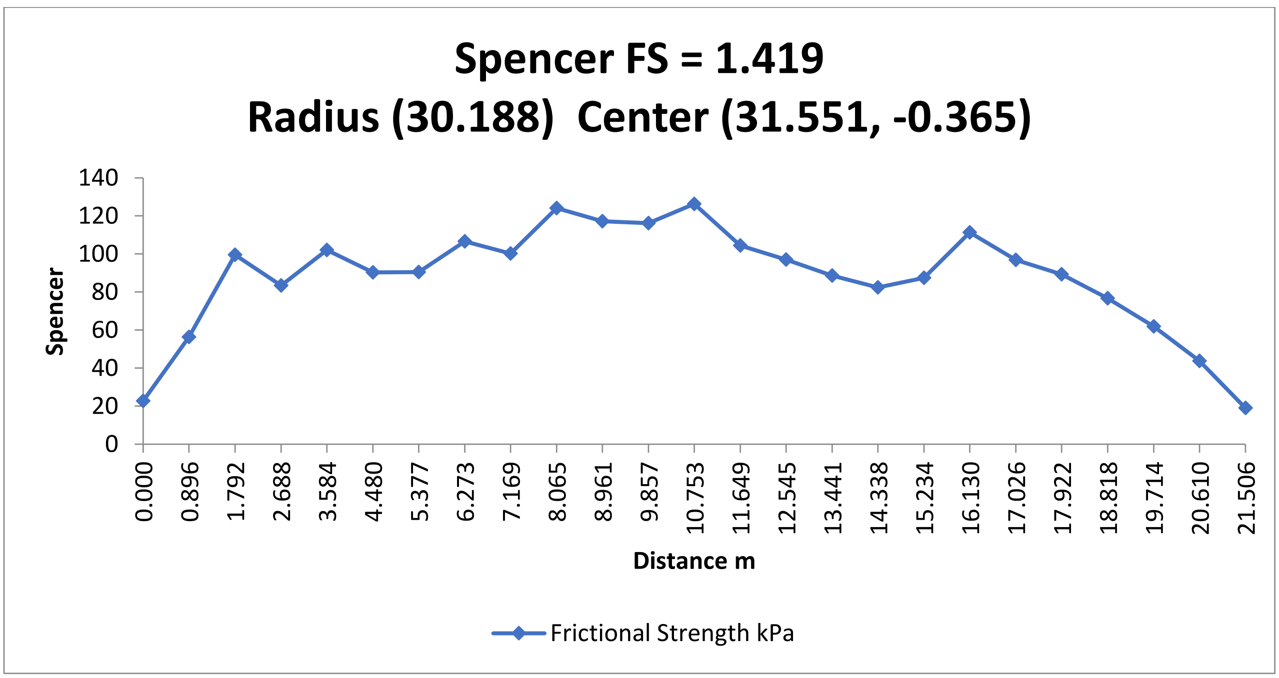
- Support Type: Grouted Tieback
- Force Application: Active
- Out-of-Plane Spacing: 2 m
- Tensile Capacity: 350 kN

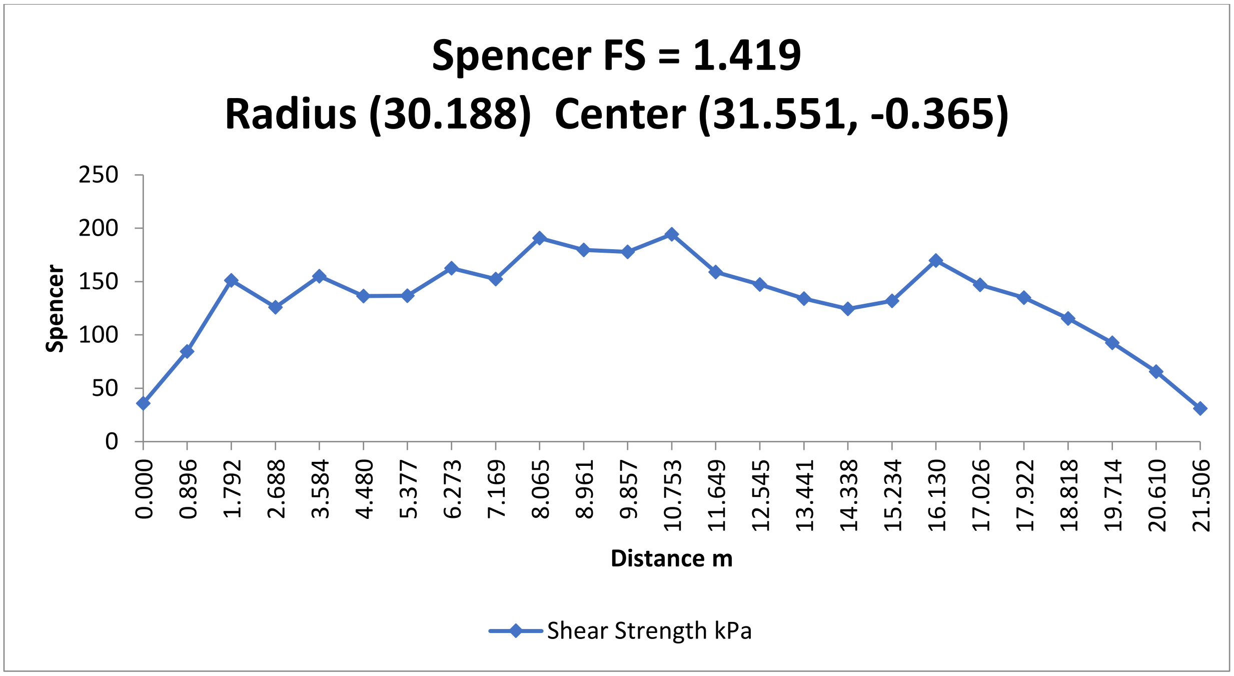
- Plate Capacity: 200 kN
- Bond length: 75 percent
- Bond Strength: 150 kN/m

	Width	Weight	Base	Base	Base	Shear	Shear	Base
	[m]	[kN]	Material	Cohesion	Friction Angle	Stress	Strength	Normal Stress
				[kPa]		[kPa]	[kPa]	[kPa]
1	0.896091	93.539	Green schist	13.3333	46.3517	25.3705	35.9904	21.6126
2	0.896091	212.275	Green schist	28.0983	35.366	59.5869	84.5294	79.506
3	0.896091	244.805	Green schist	51.6549	27.5604	106.62	151.25	190.829
4	0.896091	249.89	Green schist	42.3713	30.0163	88.6974	125.825	144.452
5	0.896091	247.417	Green schist	53.1328	27.2174	109.419	155.221	198.493
6	0.896091	239.778	Green schist	46.2667	28.9159	96.2941	136.602	163.534
7	0.896091	228.273	Green schist	46.3514	28.8932	96.458	136.834	163.955
8	0.896091	221.066	Green schist	55.8524	26.6146	114.536	162.479	212.794
9	0.896091	242.289	Green schist	52.076	27.4615	107.419	152.383	193.005
10	0.896091	264.822	Green schist	66.6856	24.5189	134.538	190.854	272.223

11	0.896091	284.243	Green schist	62.3749	25.3005	126.646	179.659	248.11
12	0.896091	273.413	Green schist	61.7116	25.4265	125.424	177.926	244.455
13	0.896091	249.245	Green schist	68.0441	24.285	137.007	194.357	279.948
14	0.896091	223.754	Green schist	54.5603	26.8966	112.11	159.038	205.969
15	0.896091	197.051	Green schist	50.1948	27.9107	103.841	147.308	183.332
16	0.896091	169.229	Green schist	45.2893	29.1816	94.3992	133.914	158.694
17	0.896091	149.625	Green schist	41.8319	30.1779	87.6358	124.319	141.853
18	0.896091	159.211	Green schist	44.5928	29.375	93.0444	131.992	155.266
19	0.896091	170.205	Green schist	58.6624	26.0272	119.78	169.918	227.834
20	0.896091	178.214	Green schist	50.0765	27.9397	103.615	146.988	182.728
21	0.896091	155.456	Green schist	45.7234	29.0628	95.2417	135.109	160.84
22	0.896091	122.194	Green schist	38.6441	31.1851	81.3088	115.344	126.721
23	0.896091	88.1669	Green schist	30.8088	34.1356	65.3067	92.6434	91.2071
24	0.896091	53.4005	Green schist	22.0303	38.7163	46.3135	65.6998	54.4767
25	0.896091	17.9178	Green schist	12.04	48.1285	21.8677	31.0213	17.014


spencer FS = 1.419
Radius (30.188) Center (31.551, -0.365)
Cohesive Strength kPa
80
70
60
50
40
30
20
10
0
0.000
0.896
1.792
2.688
3.584
4.480
5.377
6.273
7.169
8.065
8.961
9.857
10.753
11.649
12.545
13.441
14.338
15.234
16.130
17.026
17.922
18.818
19.714
20.610
21.506
Distance m
Cohesive Strength kPa


Spencer FS = 1.419
Radius (30.188) Center (31.551, -0.365)
Spencer
140
120
100
80
60
40
20
0
0.000
0.896
1.792
2.688
3.584
4.480
5.377
6.273
7.169
8.065
8.961
9.857
10.753
11.649
12.545
13.441
14.338
15.234
16.130
17.026
17.922
18.818
19.714
20.610
21.506
Distance m
Frictional Strength kPa



RD 37 + 190

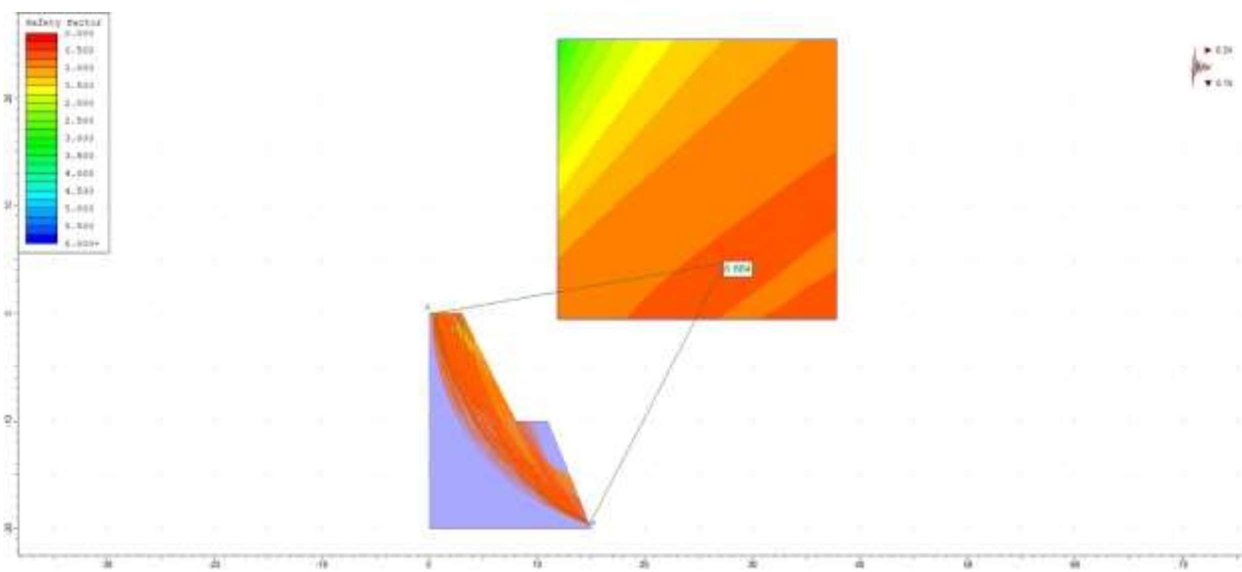


Figure 5-15 No reinforcement applied

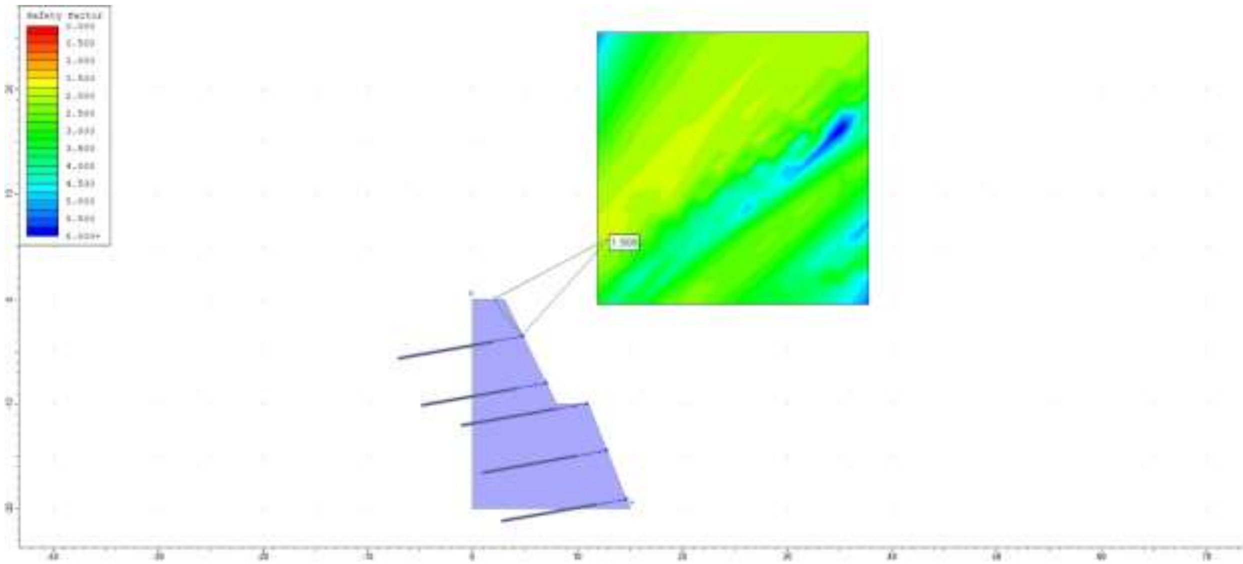


Figure 5-16 Reinforcement applied static loading

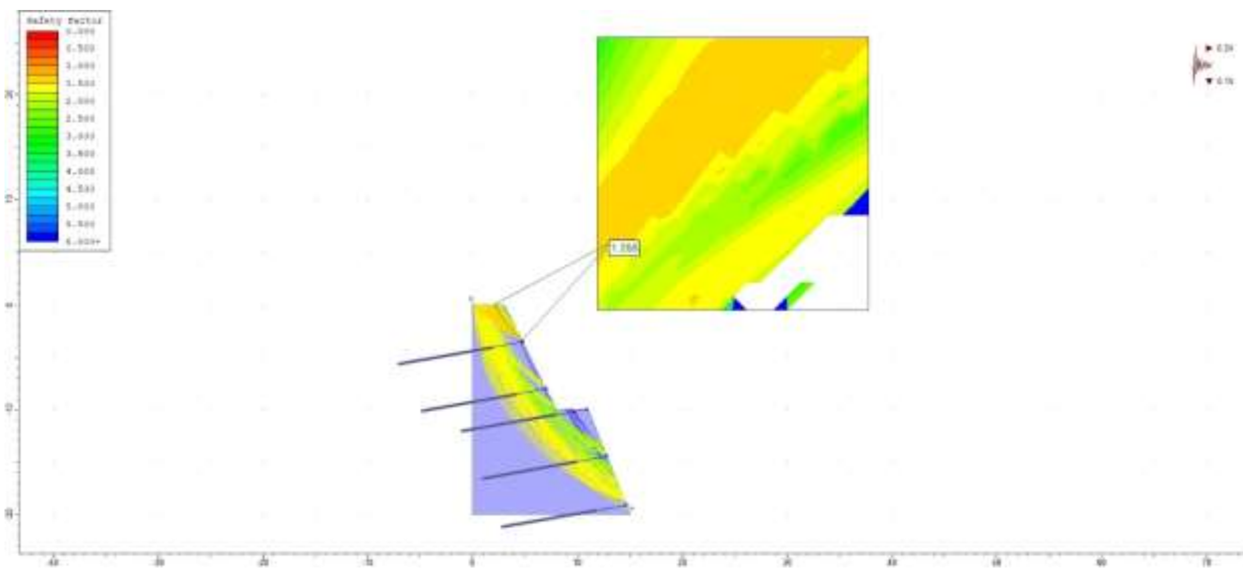


Figure 24 Reinforcement applied seismic loading

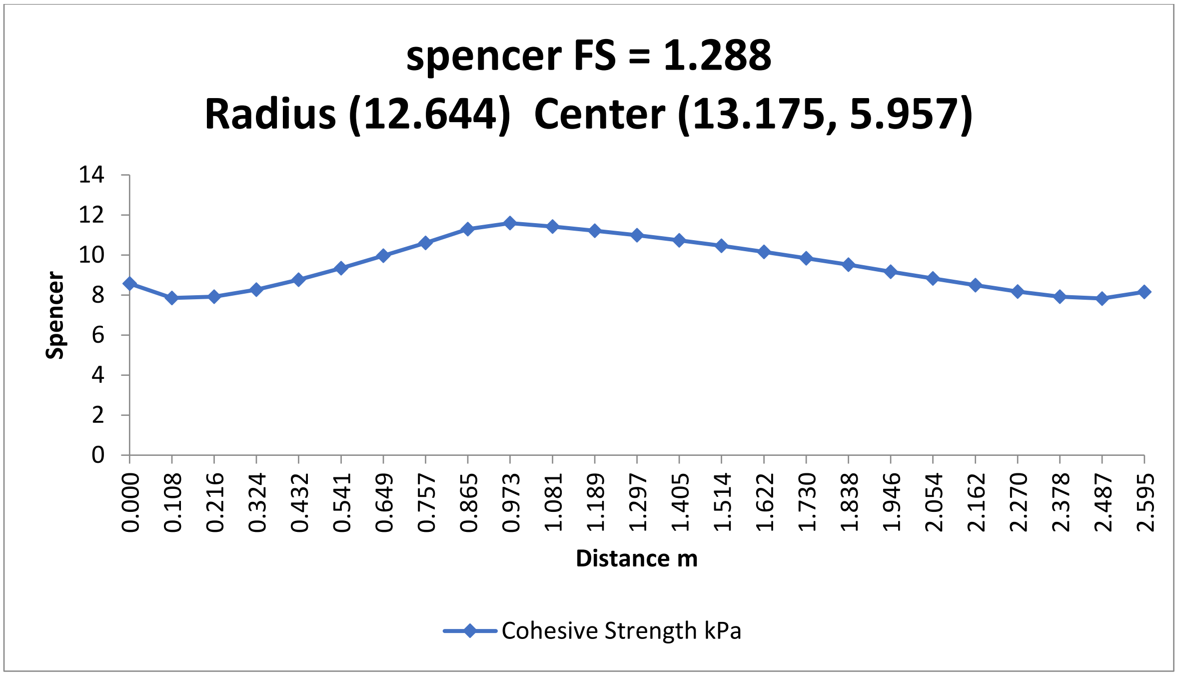
Support Properties

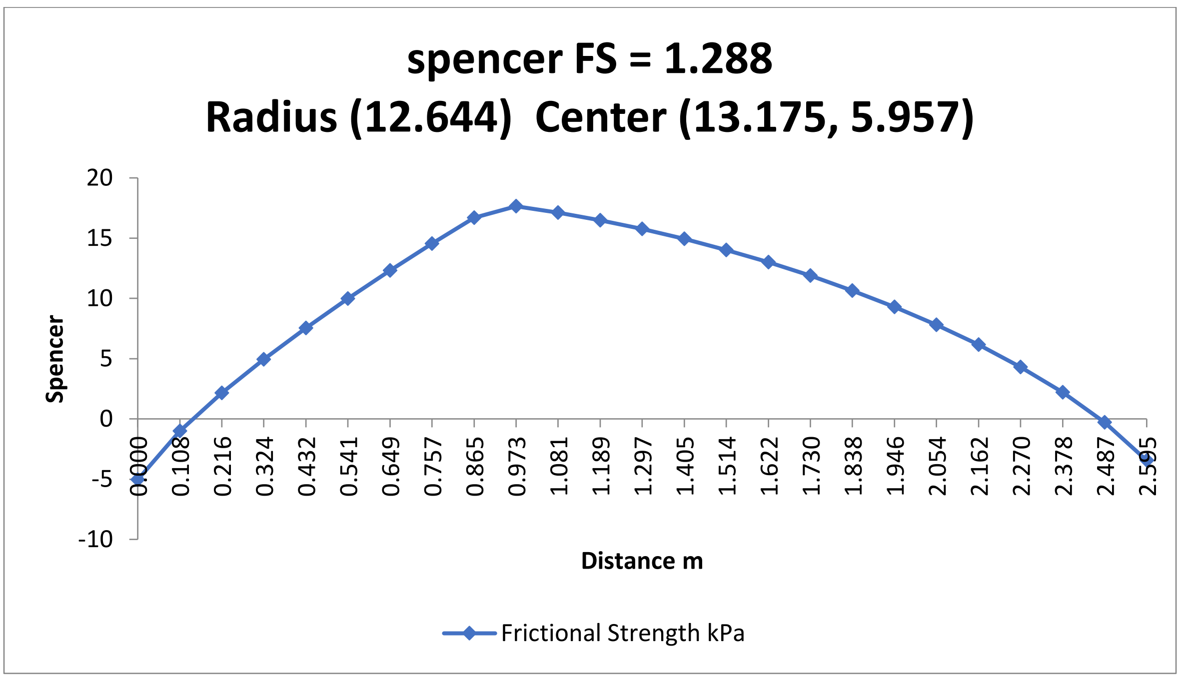
- Support Type: Grouted Tieback
- Force Application: Active
- Out-of-Plane Spacing: 3 m
- Tensile Capacity: 350 kN
- Plate Capacity: 200 kN

- Bond length: 75 percent
- Bond Strength: 150 kN/m

	Width	Weight	Base	Base	Base	Shear	Shear	Base
	[m]	[kN]	Material	Cohesion	Friction Angle	Stress	Strength	Normal Stress
				[kPa]		[kPa]	[kPa]	[kPa]
1	0.108111	0.278479	Mica Schist	8.23843	66.6613	2.31963	4.3767	-1.66621
2	0.108111	0.824158	Mica Schist	7.82628	61.8502	4.27267	8.06171	0.125967
3	0.108111	1.34827	Mica Schist	8.06338	58.6101	6.12713	11.5607	2.13394
4	0.108111	1.85263	Mica Schist	8.53084	56.1295	7.90672	14.9185	4.28753
5	0.108111	2.33882	Mica Schist	9.10854	54.11	9.62284	18.1565	6.54719
6	0.108111	2.8082	Mica Schist	9.74625	52.405	11.2824	21.2877	8.88657
7	0.108111	3.26194	Mica Schist	10.4184	50.9302	12.8902	24.3214	11.2865
8	0.108111	3.70109	Mica Schist	11.1105	49.6321	14.4499	27.2643	13.7324
9	0.108111	4.12657	Mica Schist	11.8134	48.4743	15.9646	30.1222	16.2128
10	0.108111	4.26459	Mica Schist	12.1179	48.0122	16.6038	31.3282	17.2896
11	0.108111	4.05808	Mica Schist	11.9149	48.3179	16.1787	30.5262	16.5717
12	0.108111	3.83937	Mica Schist	11.684	48.6775	15.6901	29.6042	15.7558

13	0.108111	3.60981	Mica Schist	11.4274	49.0929	15.1403	28.5668	14.8504
14	0.108111	3.36993	Mica Schist	11.1466	49.5691	14.5292	27.4139	13.8597
15	0.108111	3.12022	Mica Schist	10.8429	50.1124	13.8562	26.1441	12.7881
16	0.108111	2.86114	Mica Schist	10.5182	50.7312	13.1202	24.7554	11.6401
17	0.108111	2.59309	Mica Schist	10.1746	51.4363	12.3196	23.2447	10.4201
18	0.108111	2.31646	Mica Schist	9.81473	52.2423	11.4516	21.607	9.13315
19	0.108111	2.03158	Mica Schist	9.44259	53.1693	10.5132	19.8364	7.78419
20	0.108111	1.73879	Mica Schist	9.06395	54.2455	9.4996	17.9239	6.37934
21	0.108111	1.43838	Mica Schist	8.68802	55.5118	8.40459	15.8579	4.92551
22	0.108111	1.13062	Mica Schist	8.33082	57.0326	7.2193	13.6214	3.43148
23	0.108111	0.815784	Mica Schist	8.02329	58.9155	5.93066	11.19	1.9091
24	0.108111	0.494096	Mica Schist	7.83555	61.3684	4.51795	8.52451	0.376121
25	0.108111	0.165785	Mica Schist	7.97668	64.8829	2.94335	5.55355	-1.13596





5.3.5 Findings and Recommendations of analysis and stabilization

- Rock slope is unstable as evident from FOS which is less than 1 in all cross sections analyzed in which rock bolts have not been installed.

	RD 37+80	RD 37+130	RD 37+170	RD 37+190
FOS No Support	0.847	0.554	0.657	0.684
FOS Support applied, Static loading only	1.551	2	2.35	1.908
FOS Support, Seismic loading	1.256	1.261	1.4	1.288

- Rock bolts design is such that FOS is above 1.5 for static loads and above 1.2 for seismic loading as evident from screenshots added

	RD 37+80	**RD 37+130**	**RD 37+170**	**RD 37+190**
Support type	Grouted Tieback	Grouted Tieback	Grouted Tieback	Grouted Tieback
Spacing	1m	1m	2m	3m
Rockbolt length	12m	12m	12m	12m
Tensile Capacity	350kN	350kN	350kN	350kN
Plate Capacity	200kN	200kN	200kN	200kN
Bond length	75%	75%	75%	75%
Bond strength	150kN/m	150kN/m	150kN/m	150kN/m
Force applied	Passive	Passive	Passive	Passive

5.3.6 Suggested Rock bolt design

	RD 37+80 to RD 37 + 110	RD 37+110 to RD 37 + 140	RD 37+140 to RD 37 + 170	RD 37+170 to RD 37 + 200
Support type	Grouted Tieback	Grouted Tieback	Grouted Tieback	Grouted Tieback
Spacing	1m	1m	2m	3m
Benches	Bottom two	All	All	All
Rockbolt length	12m	12m	12m	12m
Tensile Capacity	350kN	350kN	350kN	350kN
Plate Capacity	200kN	200kN	200kN	200kN
Bond length	75%	75%	75%	75%
Bond strength	150kN/m	150kN/m	150kN/m	150kN/m
Force applied	Passive	Passive	Passive	Passive

5.4 Conclusion

Rockfall has been analyzed and a preliminary barrier is designed using Rocfall software. Slope stability has been evaluated using SLIDE and rock bolts have been designed keeping seismic loading in mind as described in this chapter.

Chapter 6
CONCLUSIONS

6.1 Investigation Results

Site investigation was carried out for detailed site characterization. Scanline Survey, geological mapping, spot discontinuities mapping was conducted to obtain in-depth information about rock strata; dip, strike, joints spacing, folds, faults and bedding planes etc. Five kind of rocks were found including Marble, Calcareous Phyllite, Graphitic Schist, Green Schist and Coal Bed. Geomechanical tests, lab investigation and testing was carried out to determine rock index properties.

As result of lab and field testing, scanline survey and geological mapping the geotechnical profile was generated which was analyzed through SLIDE software.

Furthermore, rock fall test was conducted and rock paths were analyzed using ROCFALL software and a preliminary barrier was designed.

Scanline survey data was also analyzed using DIPS and kinematic analysis at different slope profiles was carried out. Stereonets were generated and analyzed using DIPS.

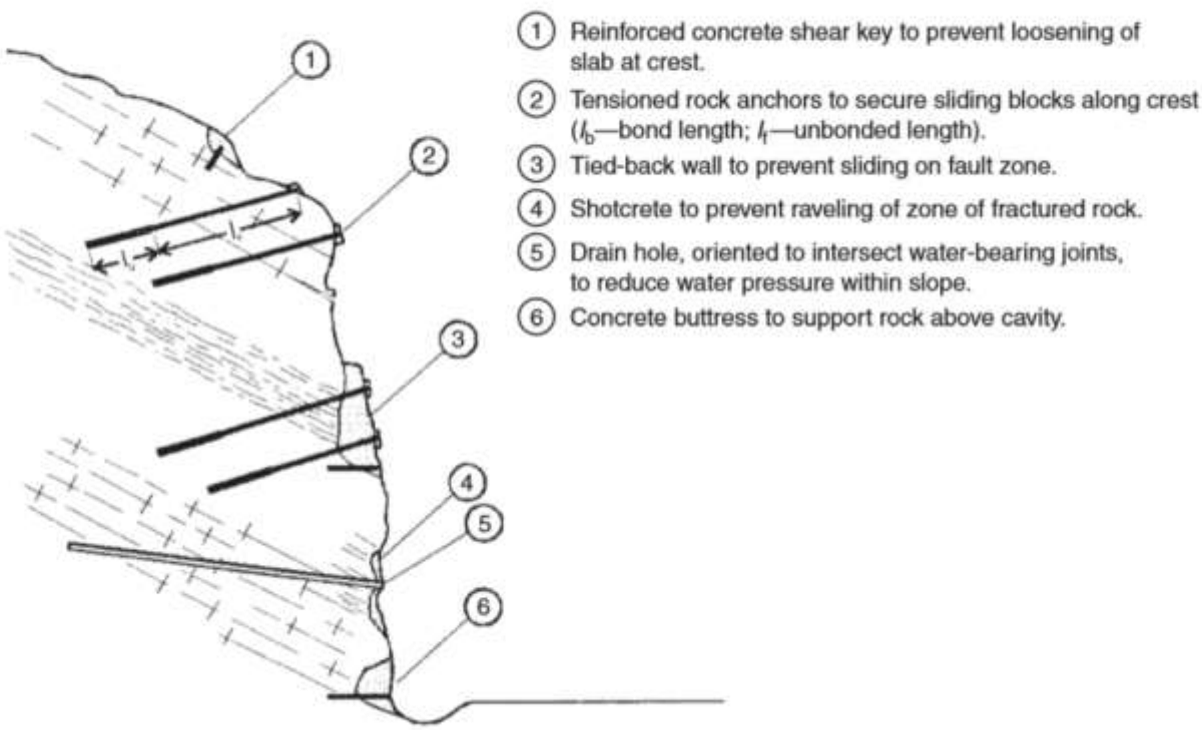
6.2 Findings

- Highly jointed folded and foliated strata was observed at RD 37 site
- Strata is highly susceptible to land sliding due to seepage and flow condition and existence of open joints
- Strength of Schist is extremely low compared to Marble due to its flaky nature which contributes to rockfall and landslide
- Existing cut was made without minding engineering parameters which is a major reason of instability
- Local fault at RD 37 + 124 was observed which contributed to instability

6.3 Recommendations

Assessment of the geotechnical profile of site using numerical modelling has led us to the following recommendations:

- Rockfall barrier design as specified in Chapter 5.
- Grouted tiebacks, rock bolts, anchors and dowels of appropriate length and spacing and orientation as proposed in Chapter 5.
- Redesigning the slope geometry i.e. proper benching of rock slope.
- Natural stabilization measures such as undertaking plantation in the region.
- Provision of road shelters along the motorway where rock slope is vulnerable to avoid economic and human loss.
- Wire meshing the rock slope can be performed for stability.
- Shotcreting can be done over bolted mesh to stabilize the slope.
- Weep drains can be provided to avoid the landslides to due to the seepage and presence of moisture in the joints.



6.4 Conclusions

- Site visits, scanline survey and lab investigation helped characterize the geological and geotechnical conditions of RD 37 site as described in Chapter 4.
- Geological mapping and scanline survey were performed for detailed investigation to generate a numerical model in SLIDE as described in Chapter 4.
- Kinematic Analysis was performed using DIPS as described in Chapter 4.
- Rock slope stability has been analyzed using SLIDE. Protection barrier and rock support has been designed using ROCFALL and SLIDE respectively in Chapter 5. Design is attached with Appendix.

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