THE ASSESSMENT OF ROCK SLOPE AT KM 8 ALONG IPOH-LUMUT EXPRESSWAY, PERAK

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by

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the requirements for the
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CERTIFICATION OF APPROVAL

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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

KHALISAH BINTI KAMAR SHAH

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And to God, who made all things possible.

ABSTRACT

This report fundamentally discusses the preliminary stage of the research done so far on the topic **The Assessment of Rock Slope at km 8 Along Ipoh-Lumut Expressway, Perak**. The objective of this study is to assess the rock slope conditions through slope stability analysis using stereonet projections. The type of rock failure that the slope is susceptible can be established from the stereonet projections. Geological data of the rock slope such as strike, dip, and dip direction will be collected from the field and plotted in the stereonet. Some of the project's challenges include the gathering of field data and finding the most suitable stabilizing methods according to the collected data. The three most common modes of rock slope failure, namely: (1) slab sliding; (2) wedge sliding; and (3) toppling are studied during this stage of the research.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF STUDY

Slope stability analysis is applied in the geotechnical field with the purpose of designing safe and functional rock and soil slopes. A rock and soil slope stability permits one to evaluate: (1) the optimal staged excavation or construction time sequence determination; (2) consolidation works such as retaining walls, drainage systems or rockbolting, which can stabilize a slope; and (3) the role which design parameters such as slope angle and excavation or embankment height play in the work stability.

Slope failures range from being circular in relatively homogenous materials in soils to being non-circular or planar in layered soils. However, in rocks just about every slope failures are controlled by the presence of discontinuities such as joints, faults and fractures. The mentioned discontinuities are planes of weakness across which there is little or no tensile strength. The feasibility of slope failures and the tendency to distribute depend on the extent, pattern and types of discontinuity present in the rock mass. Therefore, the assessment of rock slopes for their susceptibility to failure must incorporate a description system and a means for presenting orientation data in a form that can be used directly in stability analysis. This can be seen in the hemispherical projection.

1.2 PROBLEM STATEMENT

In Malaysia, rock slopes along roads and highways are a common sight. Rock failures along the roads are also a common occurrence not only in Malaysia, but also in other countries around the world. Apart from endangering motorists and road-users should the slope collapse, rock failures also demand rehabilitation and maintenance which are not only uneconomical, but time-consuming as well. The case study of this project is situated in Kilometer 8, along the Ipoh-Lumut Expressway. This project will target to design the appropriate stabilizing methods to cater the mode of rock failure for this particular slope.

1.3 OBJECTIVES

The goals of this research are:

- To gather geological data of the rock slope.
- To organize the data in graphical presentation.
- To use the data to carry out slope stability analysis.
- To derive the suitable methods of stabilizing the slope from the slope stability analysis results.

1.4 SCOPE OF STUDY

The scope of study for this project can be divided into two sections. The first part of the study will mainly revolve around the collection of geometrical data such as strike, dip, and dip direction. Other data such as visual and physical identification of the rock are also required. The second part of the study will focus on the methods of slope stability analysis and the stabilization methods according to the slope stability analysis results.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 Introduction

From visual inspection, the type of rock from the rock slope at Kilometer 8 along the Ipoh-Lumut Expressway has been identified as fine-grained granite. According to Lutgens and Tarbuck (2003), a rock is any solid mass of mineral or mineral-like matter that occurs naturally as part of our planet. Granite is one of the common rocks in Malaysia. It falls under the igneous rock category. Amphibole and muscovite are among the other minor constituents of granite. Granite rock is by-product of mountain-building and is very resistant to weathering.

Rock slopes has long peaked the interest of geotechnical engineers as part of the natural terrain and environment in which they work. Natural rock slopes are often used in forming the foundations of buildings, surface of penstocks, and also as abutments of bridges and dams. Artificial slopes on the other hand, are produced from excavations for transportation routes, buildings, dams, powerhouses, and portals (Goodman and Kiefer, 2000).

There are many ways that rock masses can fail. It is useful to identify and distinguish the various modes of rock mass failure for design purposes.

2.2 MODES OF FAILURE OF ROCK SLOPES

There are three idealized, simplified failure modes as explained by Hoek and Bray (1981). They are: (1) plane failure; (2) wedge Failure; and (3) toppling.

2.2.1 Plane Failure

Plane failure is very rare and it only occurs occasionally when the required geometrical conditions are fulfilled. The conditions of plane failure (on a single plane) according to Hoek and Bray (1981) are:

- The plane on which sliding occurs must strike parallel or nearly parallel (within ±200) to the slope face.
- The failure plane must 'daylight' in the slope face, meaning that the dip must be smaller than the dip of the slope face.
- The dip of the failure plane must be greater than the angle of friction of this plane.
- Release surfaces which provide negligible resistance to sliding must be present in the rock mass to define the lateral boundaries of the slide. Alternatively, failure can occur on a failure plane passing through the convex 'nose' of a slope.

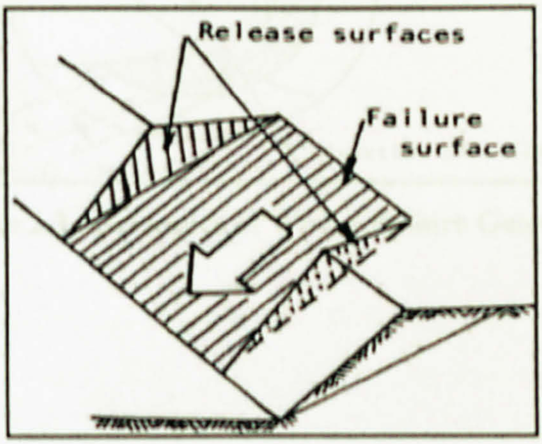


Figure 2.1. Plane Failure

2.2.2 Wedge Failure

Wedge failure differs from plane failure in which the structural characteristics upon which sliding can occur strike across the slope crest and where sliding takes place along the line of intersection of two such planes (Hoek and Bray, 1981).

The wedge geometry is outlined in Figure 2.2. for analysis purposes as shown below.

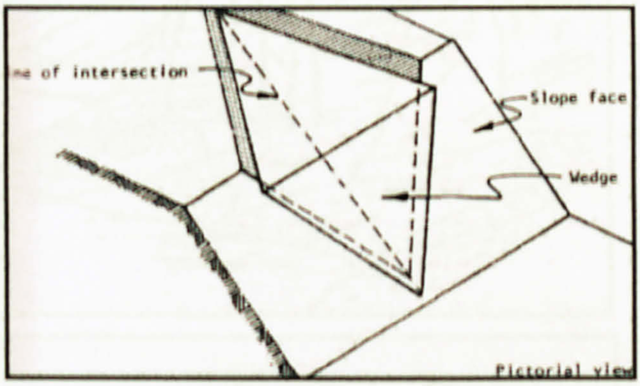


Figure 2.2. Wedge Failure Geometry

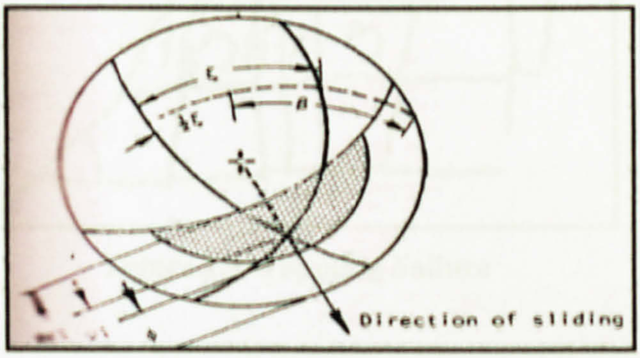


Figure 2.3. Stereoplot of Wedge Failure Geometry

2.2.3 Toppling Failure

Toppling failure is also known overturning of multiple columns (Goodman, 1989). Toppling failure is less common compared to block and wedge sliding. There are several types of toppling failure as described by Bray and Goodman:

- Flexural Toppling
- Block Toppling
- Block-Flexure Toppling

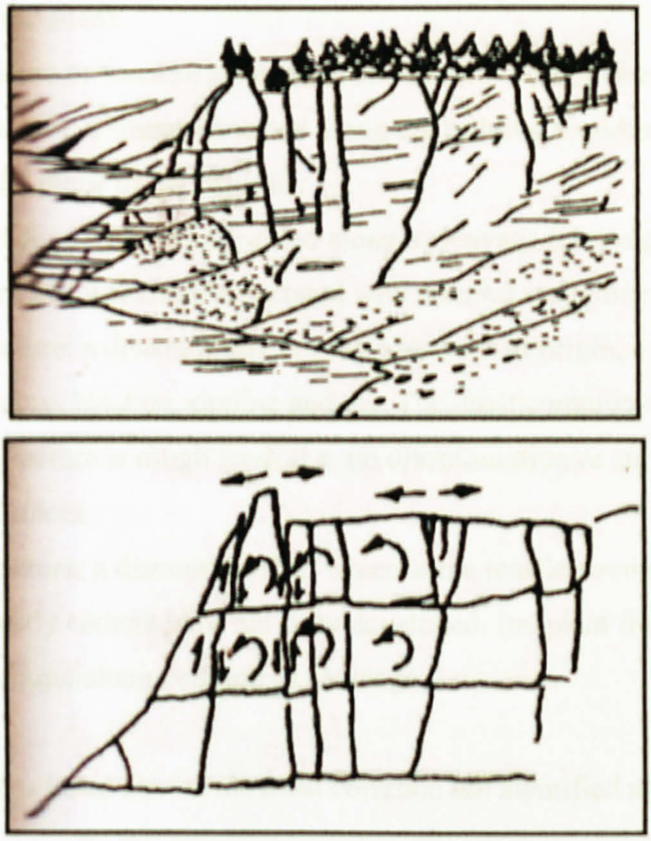


Figure 2.4. Toppling Failure

2.3 DISCONTINUITIES

Discontinuities are usually categorized according to the way that they are formed. Duncan C Wyllie and Christopher W Mah (2004) [8] states the usefulness of this categorization in geotechnical engineering is because each discontinuities in a category usually have similar characteristics in terms of shear strength properties and dimensions. Both properties can be utilized in the initial review of stability of a site.

2.3.1 Description of Discontinuities

According to BS 5930: 1999 *Code of practice for site investigations,* includes the following type of discontinuity [7]:

1) Joint: a discontinuity in the body of rock along which there has been no visible displacement.
2) Fault: a fracture or fracture zone along an identifiable displacement.
3) Bedding fracture: a fracture located along the bedding (bedding is a surface parallel to the plane of deposition).
4) Cleavage fracture: a fracture located along a cleavage (cleavage is a set of parallel planes of weakness associated with mineral realignment).
5) Induced fracture: a discontinuity not from geological origin, e.g. brought about by coring, blasting, ripping and etc. The most common characteristic of induced fracture is rough fresh (i.e. no discolouration or surface mineral coatings) surfaces.
6) Incipient fracture: a discontinuity in which some tensile strength is retained. It may be partly cemented or not fully developed. Incipient fractures are commonly found along bedding or cleavage.

Out of the discontinuities listed above, the most common are identified as joints and bedding fractures.

2.4 ROCK MASS CHARACTERISTICS

This data provides a guideline on indentifying the likely behaviour of the rock. The characteristics are explained by Wyllie and Mah (2004) as listed below:

2.4.1 Rock type

The rock type can be defined as the origin of the rock, (be it igneous, sedimentary or metamorphic), the colour, grain size, and mineralogy (Deere and Miller 1966). It is important for rocks to be defined as different rock types have vast differences from one to another in terms of performance (e.g. granite is usually more massive and possesses higher strength compared to shale).

2.4.2 Discontinuity type

There is a wide range of discontinuity types (Refer section 2.3). The shear strengths of each discontinuity types are different.

2.4.3 Discontinuity orientation

The orientation of discontinuities can be expressed as the dip direction (or strike) of the surface. The dip of the plane is the maximum angle of the plane to the horizontal angle (angle ψ), while the dip direction is the direction of the horizontal trace of the line of dip, measured clockwise from the north angle α. The strike and dip results can be directly plotted on a stereonet to analyze the structural geology.

2.4.4 Spacing

Discontinuity spacing can be mapped in rock faces and in drill core, with the true spacing being calculated from the apparent spacing for discontinuities inclined to the face.

2.4.5 Persistence

Persistence is the measure of the continuous length or area of the discontinuity.

2.4.6 Roughness

The roughness of a discontinuity surface is often an important component of the shear strength, especially when discontinuity is undisplaced and interlocked. Roughness becomes less important where the discontinuity is infilled, or displaced and interlock is lost. Roughness should be measured in the field on exposed surfaces. Usual practice would be to use the standard roughness profiles during the preliminary mapping. If there is a critical discontinuity that will control stability, then the values estimated from the profiles could be calibrated with a limited number of detailed measurements of roughness.

2.4.7 Wall strength

Shear strength of rough surfaces are influenced by the strength of the rock forming the walls of discontinuities. High stresses, compared to the wall strength are generated at local contact points during shearing, the asperities will be sheared off resulting in a reduction of the roughness component friction angle. There is often a reduction in rock strength during the initial stages of weathering on the discontinuity surfaces that may result in a declining roughness value. Usually, it is sufficient to estimate the compressive strength from the simple field tests as shown in Table 2.1 (ISRM, 1981b), or, by carrying out point load tests if core lump samples are available. The Schmidt hammer test is another method of approximating the compressive strength of rock at discontinuity surfaces.

Table 2.1 Classification of rock and soil strengths (ISRM, 1981b)

Grade	*Description*	*Field identification*	*Approx. range of uniaxial compressive strength (MPa)*
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100–250
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50–100
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer.	25–50
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0–25
R1	Very weak rock	Crumbles under firm blows with point of geological hammer and can be peeled by a pocket knife.	1.0–5.0
R0	Extremely weak rock	Indented by thumbnail.	0.25–1.0
S6	Hard clay	Indented with difficulty by thumbnail.	>0.5
S5	Very stiff clay	Readily indented by thumbnail.	0.25–0.5
S4	Stiff clay	Readily indented by thumb but penetrated only with great difficulty.	0.1–0.25
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort.	0.05–0.1
S2	Soft clay	Easily penetrated several inches by thumb.	0.025–0.05
S1	Very soft clay	Easily penetrated several inches by fist.	<0.025

Notes

2.4.8 Weathering

As described in (7), the diminution of rock strength due to weathering will also diminish the shear strength of discontinuities. Weathering also will deplete the strength of the rock mass due to the diminished strength of the intact rock. Weathering is divided into several categories. They range from fresh rock to residual soil. Weathering of rock takes the form of both disintegration and decomposition. Disintegration is the result of environmental conditions such as wetting and drying, freezing and thawing that disintegrates the exposed surface layer. Decomposition on the other hand, refers to the effects generated in rocks by chemical agents such as oxidation, hydration, and carbonation.

2.4.9 Aperture

In an open discontinuity where the intervening space is filled with water or air, the perpendicular distance separating the adjacent rock walls is called the aperture. Aperture is differentiated from the width of a filled discontinuity. It is useful in forecasting the likely behaviour of the rock mass. For example, hydraulic conductivity and deformation under stress changes. Potential reasons that induce aperture include scouring of infillings, solution of the rock forming the walls of a discontinuity, shear displacement, and dilation of rough discontinuities, tension features at the head of landslides and relaxation of steep valley walls following glacial retreat or erosion. Measurements of aperture can be done in outcrops or tunnels while taking extra precautions to discount any blast-induced open discontinuities.

2.4.10 Infilling

The material separating adjacent walls of discontinuities, such as calcite or fault gouge is termed as infilling. In order to forecast the behaviour of the discontinuity, a complete description of the filling material is compulsory. This also includes mineralogy, particle size, over-consolidation ratio, water content/conductivity, wall roughness, width and fracturing/crushing of the wall rock.

2.4.11 Seepage

Data on aperture can be obtained through the location of seepage from discontinuities. This is because the groundwater flow is almost completely confined in the discontinuities (secondary permeability). There are several categories of seepage. They range from very tight and dry to continuous flow that can scour infillings. Through these observations, water tables can be located. The flow quantities will help predict conditions during construction in cases of flooding and pumping requirements of excavations.

2.4.12 Number of Sets

The extent to which the rock mass can deform without failure of the intact rock is influenced by the number of sets of discontinuities that intersect each other. As block size decreases and number of discontinuity sets increases, the possibility of the block to rotate, translate and crush under applied loads gets higher.

2.4.13 Block size / shape

The discontinuity spacing, persistence and number of sets determine the block size and shape. Some examples of block shapes are blocky, tabular, shattered and columnar. Block size can be approximated through the selection of a few typical blocks and measuring the average dimensions.

2.5 ROCK MASS CLASSIFICATION SYSTEMS

Owing to the intricacy and uncertainty of geomechanical factors affecting rock construction, empirical design method is still applied in the current engineering field. From the empirical design method, several rock mass classification systems were proposed in the 1970s. the rock mass classification system has been utilized largely in the engineering design and construction of tunnels, slopes and foundations (Liu and Chen, 2005). The purpose of rock mass classification is to provide quantitative data and guidelines. The collected information can improve and simplify geologic formations that are initially complex. Lysandros Pantelidis (2008) stated that rock mass classification systems were proposed in order to reduce and identify high-risk failures and also to effectively prioritize preventive measures as well. It is also utilized as a means to evaluate the performance of rock cut slopes based on the most important inbuilt and structural parameters.

Table 2.2: Existing rock mass classification systems (Pantelidis L., 2008)

Name of the system	Abbreviation	Authors	Application	Comments
-	-	Ritter [30]	Tunnels	The first attempt for the formalization of an empirical approach to tunnel design.
Rock load	-	Terzaghi [31]	Tunnels	The earliest reference to the use of rock mass classification for the design of tunnel support.
Stand-up time	-	Lauffer [32]	Tunnels	Related to the stand-up time of an unsupported tunnel excavation.
Rock Quality Designation	RQD	De Deer [33]	General	Component factor of many classification systems.
Rock Structure Rating	RSR	Wickham et al. [34]	Small tunnels	First rating system for rock masses.
Rock Tunneling Quality Index	Q	Barton et al. [35]	Tunnels	They are the most commonly used classification systems for tunnels.
Rock Mass Rating	RMR	Bieniawski [28,29,36,37]	Tunnels and cuttings	A raw rating adjustment for discontinuity orientation for application in slopes was added in the 1979 version of the RMR system.
Mining Rock Mass Rating	MRMR	Laubscher [38-40]	Mines	Based on RMR (1973).
Rock Mass Strength	RMS	Selby [41,42], Moon and Selby [43]	Cuttings	Based on natural slope database.
Slope Mass Rating	SMR	Romana [44], Romana et al. [45]	Cuttings	Based on RMR (1979). The most commonly used classification system for slopes.
Slope Rock Mass Rating	SRMR	Robertson [46]	Cuttings	Based on RMR. The classification is provided for of weak altered rock mass materials from drill-hole cores.
Chinese Slope Mass Rating	CSMR	Chen [47]	Cuttings	Adjustment factors have been applied to the SMR system for the discontinuity condition and slope height.
Geological Strength Index	GSI	Hoek et al. [48]	General	Based on RMR (1976).
Modified Rock Mass Rating	M-RMR	Ünal [49]	Mines	For weak, stratified, anisotropic and clay bearing rock masses.
Geological Strength Index	GSI	Hoek et al. [50], Marinos and Hoek [51,52], Marinos et al. [53]	General	For non-structurally controlled failures.
Rockslope Deterioration Assessment	RDA	Nicholson and Hencher [54], Nicholson et al. [55], Nicholson [56-58]	Cuttings	For shallow, weathering-related breakdown of excavated rockslopes.
Slope Stability Probability Classification	SSPC	Hack [59], Hack et al. [60]	Cuttings	Probabilistic assessment of independently different failure mechanics.
Volcanic Rock Face Safety Rating	VRFSR	Singh and Connolly [61]	Cuttings (temporary excavations)	For volcanic rock slopes to determine the excavation safety on construction sites.
Falling Rock Hazard Index	FRHI*	Singh [62]	Cuttings (temporary excavations)	Developed for stable excavations to determine the degree of danger to workers
-	-	Mazzaccola and Hudson [63]	Natural slopes	A rock mass characterization method for the indication of natural slope instabilities

According to Liu and Chen (2005), the most commonly used rock mass classification systems are the Rock Structure Rating, RSR (Wickham et al., 1972), the Rock Mass Rating, RMR (Bieniawski, 1973, Bieniawski 1975, Bieniawski 1979, and Bieniawski 1989), and the NGI Q-system (Barton et al., 1974).

2.6 THE ROLE OF GROUNDWATER IN SLOPE STABILITY

2.6.1 Introduction

Water can impose highly destructive effects on surface mining operations. Rock cuts for road and highway purposes are similar to pen pit mining.

One of the most noteworthy difficulty related to groundwater is the effect inflicted by water pressure on the angle at which slopes can be excavated. According to Atkinson (2000), the presence of water pressure in discontinuities such as joints, bedding planes, fractures, and etc. in a rock mass reduces the effective stresses on such discontinuities with a consequent reduction in shear strength.

In cases where a 'wet' slope is present, dewatering the slope is usually a more economical alternative rather than flattening it. There are also other problems associated to water. Among the problems are:

a) Problems related with wet ore (for example, material bind-up in crushers, excessive moisture content in coal, disintegration of kimberlite) or overburden / waste rock (for example, stacking problems).
b) The direct costs of water management (for example, wells, pumps, pipelines, electric power, dewatering drifts).
c) The demand for waterproof explosives.
d) Higher maintenance for equipment (for example, excessive tire wear on wet ground).
e) Slowdowns or shutdowns that occur periodically caused by pump failures.
f) Engineering costs (the need for specialized staffs or consultants).

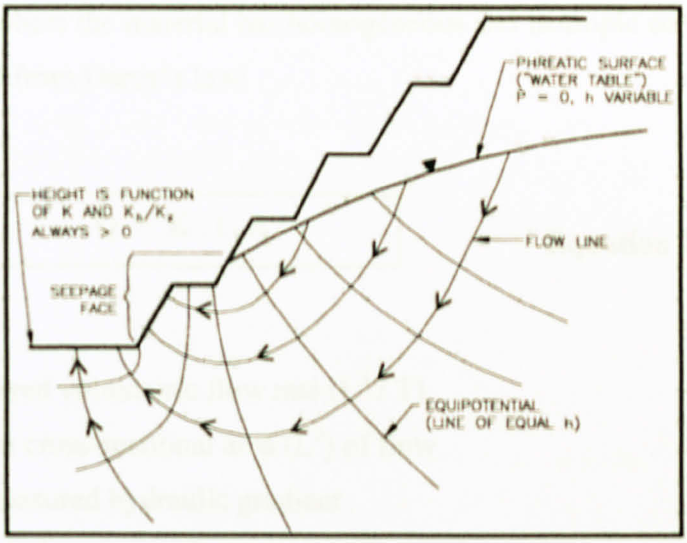


Figure 2.5 Groundwater flow system in a pit slope (Atkinson, 2000)

Figure 2.5 shows a flow net which indicates the theoretical flow lines and the equipotential lines. Equipotential lines tell the location where the hydraulic head is equal in a flow field. Hydraulic head can be defined by:

$$\boxed{h = P / \rho g + z = h_p + h_z}$$

Equation 1.0

Where:

h = hydraulic head

p = pressure

ρ = water density

g = gravitational constant

z = elevation (z = 0 at sea level)

h_p = pressure head

h_z = elevation head

The hydraulics conductivity of groundwater flow is one of the most significant property of a soil or rock.

In a case where the material has homogeneous and isotropic conductivity, it can be derived from Darcy's law:

$$\boxed{Q = K \cdot i \cdot A}$$ **Equation 2.0**

Where:

Q = measured volumetric flow rate (L^3 / T)

A = known cross-sectional area (L^2) of flow

i = the measured hydraulic gradient

K = empirical constant of proportionality

There are about 13 orders of magnitude ranges for geologic materials in their hydraulic conductivities as stated by Atkinson (2000). It is quite common to have materials that differs as much as four orders of magnitude situated side by side to each other. Figure 2.6 demonstrates the ranges of hydraulic conductivity for consolidated and unconsolidated materials.

Fractured rock masses with heterogeneous (whereby it varies from rock unit to rock unit and often within a rock unit) and anisotropic (varies with direction) hydraulic conductivities often takes place in hard-rock mines and almost fully traceable to the fractures. Put differently, usually the contribution of total hydraulic conductivity of the rock mass can be ignored from the rock matrix.

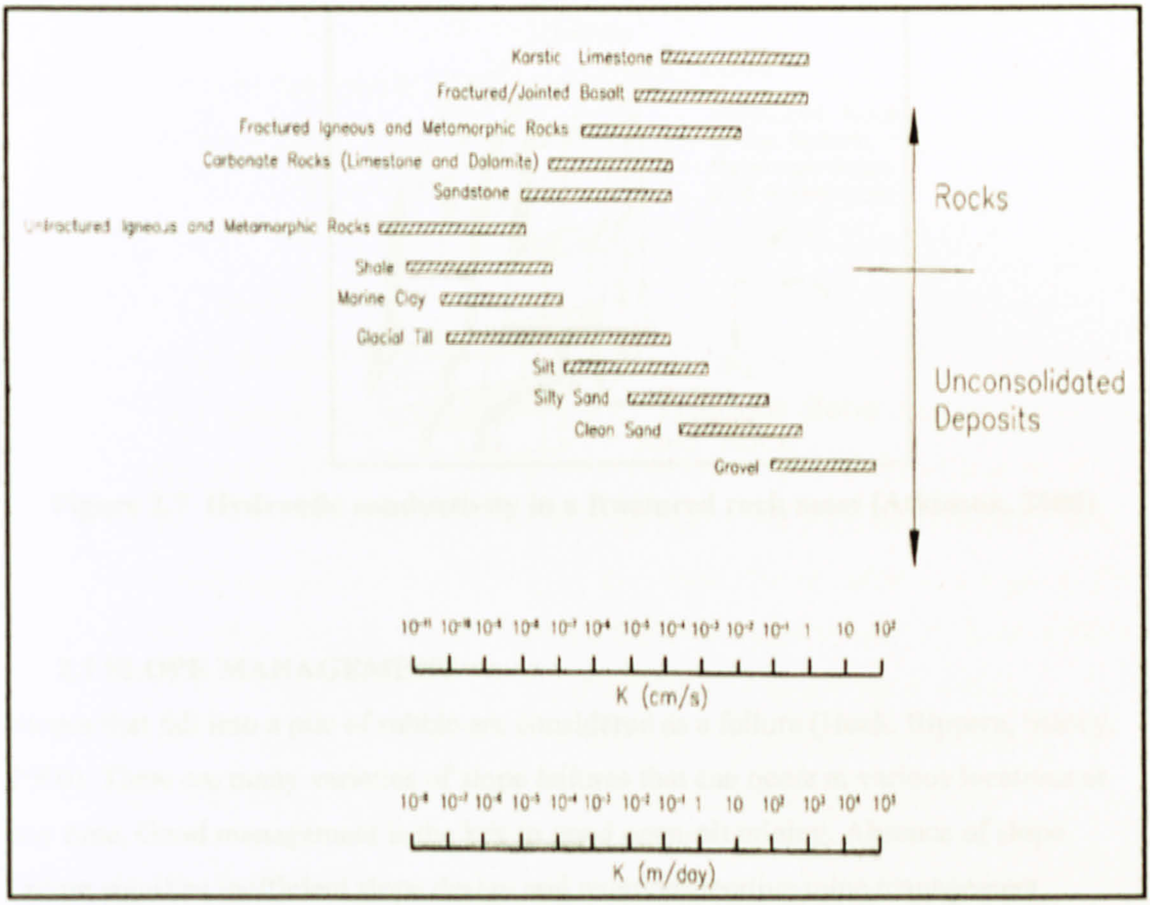


Figure 2.6 Hydraulic conductivity of various geologic materials (Atkinson, 2000)

The concept of homogeneity in rock masses are rather simple to understand as it has the ability to affect ore grade in addition to hydrogeologic and geotechnical properties. Figure 2.7 indicates a typical rock mass. There are at least three joint sets apart from the heterogeneities related to fracture and shear zones. The joint sets most likely result in different hydraulic conductivities in different directions within the rock mass. Should one of the joint sets is slightly more conducive than the others, this can be exploited with the dewatering system.

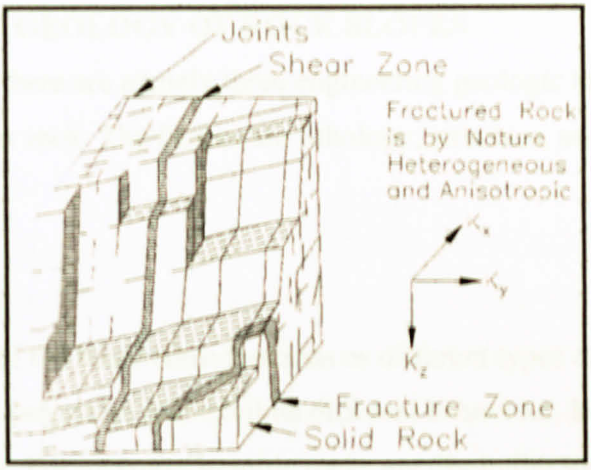


Figure 2.7 Hydraulic conductivity in a fractured rock mass (Atkinson, 2000)

2.7 SLOPE MANAGEMENT

Slopes that fall into a pile of rubble are considered as a failure (Hoek, Rippere, Stacey, 2000). There are many varieties of slope failures that can occur at various locations at any time, Good management is the key to good open-pit mining. Absence of slope failure signifies inefficient slope design and overconservative mine management.

2.7.1 Tools For Slope Instability Detection

The most reliable method for detecting slope instability is by monitoring slope movement. There are several measurement tools based on observations of many targets placed at selected locations on the benches of the mine that can be utilized. Electro-optical distance measuring (EDM) equipment and global positioning by satellite (GPS) systems are among the methods which offers sufficient accuracy within less than 1cm for measuring distances of 1km or more. These are used to monitor the relative positions of the targets regularly and are adequate enough to give advance warning of most slope instabilities.

2.8 ENGINEERING GEOLOGY OF ROCK SLOPES

Tan (2000) explains that there are namely three engineering geologic factors that govern the stability of a particular rock. The factors are lithology, structure, and weathering grade.

2.8.1 Lithology

Lithology is one of the basic considerations as different types of rock contains different material properties and exhibits different behaviour. Basically lithology is the type of rock. For instance, granitic rocks are dissimilar to shales, schists or limestone. It is concluded that the inherent geological structures connected with each rock type are also different apart from the different nature and origin of the mentioned rock types. Granitic rocks are frequently intersected by three or more sets of major joints that controls the stability or the rock slope at certain locations. Shales and sandstones are dominated by bedding planes instead, and those bedding planes are responsible for the stability of the cut-slopes.

The differences in types of rocks yield different types of failure and failure modes along highways or roadway projects. Some examples of such failures happened along the Karak Highway with granitic rock slopes (Tan, 1987), Senawang-Air Keroh Highway with graphitic schist slopes (Tan, 1992), and the Ipoh-Simpang Pulai Highway with limestone cliffs (Tan 1999). In hard rocks like granite and limestone, failure modes are controlled by major joints or faults.

2.8.2 Structure

Geologic structures involve characteristics such as faults, joints, bedding planes, foliation, folds, and etc. As a whole, major fracture planes like faults and major joints are the more critical structures since they represent breaks / discontinuities or weaknesses in the rock mass. As an example, rock slope stability is restricted by the fracture planes. Assessment of rock slope stability through measurements and analysis of discontinuities can be carried out using stereonets, (Hoek &

Bray, 1974), (Tan, 1999). Other geological structures that play a significant role in other engineering works are dykes and sills.

2.8.3 Weathering

In humid, tropical regions like Malaysia, weathering is especially crucial since intense chemical weathering reduce rocks into weaker soil-like materials consequently in thick soil mantles over bedrock formations. Standard six-grade weathering classification scheme as proposed by Little (1969), ISRM (1977) has been frequently adopted by many authors in construction projects including rock slopes.

2.9 CASE STUDY : BUKIT LANJAN ROCK SLOPE FAILURE

2.9.1 Introduction

(Komoo et al., 2003) reported a large scale slope failure took place at kilometer 21.8 of the Bukit Lanjan Interchange on the New Klang Valley Expressway (NKVE) on the 26 November 2003 at 7:16am. The rock slope failure included an estimated 35000 m^3 of chiefly angular blocks of rock debris in various sizes which came to rest on the expressway. The failed materials blocked the entire expressway which forced the road to be closed to public.

The steep cut slope at Bukit Lanjan failed on the southern end of an approximately north-south trending cut. The northern margin of the failure exposed a continuous major discontinuity and the failure is wedge-shaped. Large scale rock slope failure such as this is rare in Malaysia.

2.9.2 Rock Mass Properties

The rock slope failure site at Bukit Lanjan is underlain by the granite bedrock. Granite is the most common igneous rock mass associated with the Main Range formation. For fresh to slightly weathered igneous rock, orientations of discontinuities are mainly the controlling characteristics. The failure material samples are used to determine specific rock type. The materials are very strong and unlikely to be the cause of failure.

2.9.3 Discontinuities

In rock mass, there are generally two types of discontinuities, major and minor discontinuities. Major discontinuities are usually continuous and can be traced from aerial photograph of whole slope face. Minor discontinuities are several sets of joints or fractures. The rock mass here possesses numerous discontinuities.

2.9.4 Degree of Weathering

Five boreholes were drilled around the failed slope. The discovered failure debris can be considered as mainly fresh rock material with minor amounts weathered rock materials and soil materials.

2.10 ANALYSIS OF FAILURE

A kinematic slope stability has been conducted using stereographic projection technique. Back analysis assessing the stability condition of the wedge was conducted using the simplified method with both wet and dry conditions. Consequently, the resulting factor of safety (F.O.S.) showed that the failure was likely to occur under wet and high water pressure conditions.

2.11 CAUSE OF FAILURE

Several triggering factors are identified as the causes of failure. they are the prolonged rainfall prior to the failure which affected the rock mass properties and also because of unfavourable discontinuity orientations.

2.12 REHABILITATION OPTIONS

Two rehabilitation measures chosen are the rock slope re-profiling which require d the rock slopes to be re-profiled to a gentler angle. Based on data analysis, an overall slope profile of 48° was considered appropriate.

Slope re-profiling reduces possibility of large scale wedge failures and / or planar failures. Localized stabilization measures like rock bolts, rock anchors, dowels, shotcrete, and also drainage works were incorporated.

The other rehabilitation measure is rock slope reinforcement. This can be achieved by means of rock anchors, rock bolts, dowels, concrete buttress, netting, shotcrete and horizontal drains.

CHAPTER 3

METHODOLOGY / PROJECT WORKS

3.1 PROJECT IDENTIFICATION

For the first part of the research, focus is given to geological field data gathering. Readings of dip, and dip direction are taken and recorded. Other physical details are recorded in the Discontinuity Survey Data Sheet.

3.2 DIP

Pusch (1995) explained that dip is the inclination of a structural plane expressed as the angle between this plane and the horizontal plane. The figure is associated with an indication of the direction. Dip is measured using a Brunton Pocket Transit.

3.3 STRIKE

Strike is the orientation in the horizontal plane of the line of intersection of a structural plane and the horizontal plane. It is expressed in relation to the direction of Geographic North (Pusch, 1995). Strike can be measured using a Brunton Pocket Transit or any compass.

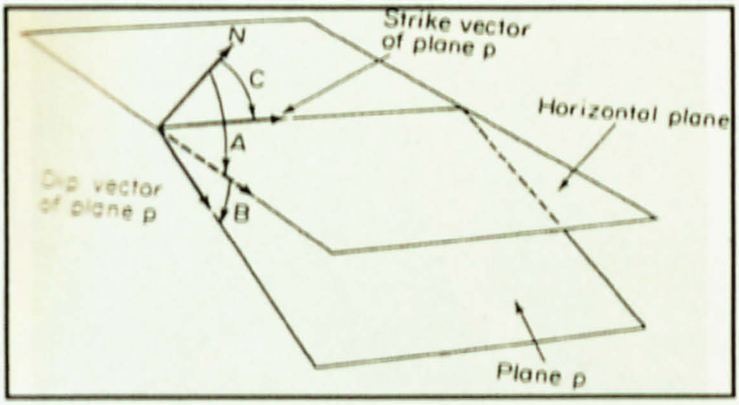


Figure 3.1. Definition of Strike and Dip

3.4 BRUNTON POCKET TRANSIT

A Brunton Pocket Transit is used to calculate the strike and dip of geological features on the field. These geological features include faults, contacts foliation, and etc. Strike can be calculated by leveling the bull's eye level of the compass along the plane that needs to be measured.

Dip on the other hand, can be calculated by laying the compass on its side, so that it is perpendicular to the strike measurement and rotating the horizontal level until the bubble is stable before recording the reading.

Figure 3.1b: Using the Brunton Pocket Transit

3.5 CASE STUDY

Coordinate of the exact location of the rock slope is N 04° 32.851' E 101° 01.789' at elevation of ±36 feet.

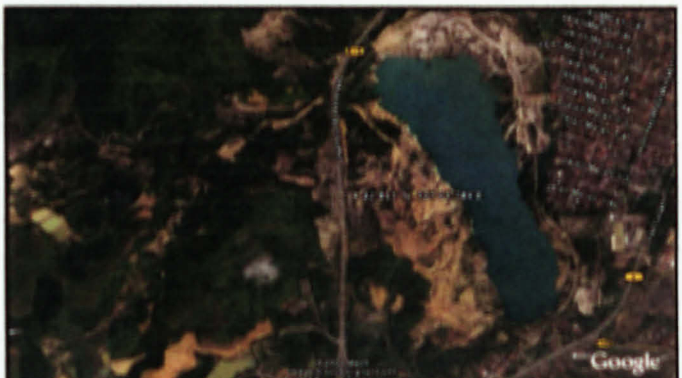


Figure 3.2. Google Earth Image of the Location

Figure 3.3. Zoomed Google Earth Image of the Location

3.6 PLOTTING TECHNIQUES

To successfully plot on a stereonet, visualizing is essential. A general image should be imagined at first of where the great circle or point that will be plotted will fall on the stereonet (Stephen Marshak and Gautam Mitra, 1988).

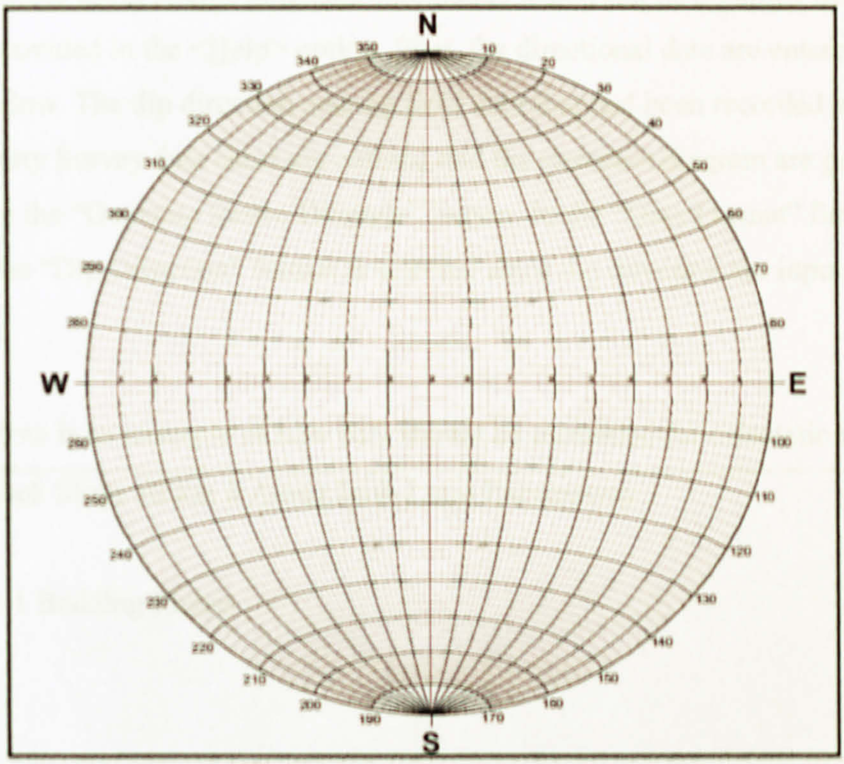


Figure 3.4: Stereonet, Schmidt net, or equal-area net

The data collected from the field will be plotted on the stereonet as shown above. From the plot, structural geological features that influence rock slope stability can be identified.

3.7 PLOTTING DATA

Manual plotting is achievable if there are only a few data to consider. However, in this case it is possible to have hundreds of readings. To manually plot all of the data would be time-consuming and disorganized. There are many computer programs for plotting stereograms and also for data orientation analysis. Among the popular ones are Stereonet, Stereo-nett, Stereoplot, Quickplot, Stereopro and STEREO. For this particular project, STEREO, a member of RockWare Utilities collection is used. STEREO has the ability to read the strike and dip of planar and/or linear data and create a stereonet diagram plot based on the keyed-in data. Apart from the stereonet diagram, this program also compute statistics stored in an optional report.

Stereo operates using simple language commands which can be obtained from the example provided in the <Help> option. First, the directional data are entered into a text editor window. The dip direction and dip field data that had been recorded earlier in the Discontinuity Survey data sheet are entered and the stereonet diagram are generated by clicking on the "Generate Stereo Diagram" button. In the "Data Format" function window, the "Dip Direction" format is selected since dip direction are inputted instead of strike.

Shown below is an example of how data should be formatted for orientation analysis:

```
TITLE: Rock Slope At km 8 Along Ipoh-Lumt Expressway

PLANES: 1 Bedding Planes
44 80
48 84
53 80
56 81
```

0 90
47 90
164 80
59 76
92 90
170 78
100 58
163 89
53 85
163 89
53 85
163 89
170 80
48 77
163 87
56 73
164 83
69 80
170 85
120 53
163 81
166 81
150 90
85 44
31 74
166 77
57 74
171 83

END-DATA:

3.8 CONSTRUCTING A GREAT CIRCLE AND A POLE REPRESENTING A PLANE

Step 1: Trace the circumference of the net and mark the north point with the tracing paper located over the stereonet from the centre of the pin. The dip direction is measured off clockwise from north and this position is marked on the circumference of the net.

Step 2: The tracing paper is rotated about the centre pin until the dip direction mark lies on the W-E axis of the net. The dipping is measured from the outer circle of the net and the great circle which corresponds to the plane dipping at this angle is traced. The position of the pole is found by measuring the dip angle from the centre. The pole lies on the projection of the dip direction line.

Step 3: Rotate the tracing paper back to its original position so that the north mark is facing north on the net. After this step, the great circle and the pole representing a plane dipping with that particular dip in the particular dip direction will appear.

3.9 CONSTRUCTING THE LINE OF INTERSECTION OF TWO PLANES

Step 1: To draw the great circle, mark the dip direction on the circumference of the net. The tracing is rotated until this mark lies on the W-E axis and tracing the great circle with respect to the dip.

Step 2: The tracing is rotated until the intersection of the two great circles lies along the W-E axis of the stereonet.

Step 3: The tracing is rotated again until the north mark on the tracing faces the north point of the stereonet.

3.10 FIELD DATA COLLECTION

3.10.1 Setting Up For Chainage Measurements

In order to identify joint sets from the data collected and to fill in the Data Sheet For Discontinuity, the chainage data should be obtained beforehand. The set up for chainage measurements can be done simply using wooden stakes, rope, measuring tape, and a long stick. The set up is explained in steps below:

1) A wooden stake is driven into the ground for every 15-20 metres interval.
2) The rope is tied firmly to the stakes from one point to another.
3) After measuring the dip direction, the long stick is used to extend the length of the discontinuity towards the line of rope.
4) The length where the stick is extended over the line of rope is marked and measured.
5) The process is repeated for each point collected.

3.10.2 Filling in Data Sheet For Discontinuity

To fill in the data sheet, apart from measurements of dip and dip direction, persistence, and aperture, physical evaluation of the other properties such as infilling, consistency, and roughness is also necessary. This is gauged from experience and own judgment based on rough observations.

3.20 METHODS OF STABILIZATION

There are generally four categories of stabilization methods for rocks (Franklin & Maurice (2000). They are:

3.20.1 Drainage Systems

Drainage systems help increase stability by decreasing the pressure of water in the slope. Adequate drainage of groundwater and surface runoff is an important part of stabilization method. The hydraulic head in a rock slope is lowered which resulted in reduced stripping and increased ore recovery. Among the drainage systems available for application are collector ditches; which reduces erosion, trench drains; which lowers water table directly beneath the slope face to counteract shallow sliding, gravity-drain holes; which intersect as any water conducting joints as possible. Drainage galleries are used for large-scale slides while vertical drains are applied for short-term dewatering during construction.

3.20.2 Cut and Fill

Cut and Fill methods are applied by placing or removing material at the toe or crest in order to reduce the overall slope angle.

3.20.2.1 Recontouring

Recontouring reduces overall angle by unloading the slope crest, loading the toe, general excavation and benching, or by a combination of several methods to enhance slope stability. Toe berms are usually combined with gabion walls or reinforced earth walls.

Slide debris has a stabilizing influence, however it can interrupt with natural drainage channels by loosening, softening, and reducing shear strength. Slide debris should be removed often and replaced by free-draining material, or excavated and recompacted in thin lifts with drainage blankets.

3.20.3 Erosion Protection Systems

Erosion Protection Systems functions in reducing or preventing erosion of the slope face or toe. Toe erosion is mainly caused by river flow or wave action. This is mainly controlled using concrete or crib walls, gabion walls, riprap, groynes, or spur dikes. Surface erosion can be prevented by hydraulic seeding, encouraged vegetation, and surface drains.

3.20.4 Reinforcement Systems / Retaining Systems

Reinforcement and retaining system supplied sufficient support to suppress the beginning of slides and falls. Small scale slab, wedge, and toppling failures less than 3 to 4 metres deep are stabilized using rockbolts and dowels. However, where the joints are closely spaced (block size less than 200mm) or the rock is loose, bolting is unsuitable. This is because the drilling vibrations will risk disintegration and rockfalls. The rock face should be made safe by thorough scaling before reinforcement. Bolts can be installed on a regular grid or spot-located to anchor critical blocks. Steel components need to be protected against corrosion for permanent stabilization.

Retaining structures can be used alone or together with high-capacity soil and rock anchors to stabilize potentially unstable rock cliffs, translational rock slides and large boulders. Open-pit mine slopes are stabilized by anchoring. To distribute load to the rock, steel plates or cast concrete blocks are used as bearing pads. Anchors must be deep enough to be seated in fixed firm rock.

Slaking and raveling can be effectively prevented by shotcreting combined with bolting and anchoring which binds loose blocks together into a coherent skin and seals the rock face to keep it from wetting and drying. It provides a fast, mechanized and simple solution to many rockfall problems. To relieve water pressures, weep holes are drilled and pipes installed behind the shotcrete to drain water from permeable strata.

Anchored cable nets and mesh restrain masses of small loose rocks or individual rocks up to 2 to 3 meters. Anchored mesh is similar in concept to anchored cable nets although much lighter and less expensive. Welded or woven steel mesh is pulled tight to the rock face with rockbolts.

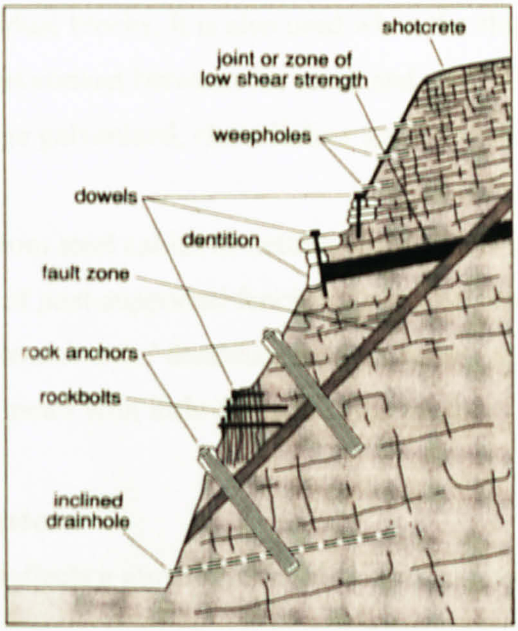


Figure 3.5 Various methods for stabilizing rock slopes (Geotechnical Control Office, Hong Kong, 1984)

3.21 PROTECTION METHODS

In other cases where prevention against stability proved to be uneconomical or impractical, there are three protection systems:

3.21.1 Catch Systems

Catch systems act as a passive defense that keeps falling rock from damaging downslope structures.

A bench is cut horizontally, a ditch is cut down, while a berm is built up. All three methods are easy to maintain and economical. To intercept falls before they could gather momentum, slope ditches are employed at the top of a scree slope to catch rockfalls before they could roll down. Toe ditches such as roadside ditches are designed to catch rocks at the toe of the slope.

Should there be any lack of space for the ditch's width, catch net, catch wall, draped mesh curtain, or draped mesh blanket are sufficient as ditch replacements. Unlike anchored mesh which functions to prevent rockfalls from gaining momentum and velocity by guiding them into a toe ditch, draped mesh are for smaller individual blocks. It is also used when the slope is uniform enough for continuous contact between the mesh and rock. The material of the mesh is 9- or 11-gauge galvanized, chain link or gabion wire.

Catch fences made from steel cables or netting absorb more energy compared to the more rigid types of post-supported fencing. Cable-suspended net systems that are anchored to sound rock / deadman anchors buried in soil / scree can catch blocks up to 1 meter with little damage to the mesh.

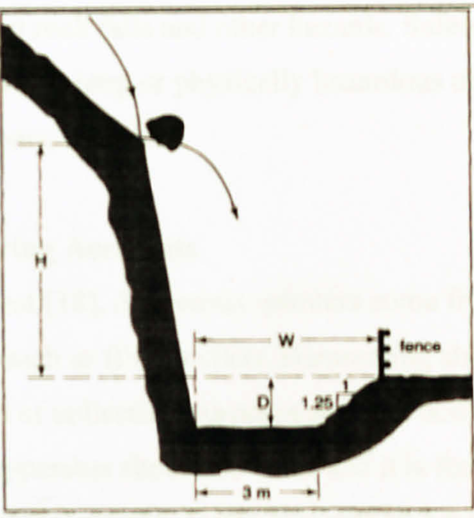
3.21.2 Deflection Systems

Deflection systems deflects a moving rockfall or causes it lose sufficient energy to reduce hazards.

3.21.3 Avoidance Systems

Totally avoid the slope instability through systems such as relocation, tunneling, or route diversion.

Table 3.1 Design citeria for shaped ditches to catch falling rock (after Ritchie, 1963, and Piteau and Peckover, 1978)



Rock slope angle	Height (m)	Fallout area width W (m)	Ditch depth D (m)
Near vertical	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	> 20	6.1	1.2
0.25 or 0.3:1	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	20 to 30	6.1	1.8*
	> 30	7.6	1.8*
0.5:1	5 to 10	3.7	1.2
	10 to 20	4.6	1.8*
	20 to 30	6.1	1.8*
	> 30	7.6	2.7*
0.75:1	0 to 10	3.7	1.0
	10 to 20	4.6	1.2
	> 20	4.6	1.8*
1:1	0 to 10	3.7	1.0
	10 to 20	3.7	1.5*
	> 20	4.6	1.8*

*May be 1.2 m if catch fence is used.

3.22 HEALTH, SAFETY AND ENVIRONMENT

Risks while carrying out geological fieldwork can be reduced by knowledge, experience, and suitable safety precautions. The risks should be assessed as far as possible in accordance to the weather, conditions of the day, and the types of persons involved.

3.22.1 Attire

Hard hats or safety helmet should be used at all times on the field as a safety precaution against rock falls and other hazards. Safety boots are also required to protect the feet from sharp or physically hazardous objects such as broken glass, rusty nail, steel bars, and etc.

3.22.2 Hammering Accidents

According to West [18], dangerous splinters come from mainly hammering on very hard rocks such as flint or chert. Hammering should only be done for specific research or collection purposes and it is best to minimize it. Goggles or eye-protection apparatus should be worn and it is forbidden to hammer when there are other people nearby. Using hammer on a hammer also causes splintering.

3.22.3 Snake Bites

Care should be taken when walking on grass or footpaths. If not disturbed, snakes generally will not attack. Hospital treatment for anti-venom serum will be needed in case of a poisonous snake bite.

3.22.4 Risk to Students Working Alone

There have been reports of accidents occurring when students are working alone without obtaining the help needed. Some students have also been attacked while working on the field alone. It is better to work in groups to reduce potential risks.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 LENGTH

From the measurements carried out on site, the length of the rock formation is approximately 267.4m.

4.2 VISUAL OBSERVATION

From visual observation, some parts of the rock have already experienced minor failure. This is proven by the broken pieces found at the toe along the roadside drain. A closer observation reveals that the type of failure can be categorized as wedge failure. The rock is identified as fine-grained granite.

Through panoramic photographs taken from the opposite side of the road, discontinuities like faults for example, are identified. Faults that occur in sets can be seen from the photographs.

Figure 4.1: Fault lines

Figure 4.2: Fault lines

Figure 4.3: The rock covered by vegetation, hindering visual observation

4.3 DATA SHEET FOR DISCONTINUITY SURVEY

Directional data are collected and recorded into the data sheet for Discontinuity Survey. It contains general information such as site location, date, and also the nature and orientation of discontinuities such as chainage, type, dip, dip direction, persistence, aperture, roughness, differentially weathered zone width, waviness wavelength, waviness amplitude, and water. A sample of this Discontinuity Survey data sheet can be found in Appendix A.

Table 4.0 Discontinuity Survey Data Keyed in Microsoft Excel

DISCONTINUITY SURVEY DATA SHEET

GENERAL INFORMATION

Site: Ipoh-Lumut Expressway   Date: 20-02-09   Sheet no: 01 of 01

NATURE AND ORIENTATION OF DISCONTINUITIES

Chainage (	Type	Dip	Dip Directi	Persistenc	Aperture	Infilling	Consistenc	Roughness	Weathered	Waviness (	Waviness (	Water	Remarks
206	1	80	44	27	4	1		4				1	
306	1	84	48	26	5	1		3				1	
385	1	78	170	53	3	1		2				1	
422	1	80	164	80	7	1		2				1	
458	1	80	53	10	5	1		4				1	
497	1	78	170	14	5	1		2				1	
636	1	81	56	8	5	1		2				1	
639	2	90	0		5	1		2				1	
639	1	90	47	52	4	1		2				1	
729	1	80	170	100	4	1		2				1	
806	1	90	92	10	7	1		2				1	
823	1	89	163	33	5	1		2				1	
827	1	89	163	43	5	1		2				1	
830	1	76	59	12	7	1		2				1	
835	1	89	163	30	5	1		2				1	
886	1	58	100	28	5	1		2				1	
893	1	87	163	34	6	1		2				1	
967	1	83	164	190	5	1		2				1	
1044	1	85	53	5	5	1		2				1	
1110	1	85	170	115	6	1		2				1	
1112	1	81	52	19	5	1		2				1	
1142	1	81	163	15	7	1		2				1	
1235	1	81	166	953	7	1		2				1	
1293	1	77	48	12	4	1		2				1	
1374	1	73	56	28	4	1		2				1	
1553	1	53	120	9	5	1		2				1	
1575	1	80	69	20	5	1		2				1	
1588	1	90	150	150	7	1		2				1	
2107	1	77	166	52	5	1		2				1	
2602	1	74	31	79	6	1		2				1	
2782	1	44	85	150		1		2				1	
3154	1	74	57	124	6	1		2				1	
6325	1	83	171	120	6	1		2				1	
6725	1	82	44	55	5	1		2				1	
7064	1	78	138	25	7	1		2				1	
7113	1	82	174	170	7	1		2				1	
10053	1	77	180	17	5	2		3				1	
10806	1	88	70	7	5	2		3				2	
11076	1	74	48	26	4	2	6	3				2	
11369	1	76	174	164	3	2		3				3	
11524	1	81	138	170	4	2	6	3				4	
12169	1	83	78	19	4	2	7	4				1	
12195	1	77	86	27	4	2		3				1	
12196	1	51	94	15	4	2		4				1	
12238	1	61	157	80	4	2		3				1	
12263	1	65	142	80	4	2	7	4				1	
12371	1	90	120	10	4	2	7	4				1	
12461	1	78	83	14	4	2	7	4				1	

4.3.1 STEREO Plots

Below is the stereonet diagram generated using STEREO and data collected from 20th February 2009 to 23rd February 2009.

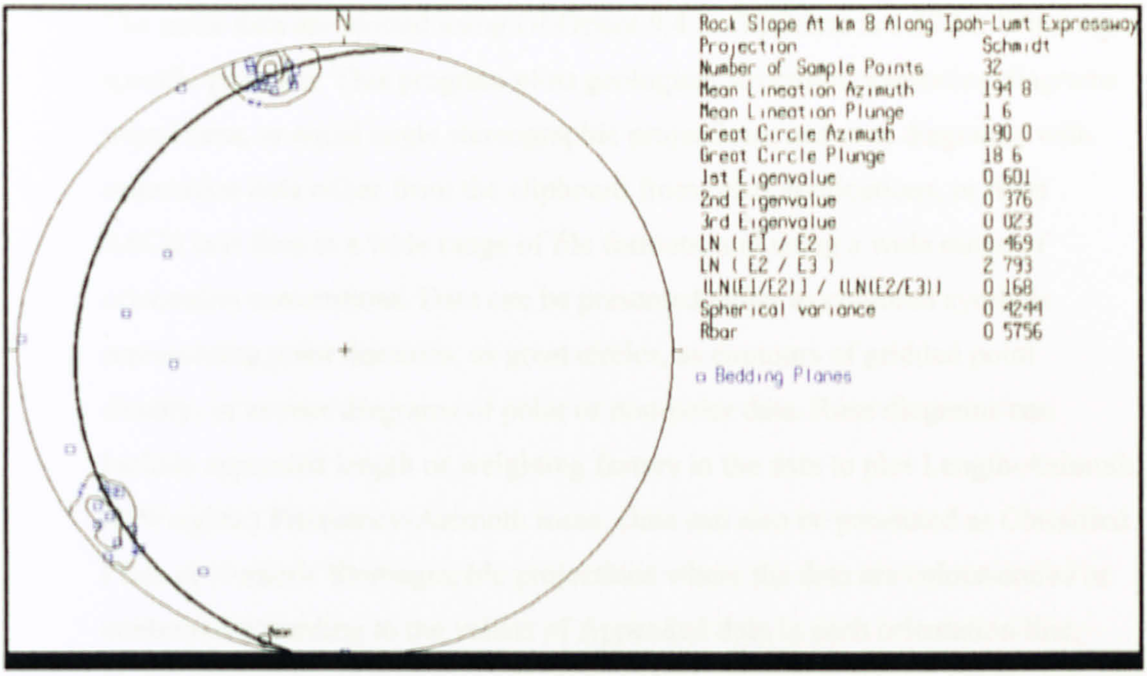


Figure 4.4: Stereonet diagram projection for 32 sample points

Statistical information regarding the diagram above are as shown here:

Projection .... Schmidt

Number of Sample Points 32

Mean Lineation Azimuth 194.8

Mean Lineation Plunge 1.6

Great Circle Azimuth 190.0

Great Circle Plunge 18.6

1st Eigenvalue 0.601

2nd Eigenvalue 0.376

3rd Eigenvalue 0.023

LN (E1 / E2) 0.469

LN (E2 / E3) 2.793

(LN(E1/E2)] / (LN(E2/E3)) .. 0.168

Spherical variance 0.4244

Rbar 0.5756

4.3.2 GEOrient Plots

The same data are plotted using GEOrient 9.4.3. GEOrient is a 32-bit Windows-specific program. This program plots geological structural orientation diagrams (equal area, or equal angle stereographic projections, and rose diagrams, with orientation data either from the clipboard from other applications, or from ASCII text files in a wide range of file formats, and using a wide range of orientation conventions. Data can be presented either as coloured symbols representing point densities, as great circles, as contours of gridded point density, or as rose diagrams of polar or non-polar data. Rose diagrams can include appended length or weighting factors in the data to plot Length-Azimuth or Weighted Frequency-Azimuth roses. Data can also be presented as Classified Plots or Numeric Stereographic projections where the data are colour-coded or contoured according to the values of Appended data in each orientation line.

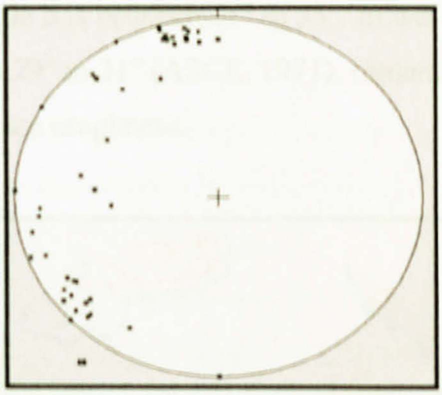


Figure 4.5 GEOrient pole plots

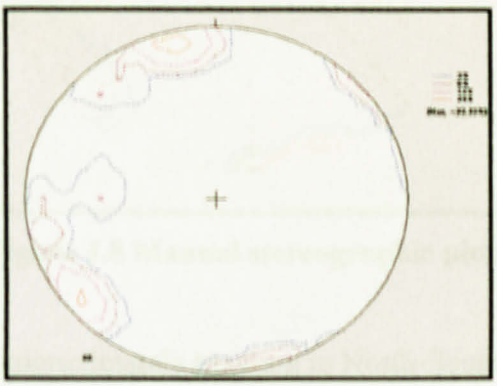


Figure 4.6 GEOrient density plots

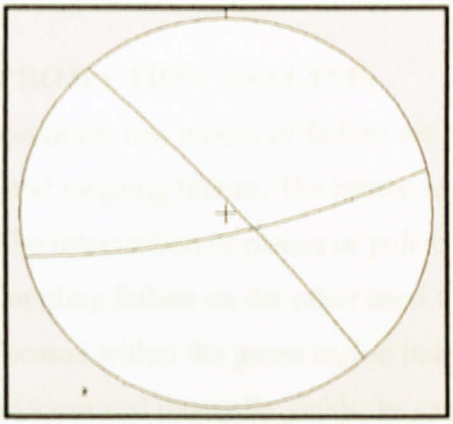


Figure 4.7 GEOrient plot of intersection of planes of pole concentration

Apart from GEOrient plots, the data has also been plotted manually. The pole plots, contour plots, and intersection of planes of pole concentrations plots. All three types of plots are drawn into one single tracing paper. A friction angle ø, of 30° is adopted for this plot. For fine-grained granite in dry conditions, the range of friction angle ø is between 31° to 35°. In wet conditions, the friction angle ø ranges from 29° to 31° (ASCE, 1971). Generally, the friction angle ø depends on the surface roughness.

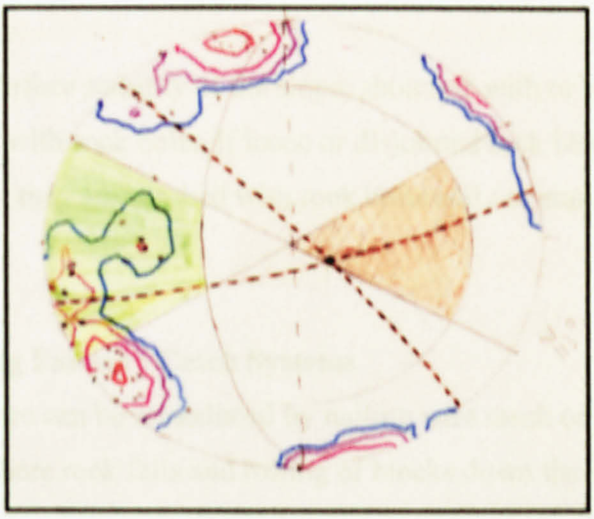


Figure 4.8 Manual stereographic plots

The slope face is approximately trending in North-South direction, as indicated by the grey dotted line.

4.4 STEREOGRAPHIC PROJECTION ANALYSIS

From the stereographic projections, two modes of failure are identified. The failures are identified as wedge failure and toppling failure. The brown region is the range of section for wedge failure. The intersection of planes of pole concentrations falls within the wedge failure region. Toppling failure on the other hand falls within the green region. Pole plots that are located within the green region indicate toppling failure. Plotting with STEREO, GEOrient and manually yields the same outcome in terms of stereographic projections.

The slope face is approximately 90° and the direction of fall of the wedge failure is 113°.

4.5 STABILIZATION METHODS

4.5.1 Wedge Failure : Rock Bolting and Rock Anchors

Wedge failure can be stabilized by providing rock bolts or rock anchors distributed over the slope and orienting them as advantageously as possible with respect to the location and attitude (strike and dip) of the joints.

To improve surface stability of the slope, shotcrete with or without wire mesh is often adapted with rock bolts. If loose or disjointed rock blocks are present on the slope face, they are stitched with rock bolts and / or supported by padding work.

4.5.2 Toppling Failure : Catch Systems

Toppling failure can be remediated by nailing wire mesh or geo-grid into the steep slope where rock falls and rolling of blocks down the slope can be prevented.

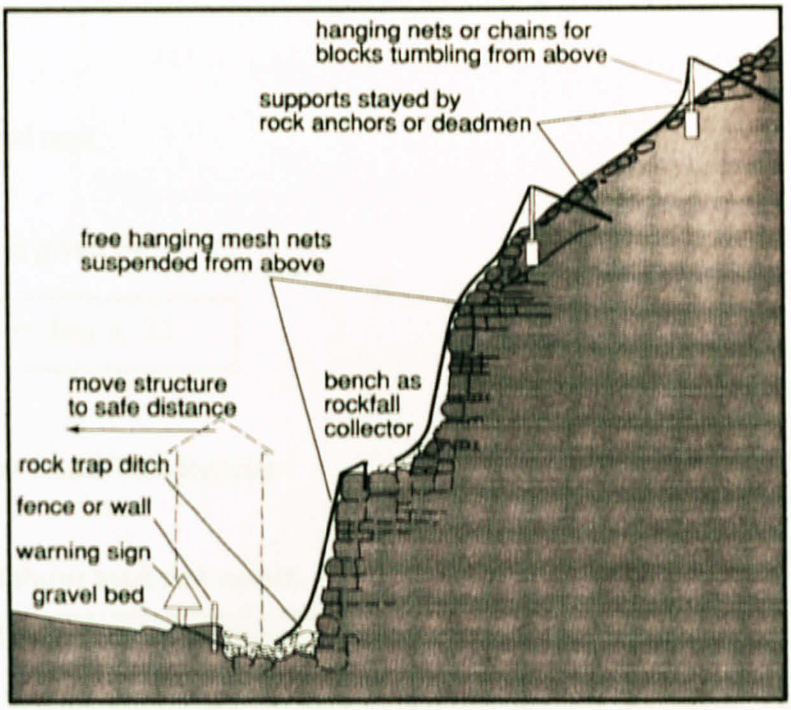


Figure 4.9 Rockfall control measures (From Geotechnical Control Office, Hong Kong, 1984)

4.6 POINT LOAD TEST : TEST ON IRREGULAR SHAPES

Ramamurthy (2007) mentioned that when regular cores are not possible to be obtained, the irregular pieces from excavation can also be used in estimating the compressive strength by testing a roughly chiseled spherical mass with diameter ranging from 30mm to 50mm. The point load tester is used to conduct this test. The specimen is tested in between two hardened conical tips with 5mm curvature and 60° conical angle, in a rigid frame.

All point load test results are to be referred to as 50mm size of specimen and the point load strength index (Is_{50}) can be calculated as:

$$\boxed{Is_{50} = 142.6\, P \;/\; A^{0.775}}$$

Equation 4.0

Where:

P = Failure load

A = Cross-sectional area

The compressive is given by:

$$\boxed{Co = Is_{50} \times 22}$$

Equation 5.0

4.6.1 Point Load Test Results

Table 4.0 Point load test results

Rock no.	Length (mm)	Width (mm)	Force (kN)	Force (lbf)	Is(50)	Co (MPa)
1	121.13	65.29	30	6500	7.535	165.766
2	90.03	53.64	8	1800	6.743	148.352
3	81.53	43.19	43.19	3750	6.281	138.175
4	110.57	73.62	8	1800	7.584	166.846
5	113.04	36.85	14	3200	6.523	143.499
6	107.78	62.51	29	6500	7.268	159.894
7	115.67	68.79	17	3800	7.545	165.993
8	175.06	76.28	15.5	3488	8.477	186.501
9	108.83	61.64	23.8	5300	7.261	159.739
10	176.55	53.99	2.5	500	7.858	172.877

From the point load test conducted on 10 samples, an average compressive strength of 160.1 MPa has been obtained. This suggests that the outcrop consisted of relatively fresh igneous rock. Little signs of weathering can be traced from the samples. Fresh rocks also exhibit mineral grains with original composition and shape.

Table 4.1 Ranges of properties of some intact rocks (Ramamurthy, 2007)

Rock type	*Dry density,* kN/m^3	*Compressive strength,* MPa	*Tensile strength,* MPa	*Young's modulus,* MPa × 10^3	*Poisson's ratio*
Basalt	28–29.5	100–350	10–30	20–80	0.19
Diabase	—	140–240	—	70–100	0.25
Diorite	27–30.5	150–300	12–30	4–10	—
Dolerite	—	227–319	12–26	60–90	0.15–0.29
Dolomite	25–28.7	30–500	15–25	25–80	0.29
Gneiss	28–30	50–250	5–20	24–80	0.1–0.40
Granite	26–29	100–340	7–25	2–75	0.1–0.39
Limestone	22–26	30–250	5–25	3–82	0.08–0.39
Marble	26–27	50–250	7–20	3–44	—
Phyllite*	26.9–27.9	79–102	15–19	7.5–14.5	0.33
Quartzite	26–27	150–320	10–30	16–94	0.11–0.25
Sandstone	20–26	20–300	4–25	0.6–68	0.1–0.40
Schist	26.3–28.8	29–190	9–29	6–57	0.1–0.25
Shale	20–24	5–100	2–10	2.5–44	0.1–0.19
Siltstone	24.4–26	25–50	3–6	26–62	0.27
Slate	26–27	100–200	2–5	0.6	—
Coal	—	10–39	—	2.4–5.3	0.33–0.4211

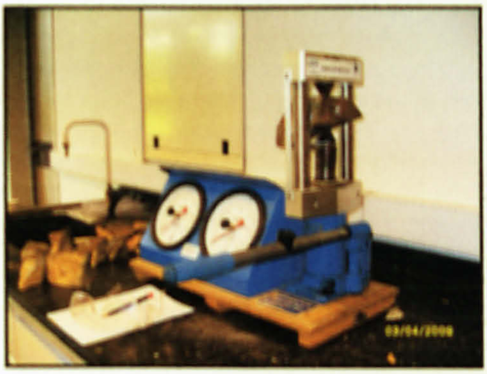


Figure 4.10 Point Load Tester

Figure 4.11 Conducting the point load test

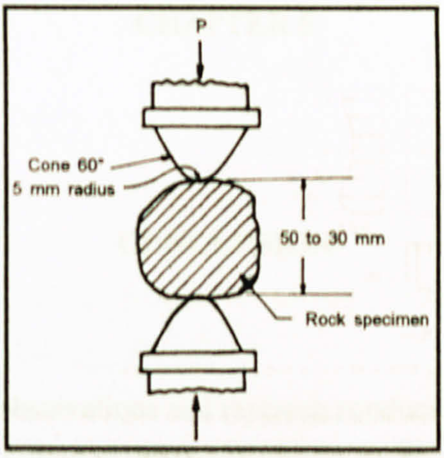


Figure 4.12 Point load test arrangement

CHAPTER 5

CONCLUSION

As a conclusion, from the observations and research conducted, it is relevant to execute a rock slope assessment at kilometer 8 along Ipoh-Lumut Expressway, Perak. Based on the earlier observations, the slope has already shown minor signs that will lead to failure. The evidence can be seen from the pieces of rock at the toe along the roadside drain. Stereographic projection plots of discontinuities recorded from the field can be utilized in assessing the stability of the slope.

From the analysis, two modes of failure have been identified by manually plotting the data in stereographic projection. The modes of failure are wedge failure and toppling failure. The slope face is approximately 90° and the direction of fall of the wedge failure is 113°. For wedge failure, stabilization methods can incorporate the use reinforcements such as rock bolts and rock anchors. Toppling failure on the other hand, can be managed by catch systems such as nailing wire mesh or geo-grid to the rock slope to catch rock falls and avoid them from bouncing and rolling onto the road.

For this report, more literature review has been added regarding rock mass classification systems and Q-systems and also a case study of similar occurrence of rock slope failure in Bukit Lanjan. Information concerning stereonets and plotting techniques are also included. The latest updates added are information on the stereonet plotting and orientation analysis using the computer programs STEREO and GEOrient.

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Appendix A :

Collected Data as of 23rd February 2009

(2)

GENERAL INFORMATION

Seq. No. | Site | Date: Day 20, Month 02, Year 09 | Operator | Discontinuity data sheet No. 01 of

NATURE AND ORIENTATION OF DISCONTINUITIES

Chainage or No. (cm)	Type	Dip	Dip direction	Persistence (m)	Aperture	Infilling	Consistency	Roughness	Discontinuity weathered zone width mm	Waviness wavelength	Waviness amplitude	Water	Remarks
0206	1	80	044	0023	4	1		4				1	
0306	1	84	048	0026	5	1		3				1	
0458	1	80	053	0010	5	1		4				1	
0636	1	81	056	0008	5	1		2				1	
0639	2	90	000		5	1		2				1	
0639	1	40	041	0052	4	1		2				1	
0422	1	80	164	0080	7	1		2				1	
0830	1	76	059	0012	7	1		2				1	
0806	1	90	092	0010	7	1		2				1	
0497	1	78	170	0014	5	1		2				1	
0385	1	78	170	0053	3	1		2				1	
0386	1	58	100	0028	5	1		2				1	

Type | Dip, Dip direction | Persistence | Aperture | Nature of infilling | Consistency of infilling | Roughness | Waviness | Water

Data Sheet for Discontinuity Survey

②

GENERAL INFORMATION

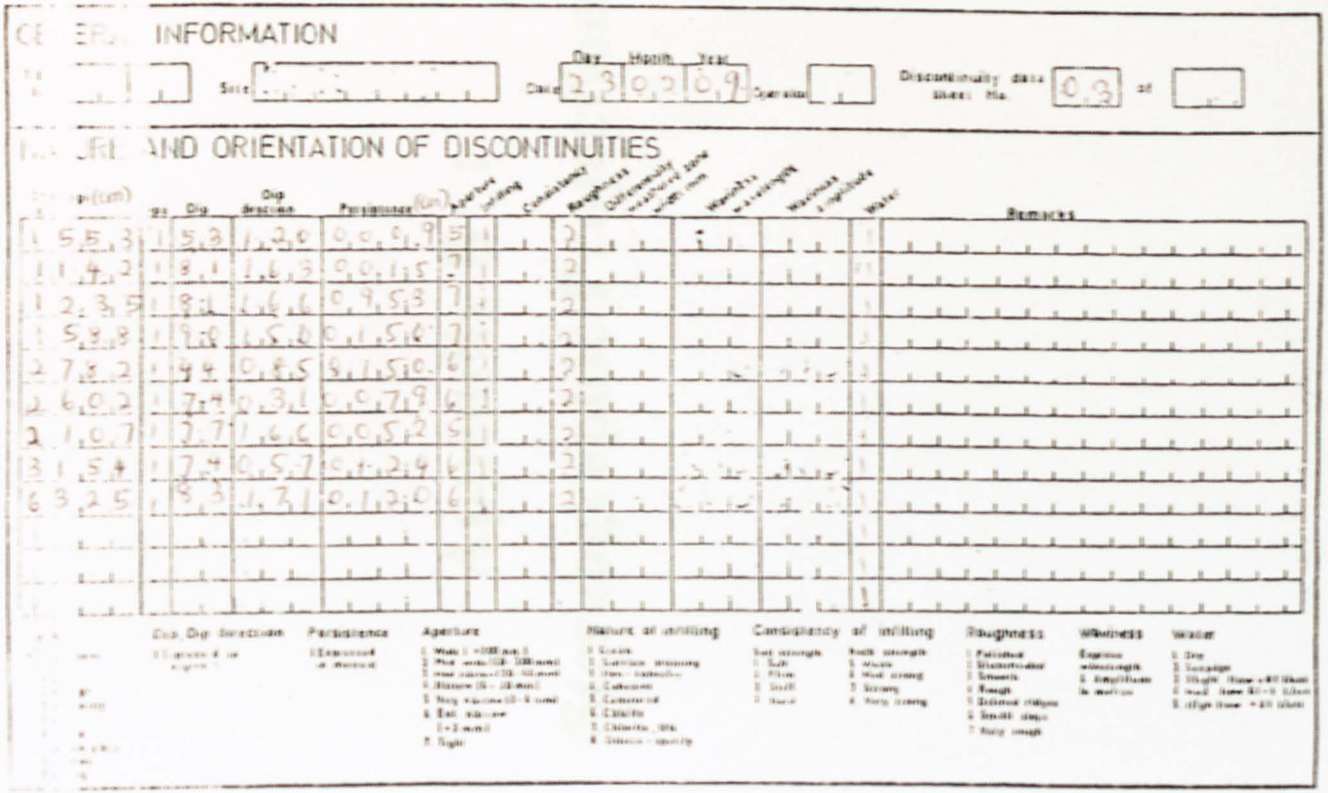
Seq. No.	Site	Date (Day / Month / Year)	Operator	Discontinuity data sheet No.	of
		2 0 / 0 2 / 0 9		0 2	

NATURE AND ORIENTATION OF DISCONTINUITIES

Chainage in m (cm)	Type	Dip	Dip direction	Persistence (m)	Aperture	Infilling	Consistency	Roughness	Discontinuity weathered zone width mm	Waviness wavelength	Waviness amplitude	Water	Remarks
0827	1	89	163	0043	5	1		2				1	
1044	1	85	053	0015	5	1		2					
0835	1	89	163	0030	5	1		2				1	
1162	1	81	052	0019	5	1		2				1	
0823	1	89	163	0033	5	1		2				1	
0729	1	80	170	0100	4	1		2				1	
1293	1	77	048	0012	4	1		2				1	
0893	1	87	163	0034	6	1		2				1	
1374	1	73	056	0028	4	1		2				1	
0967	1	83	164	0190	5	1		2				1	
1575	1	80	069	0020	5	1		2				1	
1110	1	85	170	0115	6	1		2				1	

Type — Dip, Dip direction — Persistence — Aperture — Nature of infilling — Consistency of infilling — Roughness — Waviness — Water

Data Sheet for Discontinuity Survey



GENERAL INFORMATION

Date: 23 / 02 / 09 (Day / Month / Year)

Discontinuity data sheet No. 03 of

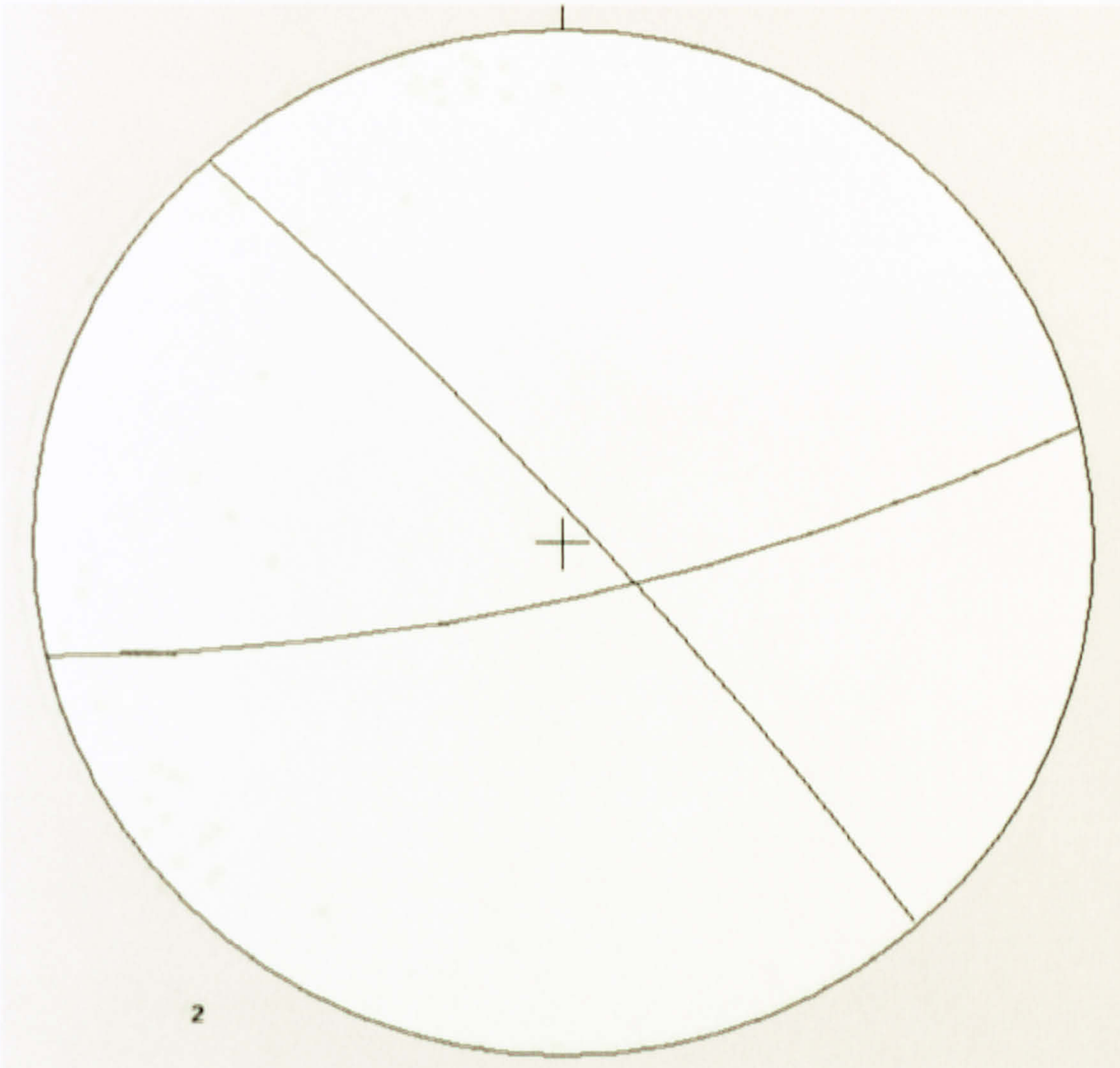
NATURE AND ORIENTATION OF DISCONTINUITIES

Distance (cm)	Type	Dip	Dip direction	Persistence (cm)	Aperture	Roughness
1553	1	53	120	0009	5	2
1142	1	81	163	0015	7	2
1235	1	81	166	0953	7	2
1588	1	90	150	0156	7	2
2782	1	44	085	0150	6	2
2602	1	74	031	0079	6	2
2107	1	77	166	0052	5	2
3154	1	74	057	0124	6	2
6325	1	83	171	0120	6	2

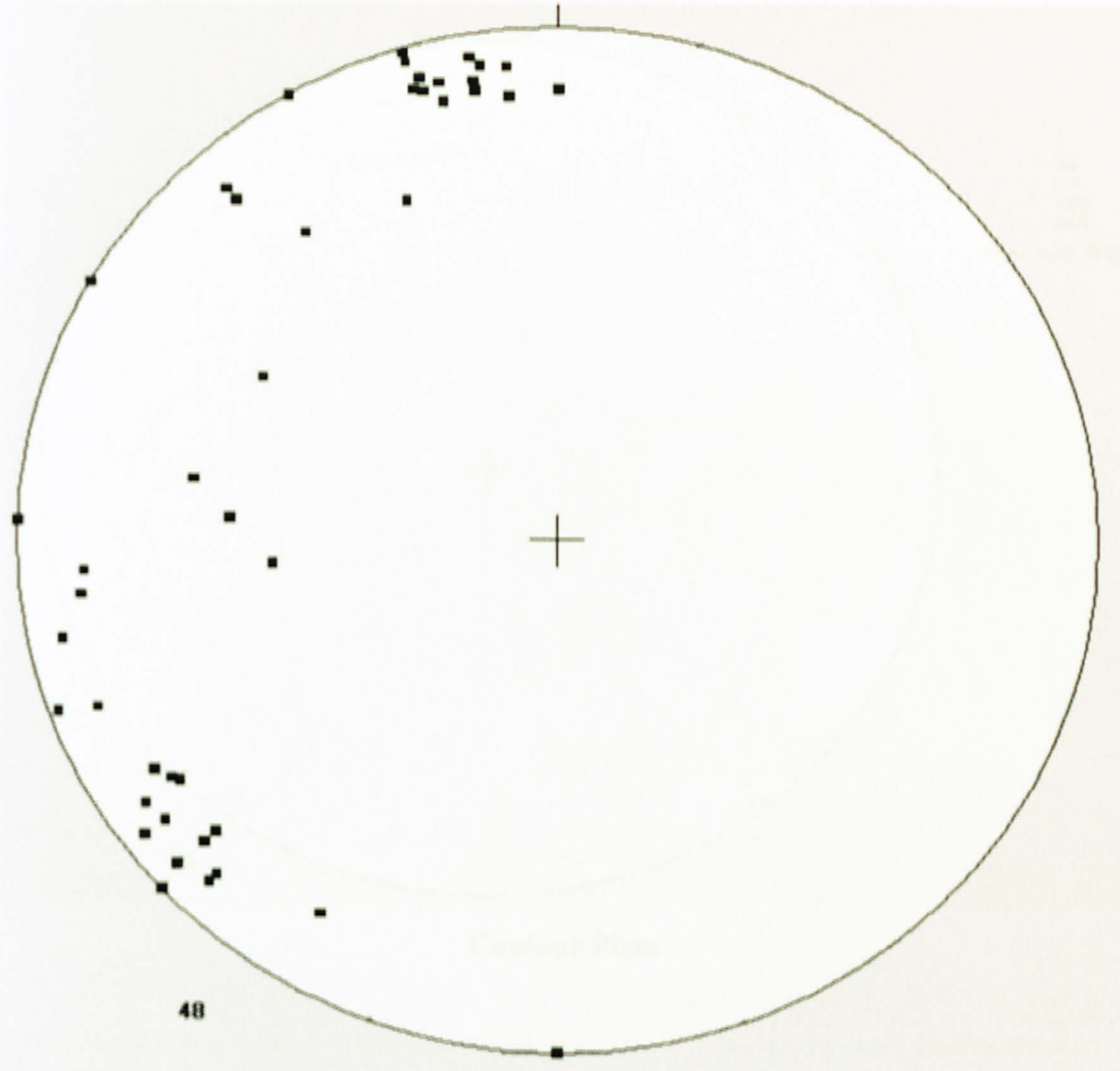
Data Sheet for Discontinuity Survey

Appendix B :

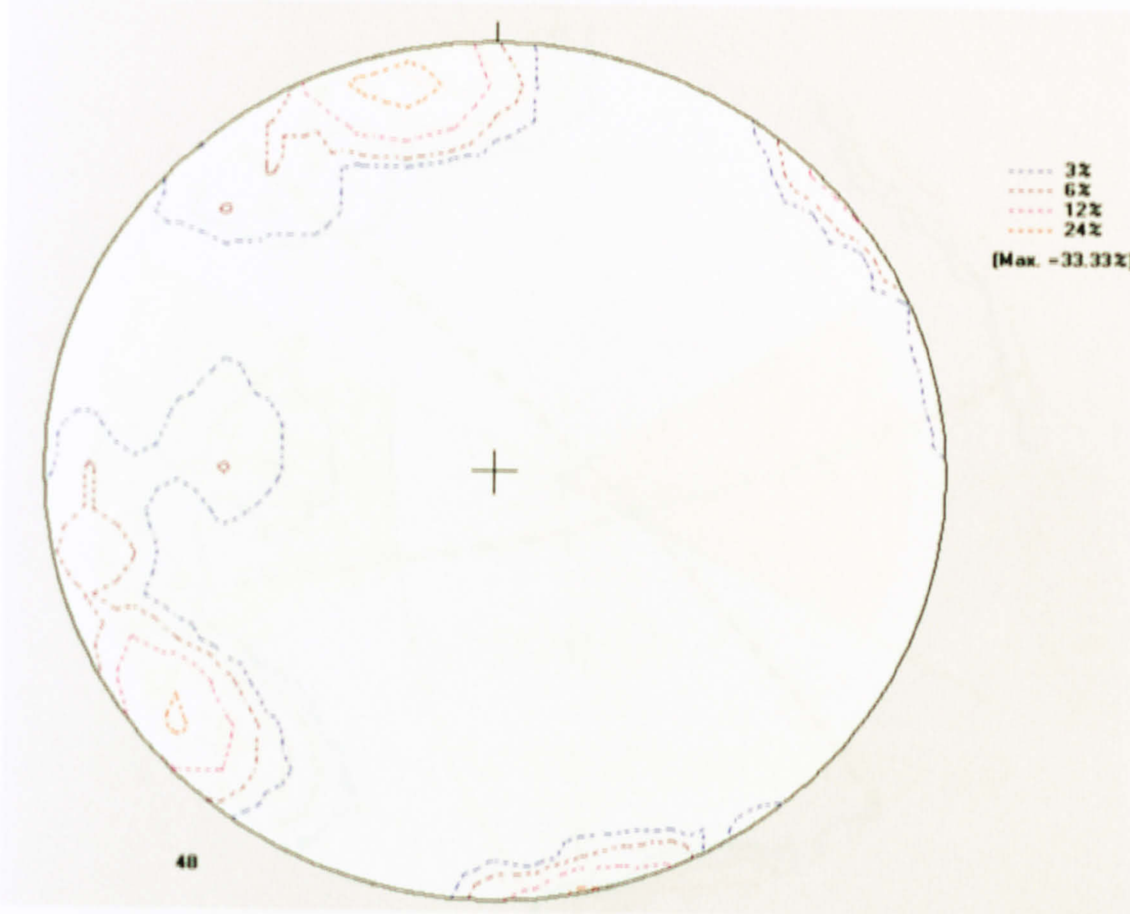
Stereographic Projection



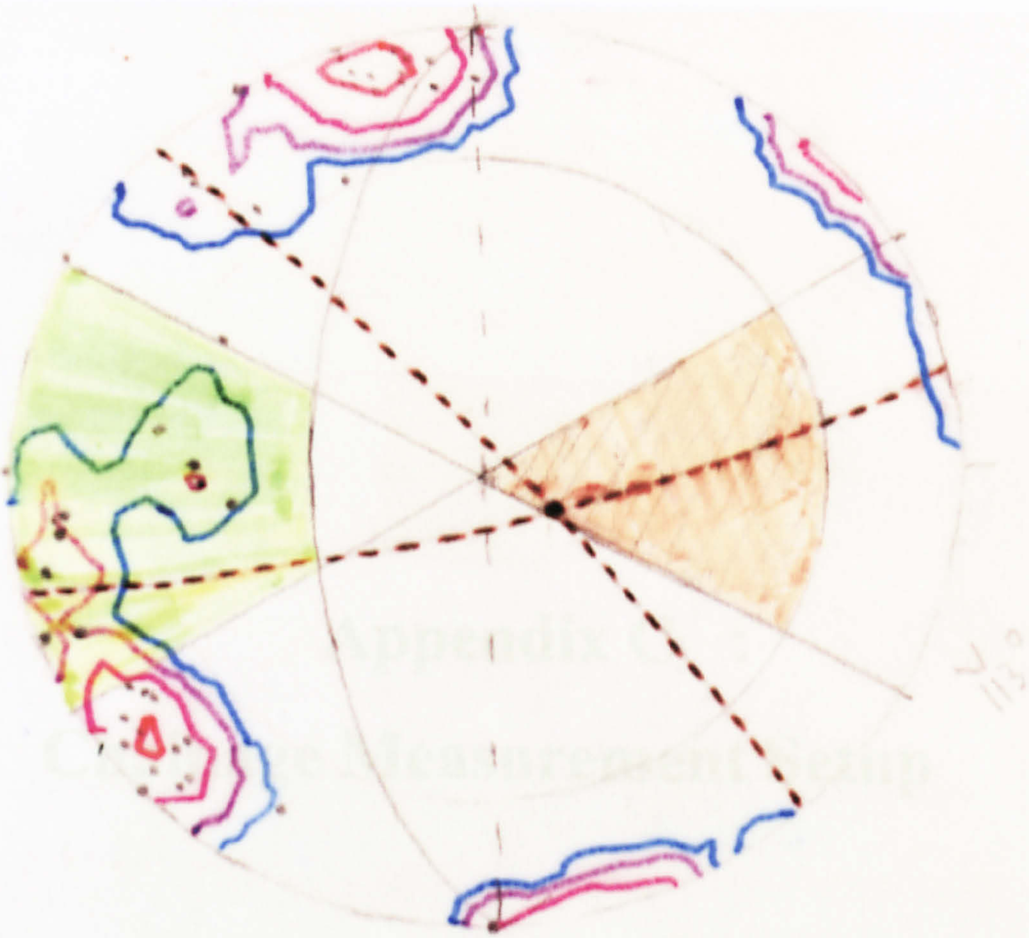
Intersection of two planes



Plot of poles



Contour Plots



Manual plots of plane intersection, friction angle, poles, contours, and slope face

Appendix C :
Chainage Measurement Setup

Appendix D :
Rockfalls and Seepage

Rockfall

Rockfall

Seepage

Seepage