

# DESIGN PARAMETERS AND MANAGEMENT STRATEGIES OF SOIL AND ROCK SLOPES

*(NCHANGA OPEN PIT – KONKOLA COPPER MINES PLC)*

By

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the award of Master of Mineral Sciences Degree*



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**Declaration**

*I, Sraj Umar Banda, do hereby declare with academic honesty that with the exception of quotes and work of other people, which I have duly referenced to and acknowledged herein, this thesis, is the result of my own original research work. No part of it has been presented in pursuit of another degree in this university or anywhere else.*

*I declare that this thesis was written according to the rules and regulations governing the award of Master of Mineral Sciences of the University of Zambia*


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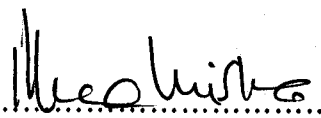

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*Certificate of Approval*

*This thesis, by Sraj Umar Banda, is approved as fulfilling the requirements for the award of Degree of Master of Mineral Sciences by the University of Zambia*

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*Co-Supervisor:*  ..... *09/06/2008*

## **Dedication**

*To my family and friends.*

## *Acknowledgements*

*I would like to thank the University of Zambia, and particularly the Department of Mining Engineering, for awarding me a Staff Development Fellowship and a study opportunity at this institution. My thanks also go to the members of staff of the three departments of the school of mines for giving me a helping and guiding hand throughout my study period.*

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*My field work would not have been completed successfully without the help of the Nchanga open Pit's staff from various sections. God bless you all.*

*To everyone who in one way or another made a positive contribution to this thesis, I can only say thank you very much.*

## ***Abstract***

*Rock slope stability is a very important research area for many geo-engineers concerned with civil or mining works. Slope stability of pits, roads, tailings dams and other embankments is very cardinal as it directly impacts on the working or utilisation of these facilities. The Nchanga Open Pit has presented special challenges.*

*This research is directed towards efforts to determine slope design parameters of the Chingola Open Pit F and D (COP F-D) Phase III, which is an important satellite pit of the Nchanga Open Pits. After recent accidents in this mine, attention has been drawn to the need to understand failure mechanisms of the slopes on this site and the need to determine acceptable risks of mining at the designed pit slopes. In addition, further analysis on the management of these slopes is required.*

*COP FD phase II has been operated since 2003 with the slope design parameters projected from phase I (which started in 2001) and as such a need arose to determine specific design parameters for phase III because of variations in geological conditions between Phase I and Phase III.*

*The methodology in this research included reconnaissance surveys, literature review, field work which comprised core logging, mapping, slope monitoring (water table, cracks and slope movements), laboratory testing, data analysis using specialised software and report preparation. In addition, a risk assessment (economic and safety for workers and equipment) was carried out based on Phase II data. Existing slope management strategies were reviewed.*

*Three geotechnical zones based on rock characteristics were established and design parameters (such as cohesion and angles of internal friction) were determined for the zones. Economic risks mainly arose from the costs of repairing ramps and upper benches damaged due to undercutting. Safety risks resulted from the fact that initially the dump (OB 5), was located about a kilometre away from the pit rim but as mining progressed this dump became part of the pit wall on the footwall.*

*Because the upper part of the pit wall was composed of dump material which was unstable, there was a high likelihood of slope failure.*

*Design slope angles have been recommended based on geotechnical zones and a comprehensive rock characterisation. Slope management strategies have been reinforced by recommending better coordination, frequency and effectiveness of monitoring systems. Corrective measures for different rock slope instabilities have also been recommended.*

*Further research work, through numerical modelling, is recommended to give better insight on slope failure mechanisms of the Nchanga Open Pit mining area.*

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## Abbreviations and Acronyms

1.	ARK	Arkose
2.	BAS	Basement Complex
3.	BSSL	Banded Sandstone Lower
4.	BSSU	Banded Sandstone Upper
5.	CDOL	Chingola Dolomite
6.	COP F/D	Chingola Open Pit F and D
7.	DOLSCH	Dolomitic Schists
8.	FOS	Factor of Safety
9.	GSI	Geologic Strength Index
10.	ISRM	International Society for Rock Mechanics
11.	KCM	Konkola Copper Mines PLC
12.	LAT	Laterite
13.	LBS	Lower Banded Shale
14.	LL	Liquid Limit
15.	MRMR	Mining Rock Mass Rating
16.	NOP	Nchanga Open Pit
17.	PDF	Probability Density Factors
18.	PI	Plasticity Index
19.	PL	Plastic Limit
20.	PQ/SM	Pink Quartzite and Shale Marker
21.	RMR	Rock Mass Rating
22.	RQD	Rock Quality Designation
23.	SSR	Slope Stability Radar
24.	SWEDGE	Surface WEDGE Analysis software
25.	SWG	Shale With Grit
26.	TFQ	The Feldspathic Quartzite
27.	TFQT	Transitional Feldspathic Quartzite
28.	TR	Transitional Arkose
29.	U	Pore Water Pressure (Uplift)
30.	UBS	Upper Banded Shale
31.	UNWEDGE	Underground WEDGE Analysis software
32.	V	Pore Water Pressure (Lateral)

# CHAPTER ONE

## INTRODUCTION

### 1.1 Background Information

*Earthworks* is a field that has over the years evolved to utilising high profile technologies. The construction and extractive industries have taken advantage of these technological advancements to enhance quality output and increased productivity, and safety. Zambia as a country has been in the fore front of copper production and as such has also benefited from the technological advancements seen in the extractive industries. Copper, being the prime driver of the Zambian economy, has been produced since pre-independence times, using various mining methods. During the 1970s Zambia was vying with Chile for the position of third largest copper producer after the U.S.A. and U.S.S.R. but in 1971 Chile undisputedly established herself in this position, (DeVletter et al, 1972). The primary methods of mining established in Zambia are variations of underground mining and those of open cast mining. Open cast mining has been practiced in Zambia since the beginning of mining activities in early twentieth century.

Open pit mining is one of the high productivity mining methods in use today in most mining projects. In recent years annual output had increased generally in the open pits. This is in comparison to low annual outputs a decade ago. This is due to improvements in ore production capacities and handling, new equipment facilitation and the general understanding of the technical part of mining such as planning, technical services and geo-engineering.

One cardinal aspect of open pit mining is that of slope stability. The economics of open pit mining is determined by the depth of the pit and slope of the pit walls.

The Nchanga Open Pit has been the pioneer project of open pit mining in Zambia and it is at this mine that some of the new advancements in open cast mining have been developed through research. Emphasis in this report was on slope stability and pit design in open pit mining, which forms the basis of the research conducted in this programme.

This study was undertaken at the Konkola Copper Mines (KCM) within the realms of a Memorandum of Understanding (MOU) signed between the University of Zambia and KCM. It had been discovered previously by geotechnical consultants that there were deficiencies in the way some the geotechnical investigations in obtaining design parameters were done and that these deficiencies led to a generally poor understanding of the rock mass strength, its deformability and the slope failure mechanisms.

## **1.2 Description of the Research Site**

The Chingola Open Pit F and D (COP F/D) is located about five kilometres to the south of the Nchanga Open Pit (NOP), which is the main and first open pit that was operated at this mine. The general location of the Nchanga Open Pits (which is the name given to all the pits including the satellite pits such as COP FD) is in the Chingola town of the Copperbelt which lies between longitudes  $27^{\circ}45'$  and  $28^{\circ}00'$  East and latitudes  $12^{\circ}30'$  and  $12^{\circ}40'$  South. Chingola town itself is found about 40 km from the border (Kasumbalesa border post) between the Republic of Zambia and the Democratic Republic of Congo. COP FD is situated on a relatively flat terrain which is underlain by sedimentary and meta-sedimentary rock formations. Copper production at COP FD Phase II is mainly at the benches below 120 m from the ground surface. Because design parameters in phase II were based on those in phase I, a need arose to determine specific design parameters for phase three. This was so because of the possibility of changes in the trends of rock and soil properties that governed slope stability in Phase I as compared to Phase II. Therefore, to base design parameters for Phase III on those of Phase I would not have been advisable as it was located far away from Phase I upon which Phase II design parameters were based. Figure 1.1 shows the location of the COP F/D.

Figure 1.1 shows the location of the Nchanga mining area (figure 1.1 (a)). It also shows F and D as different pits (figure 1.1 (b)). However this is one pit mined in phases as shown in Figure 1.1 (c).

# The Nchanga Open Pit Site Map

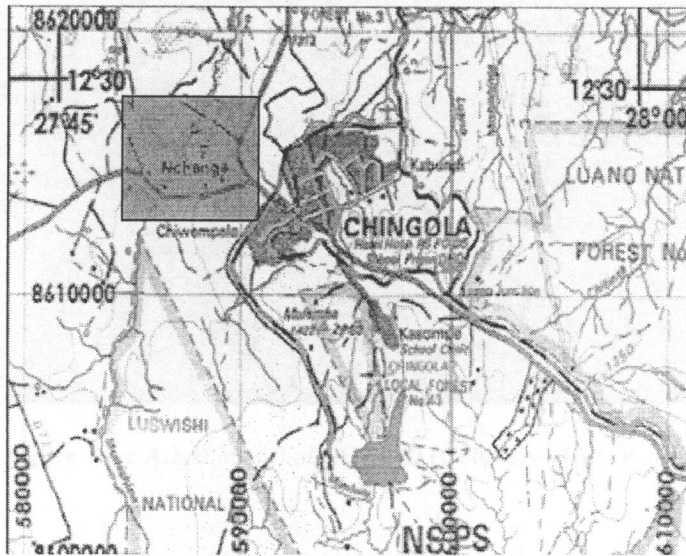


Figure 1.1 a The general Nchanga-Chingola aerial map

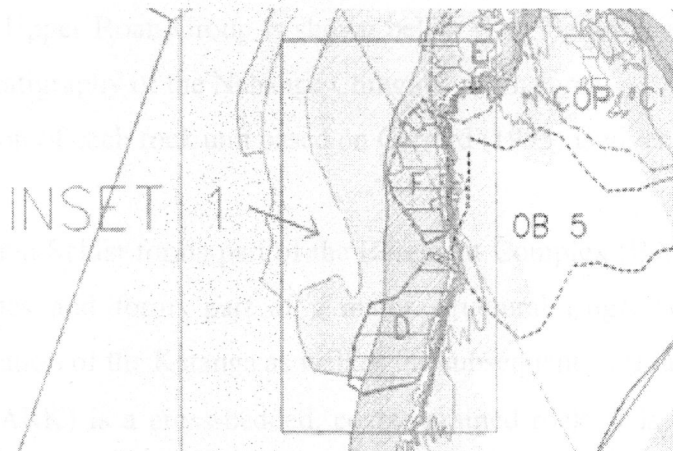


Figure 1.1 b Location of COP F/D

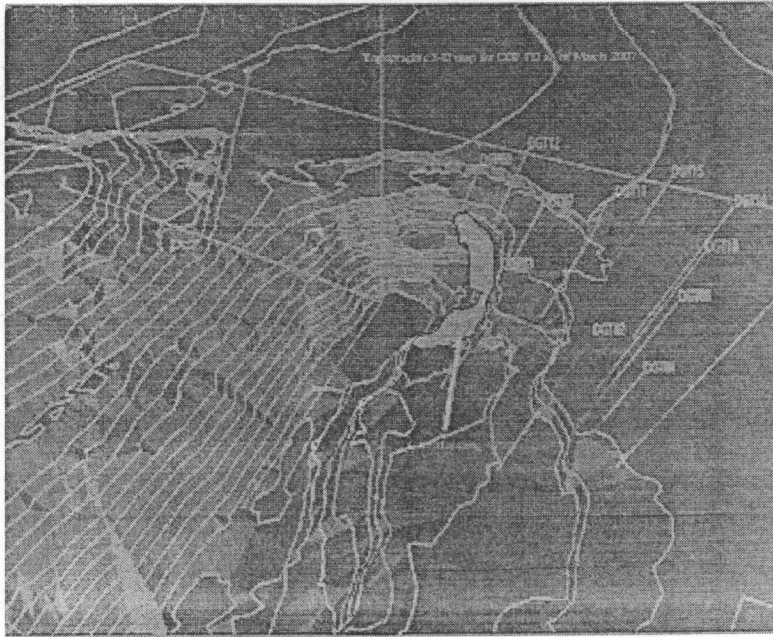


Figure 1.1 c A 3-D Model of COP F/D (Courtesy of KCM plc)

### 1.2.1 Geologic Setting of the Nchanga Chingola Mining Area

The stratigraphy range of rock units present in the Chingola area from the Basement Complex to the Upper Roan Group is shown below (De Vletter et al, 1972). Figure 1.2 shows the stratigraphy of the Nchanga Chingola mining area.

A brief description of each rock unit based on Garrard (1995) is given below:

- The Lufubu Schist forms part of the Basement Complex (BAS). It is intruded by granites and forms part of a major structural ridge, which influenced sedimentation of the Katanga as well as the subsequent Lufilian orogeny.
- Arkose (ARK) is a cross-bedded, coarse grained rock. It is schistose in part and composed principally of quartz, feldspar and sericite.
- Transition Arkose (TR), is a friable rock, up to 10 m thick and composed of quartz and sericite. It is a porous and brecciated rock. It is believed to have originated in a thrust plane, which was involved in severe shearing and utilized as a pathway for fluids. It forms the minor ore body aquifer in the COP F/D area.
- Lower Banded Shale (LBS), is a thinly bedded, friable, ill-sorted rock. It contains abundant sericite and detrital quartz. The formation marks the start of the lower hangingwall aquifer just above the ore body.

- Shale Marker (SM) is a distinctive horizon, 0.6 – 1.5 m thick and consists of valve-like, red-brown-mica-and white-quartz-laminae.
- Pink Quartzite (PQ), is a creamy or faintly pink Arkose about 4.5 – 6 m thick. Because it is flanked by less competent beds, it is prone to jointing, boundinage and attenuation.
- Banded Sandstone is presented in two types, the Lower and Upper formations. The Banded Sandstone Upper (BSSU) contains more soft and coarse micaceous material in its lower part and grades into fairly solid feldspathic quartzite in its upper part. This unit forms the top limit of the lower hangingwall aquifer.
- The Feldspathic Quartzite (TFQ): This is a relatively competent formation consisting of well-sorted, medium-sized grains of quartz, microcline and orthoclase in a matrix of the same minerals. In its upper part, it is sometimes interbedded with dolomitic schist.
- Dolomitic Schist (DOLSCH): It is interbedded with mica dolomite, schist and argillites.
- Chingola Dolomite (CDOL): Is quartzose and talcose. It marks the start of the upper hangingwall aquifer.
- Shale with Grit (SWG) and Upper Roan Dolomite (URD), together with the Chingola Dolomite constitute the upper hangingwall aquifer. The Upper Roan Dolomite forms the top limit of this aquifer.

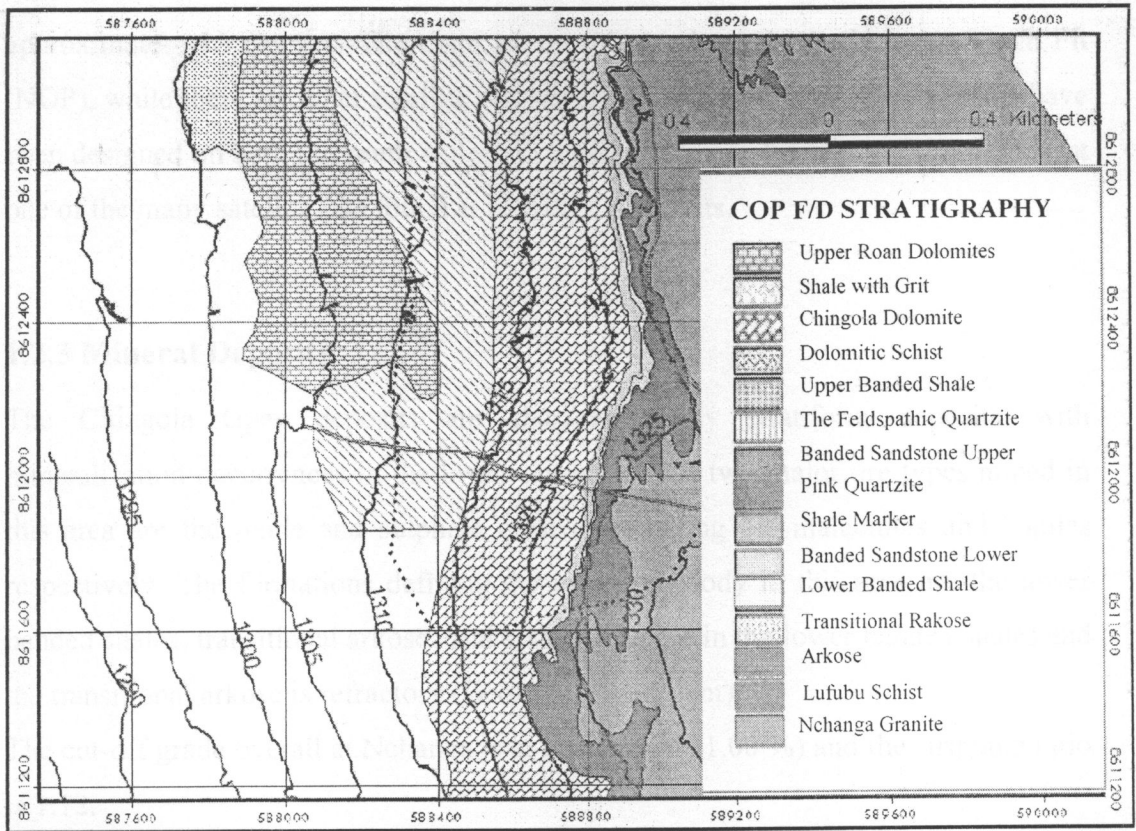


Figure 1.2 The geology of COP F/D, Nchanga. 2006 (Courtesy of KCM plc)

Figure 1.2 shows the geologic formation of the Nchanga Chingola mine Area. Generally the rock formations exhibit a high degree of weathering due to the high water table that characterizes the Chingola Open Pit F/D. In a study by Nkhuwa et al (2004), called *The Geophysical Investigation for the Dewatering of the Nchanga Mine COP F/D Area* it was reported that the Cop F/D had two major sets and a minor set of discontinuities<sup>1</sup> that were water bearing. These discontinuities provide planes of failure in the event of sliding.

### 1.2.2 The Nchanga Syncline

The Nchanga ore body is actually called the Nchanga Syncline, and it strikes in almost north-south direction. This synclorium has its fold axis dipping at

<sup>1</sup> Discontinuities are defined (Teeuw, 1995) as elongate, often vertical, fracture zones that range from a few millimetres to kilometres in width and from a few centimetres to hundreds of kilometres in length. Discontinuities that may range from 300 m to many kilometres in length are called **lineaments**; those that are shorter than 300 m are called **fracture traces**.

approximately 15 degrees. The main pit on this ore body is the Nchanga Open Pit (NOP), while there are other satellite pits namely COP (A, B, C, D, E and F) that have been designed on both the south and north limbs of the synclinalorium. COP FD is just one of the many satellite pits found at Nchanga Open Pits.

### **1.2.3 Mineral Deposits**

The Chingola Open deposits are characteristically stratiform deposits, with mineralization occurrences in the lower ore body. The two major ore types mined in this area are the oxide and sulphide ores; representing the malachites and bonites respectively. The formations defining the lower ore body in this area are the lower banded shales, transitional arkoses and arkose. The ore in the lower banded shales and the transitional arkose is refractory (Geology Department).

The cut-off grade overall at Nchanga is at one percent (1.00 %) and the stripping ratio is 1:12.

### **1.2.4 Climate of Chingola**

Chingola experiences a sub-tropical climate, which is strongly seasonal and it conforms to the general climatic conditions prevalent in Zambia. These general climatic conditions consist of:

1. A hot dry season lasting from mid August to mid November. During the day temperatures can vary between 26° and 38° C.
2. A warm wet season from mid November to mid April during which time 95% of the annual rainfall of about 1, 194 mm occurs. 25% of this rain falls in January.
3. A cool dry season from mid April to mid August. Mean temperatures vary between 14° and 23°. Minimum temperatures can reach below 10° in June and July.

It has been observed that climate affects the stability of slopes by way of soaking the tension cracks and weakening the cohesive forces that hold the soils together.

### **1.3 Research Questions**

The research questions, therefore, are: how high and how steep can a slope be allowed to go? Also, what should be the safety factor to keep the slope stable in all situations? The answer to this question can only be arrived at by considering factors that define the economic pits and safety slopes in an open pit mine. Of importance to know is that when a slope is flattened it means a high stripping ratio and therefore very costly while a steeper slope will affect the stability of the slope, compromising safety of workmen and equipment. A balance should therefore be struck when it comes to the ultimate pit limits between a high stripping ratio and a steeper slope.

This report highlights research undertaken at the Chingola Open Pit F/D. which, hereinafter, is called the research site. It will also highlight certain methodologies that have been used to come up with design parameters for the slopes in this area.

### **1.4 Problem Statement**

Most of the challenges of open pit mining are associated with safety of slopes. It is very cardinal to understand the material in which the soil and rock slopes are being excavated. Strictly speaking, most of the materials excavated are nonhomogeneous and as such require a highly dynamic approach to determine safe slopes.

Slope failures can be very disruptive. In April 2001, a slope failure at the Nchanga Open Pit south wall caused fatalities and extensive damage to mining equipment. Because of its magnitude, in terms of losses, this slope failure was an eye opener for the mine management and the industry at large.

In 2005, a landslide at Kafue Gorge caused damage to power supply installations and plunged the entire country into darkness. The importance of a thorough geotechnical analysis of constructed soil and rock structures therefore cannot be over-emphasised. Research is required at a high level to come up with lasting solutions to geotechnical problems.

In the past few years, after the 2001 slope failure, a team comprising of geotechnical consultants undertook a study to review geotechnical procedures at the Nchanga Open Pits related to slope stability. This team identified deficiencies in some procedures

undertaken by the Nchanga Open Pit in the areas of slope analysis and stability. These deficiencies had led to a generally poor understanding of the rock mass strength, its deformability and slope failure mechanisms (Lipalile, 2005).

## **1.5 Justification**

This study is important to the Mining industry and road and dam construction. Much of the fieldwork and analysis for this research was carried out using novel approaches of slope designing. It is hoped that Konkola Copper Mines and other open pit mining companies in Zambia and the world at large will benefit from this research by either replicating it in its current format or adapting it to local situations.

The Nchanga Open Pit area has relatively complex geologic structures that have not been fully understood. This research was instituted to bridge the gap that was found on information gathering through core logging, field mapping and laboratory testing for rock strength parameters.

The information obtained from the research would be useful for formulation of safety rules and practices by the Mine Safety Department of the Ministry of Mines and Minerals Development; Ministry of Works and Supply; and other organizations involved in built environment. It would also help in devising management systems of slopes; and lastly, in the development and revision of curricula at Mining Engineering Faculties e.g. at the University of Zambia.

This study also seeks to improve the geotechnical database of the Nchanga Open Pits, which is to be utilized by various sections involved in the open pit operations.

## **1.6 Aim and Objectives**

The main aim of this research therefore, are to collect information on the soil and rock mass strengths of COP F/D, to form the basis for the various soil and rock investigations designed to determine the slope design strength parameters and to devise a management strategy that will be suited to the Nchanga Open Pits, that could be incorporated into the daily routine work of the geotechnical section.

The objectives of the research were to:

1. Review and analyze various existing geotechnical methodologies currently in use at Nchanga Open Pit for slope stability analysis.
2. Determine the slope design parameters for the Chingola Open Pit F/D for Phase III.
3. Devise a system of slope management.
4. Determine various risks associated with mining.

## **1.7 Methodology**

This research was primarily composed of three activity phases:

1. Review of existing geotechnical and geological information.
2. Field investigation and data gathering.
3. Data analysis (desk work).

Information on geotechnical parameters and geologic formations was readily available, only requiring further study to fully understand the failure mechanisms of the Nchanga Mining Area rock and soil formations.

Visitations were made in various sections like the geotechnical engineering, geology and planning sections. Information on design parameters for the earlier phases in mining were reviewed.

### **1.7.1 Site Investigations (visits and monitoring), COP F/D**

Site investigations comprised of core logging, mapping, insitu stress analysis inspection of joint systems and estimation of rock strengths using the geologic hammer both when logging and during cell mapping.

### **1.7.2 Cell Mapping.**

Done with help of a photographic camera to describe inaccessible areas and physically inspected the joint systems and determined if there was any joint infilling.

Estimations of UCS were done by utilising the Hoek\_Brown failure criterion system of rock strength charts and by giving specific rock units blows with a geologic hammer and observing the behaviour of the rock after the blow to determine the strength range.

### **1.7.3 Geotechnical Core Logging**

Physical inspections of the rock cores were done at the time of drilling up to the core storage shade. Logging was also carried out on cores that had been drilled a long time before the onset of this study. This gave the researcher an opportunity to compare the particular parameters of core from various places but belonging to the same area formations. As in cell mapping the geologic hammer was used to estimate the uniaxial compressive strength of the rock formations.

### **1.7.4 Field visits and slope inspection rounds.**

Field visits were done throughout the research period. It consisted of inspection of slopes in the mined areas to gain further insight into the rock mass behaviour once slopes were cut. Three areas of interest were identified.

As the research period was stretched over different seasons, care was taken to determine the effects of seasonal changes on the slopes. For instance, during the rainy season, observations were made on water seepage rates in various positions. Areas where tension cracks were suspected were found to be very susceptible to failures in the rainy season much more than they were in dry seasons.

## **CHAPTER TWO**

### **LITERATURE REVIEW**

#### **2.1 The History of the Nchanga Mine Area**

Understanding soil and rock units starts with the understanding of soils and rock characteristics given the necessary information regarding the formation, occurrence and use of the site in which the soil or rock materials are found. The classification of these two geologic materials/masses will usually go together because it is difficult to deal with one without reference of the other. At Nchanga Open pits, for instance, the geologic formation shows a high interaction between soils and rock units such that special slope analysis considerations are undertaken due to the potential of slope instability.

The Nchanga Mining licence Area, has, as pointed out in the previous chapter, sedimentary to meta-sedimentary rock units that make up its lithological contacts. These rock units are characterised by high degree of weathering and leaching by ground water.

#### **2.2 Kinematic Analysis for Planar Failure**

The analysis of slopes cannot be done in a pre determined manner as slopes are mostly cut in either homogeneous or non homogeneous soil and rock masses. They can also either be artificial or natural. It is therefore, important to understand the materials that make up the slope. In this regard, this section will review the physical mechanical parameters that govern stability in slopes.

##### **2.2.1 Shear Strength Parameters.**

Shear strength parameters are a very important component of slope analysis. The shear strength of a soil is required for numerous analyses such as, the prediction of the stability of slopes; the design of foundations; and earth retaining structures (Fredlund et al, 1996).

The factors governing large scale slope stability are primarily: (1) the stress conditions in the pit slopes, including the effects of groundwater; (2) the geological structures, in particular the presence of large scale fractures; (3) the pit geometry and (4) the rock mass strength (Sjoberg, 1996).

The strength parameters responsible for the stability of slopes are the *cohesion* and the *internal friction angle*.<sup>2</sup> However a complete strength test of the rock formation is cardinal to determine the effective pressures at which different loadings will affect the strength of the slopes. When dealing with rock masses some assumptions must be made to simplify the process of determining rock properties. "*The most frequently used analytical methods in rock mechanics assume that the modulus of elasticity (E) is constant and of linear and elastic characteristics.*"(Massmin, 2004).

The rock therefore is to be assumed to be linearly elastic for most of the geomechanics and geotechnical procedures to hold. This is true for all rock and soil masses under investigation. The Nchanga Mining Area has mostly soft rocks that overlay the granitic intrusions of the Nchanga granites whose contacts with the upper sedimentary strata gives rise to the vast meta-sediments that characterise the large part of the mineralization in this area.

If two specimens of the same rock materials, separated by a discontinuity such as a joint which is cemented together, the displacement  $u$  of the specimen across the discontinuity will require the application of shear stress  $\tau$ , applied to the specimens as illustrated in Figure 2.1.

Plotting the shear stress level at various shear displacements, for one of the tests carried out at a constant normal stress level results in the type of curve shown in figure 2.2 (Hoek and Bray, 1977).

### **2.2.2 Limit Equilibrium Analysis of Slopes**

Limiting equilibrium methods of slope analysis have gained so much popularity during the past few decades. This is because these methods have simplified an otherwise complex mathematical problem by applying a few assumptions that make rock easy to deal with, within these assumptions.

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<sup>2</sup> Cohesion and internal friction angle are material properties that depend on the particle make up of the rock or soil mass. The infinitesimal forces that hold the material particles together add up to make up the cohesive forces of that material.

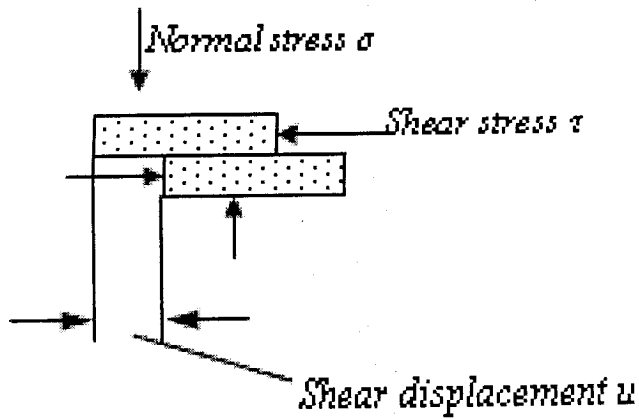


Figure 2. 1 Illustration of shear displacement

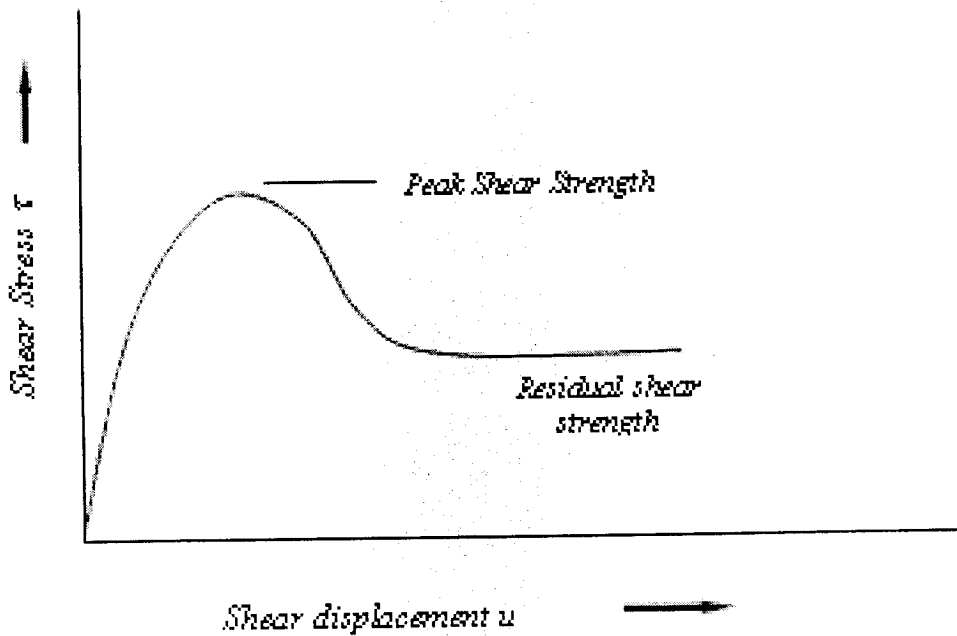


Figure 2. 2 Shear stress-strain displacement curve

If the peak shear stress values obtained from tests carried out at different normal stress levels are plotted, a curve such as the one shown in Figure 2. results. At very small displacements, the specimen behaves elastically and the shear stress increases linearly with displacement. This curve is linear within the experimental accuracy of the results, with a slope equal to the peak friction angle  $\phi_p$  and an intercept on the shear stress axis of  $c_p$ , the cohesive strength of the cementing material. This cohesive

component of the total shear strength is independent of the normal stress but the frictional component increases with increasing normal stress (Hoek and Bray, 1977).

The peak shear strength is defined by the equation:

$$\tau = c_p + \sigma \tan \phi_p \quad (1)$$

In general terms, for all shear strength solutions taken from the shear stress-shear displacement curve, equation 1 would take the following form:

$$\tau = c + \sigma \tan \phi \quad (2)$$

where  $\tau$  is the shear strength,  $c$  is the cohesion of the material  $\sigma$  is the normal stress and  $\phi$  is the internal friction angle specific to the material in question.

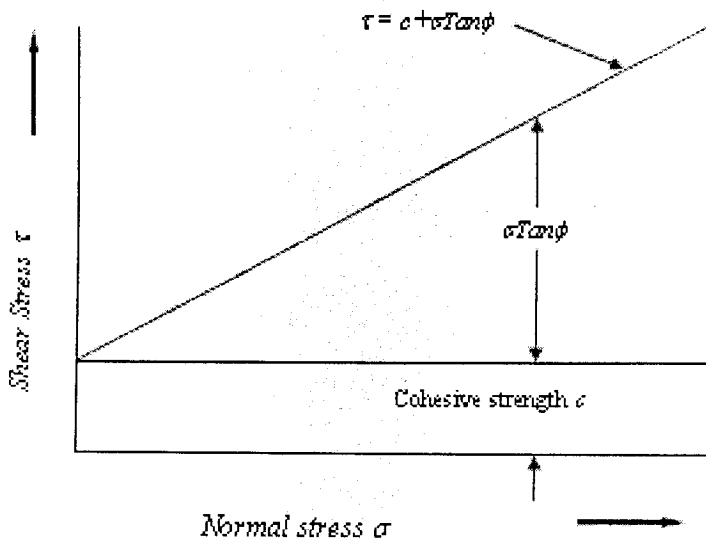


Figure 2. 3 The linear relationship between shear and normal stresses

### 2.2.3 Sliding due to Gravitational Loading

A block of weight  $W$  resting on a plane which is inclined at an angle  $\psi$  will be acted upon by gravity only.

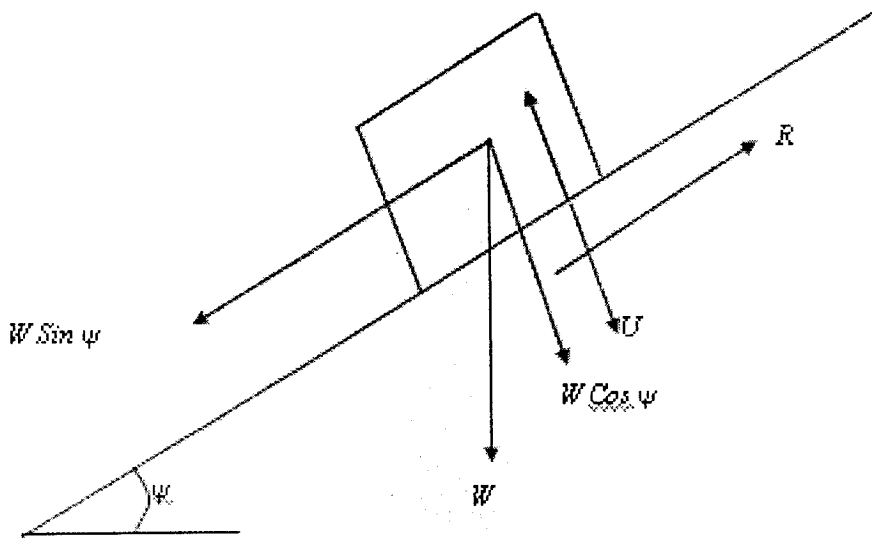


Figure 2. 4 Illustration of a sliding block with various forces acting on it

The normal stress  $\sigma$  which acts across the potential sliding surface is given by

$$\sigma = (W \cos \psi) / A \quad (3)$$

Where A is the base area of the block.

If the shear strength of this surface is definable by equation (1) above we have

$$\tau = c + \frac{W \cos \psi}{A} \cdot \tan \phi$$

$$\text{Or} \quad R = cA + W \cos \psi \cdot \tan \phi \quad (4)$$

Where  $R = \tau A$  is the shear force which resists sliding down the plane.

The block will be in limiting equilibrium condition at the point of sliding where the disturbing forces acting down the plane will balance out the resisting forces:

$$W \sin \psi = cA + W \cos \psi \cdot \tan \phi \quad (5)$$

When the soil or rock material where the rock block is to slide has no cohesion, i.e.  $c = 0$ , the limiting equilibrium condition in (4) reduces to:

$$\psi = \phi \quad (6)$$

## 2.3 Effect of Water Pressure and Effective Stress on slope stability

The effect of water pressure on the sliding blocks is such the normal stress acting across the sliding surface is reduced to the effective stress as represented by equation 7. If the uplift pressure from the ground water is  $U$ , then equation (5) becomes

$$\tau = c + (\sigma - u)\tan\phi \quad (7)$$

### 2.3.1 The Effect of Water Pressure in Tension Crack.

If the tension crack splits the block and is filled with water, the pressure in the tension crack increases linearly with depth of the crack. This exerts a total force  $V$  on the face of the block due to the water pressure. While the force  $U$  reduces the normal force acting across the failure surface, the force  $V$  will reduce the resisting forces of the block.

$$W \sin\psi + V = cA + (W \cos\psi - U) \tan\phi \quad (8)$$

Many practical problems involve assessing the shear strength of unsaturated soils. Fredlund and Morgenstein (1977) showed that the shear strength of unsaturated soils can be described by any two of three stress state variables, namely,  $(\sigma - U_w)$ ,  $(\sigma - U_a)$  and

$(U_a - U_w)$ ; where  $U_a$  is the pore-air pressure.

Fredlund et al, (1977) proposed the following equation for the shear strength of unsaturated soils:

$$\tau = c' + (\sigma_n - U_a)\tan\phi' + (U_a - U_w)\tan\phi^b \quad (9)$$

Where  $\phi^b =$  Angle indicating the rate of increase in shear strength relative to a

Change in metric suction,  $(U_a - U_w)$ , when using  $(\sigma_n - U_a)$  and  $(U_a - U_w)$  as the two state variables; and

$\phi =$  Angle indicating the rate of increase in shear strength with respect

to net normal stress,  $(\sigma_n - U_w)$  when using  $(\sigma_n - U_w)$  and  $(U_a - U_w)$  as the two state variables.

Fredlund's equation 9 compares very well with equation 7 from the kinematical analysis of slopes by Hoek et al (1977).

### 2.3.2 Prevention of sliding of a rock block

One of the effective ways of stabilizing blocks or slabs of rock which are likely to slide down is by the use of tensioned rock bolts or cable bolts. If a block is resting on an inclined surface and acted upon by the uplift force  $U$  and the force  $V$  due to water pressure in the tension crack. A rock bolt to prevent sliding is tensioned to the load  $T$  installed at an angle of  $\beta$  to the plane as shown in figure 2.4:

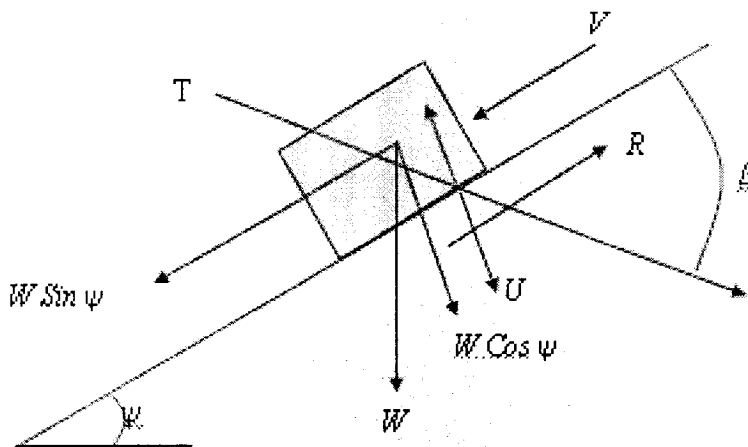


Figure 2. 5 Prevention of block slides with rock bolts

The resolved component of the bolt tension  $T$  acting parallel to the plane is  $T \cos \beta$ , while the component acting across the surface upon which the block rests is  $T \sin \beta$ . The Limiting Equilibrium condition here is as follows:

$$W \sin \psi + V - T \cos \beta = cA + (W \cos \psi - U + T \sin \beta) \tan \phi \quad (10)$$

### 2.4 Factor of Safety (FOS) of a slope

The factor of safety is a ratio of the total force available to resist sliding to the total force tending to induce sliding. This factor is used to compare the stability of slopes under conditions other than those existing at the limiting equilibrium (Hoek and Bray, 1977). This factor is given by:

$$F = \frac{cA + (W \cos \psi - U + T \sin \beta) \tan \phi}{W \sin \psi + V - T \cos \beta} \quad (11)$$

Stability analyses have usually depended on factor of safety for predicting the stability of slopes. Factor of safety values greater than 1 means the slope is stable, while values less than 1 mean the slope is unstable. A factor of safety of 1 indicates a critical slope on the verge of failure (Hoek and Bray, 1977).

For the Nchanga Mine Licence Area, the factor of safety is usually between 1.2 and 1.5. In some cases it is as high as 1.9 and 2.0. (*Field Data, 2006*).

Using the factor of safety in slope stability analysis provides an initial indication of the stability status of any given slope.

During the construction of the Mica and Revelstoke dams on the Columbia River in British Columbia, Canada, several potential slides were investigated. Two of these, the Downie Slide, a 1.4 billion cubic metre ancient rock slide, and Dutchman's Ridge, a 115 million cubic metre potential rock slide, were given special attention because of the serious consequences which could have resulted from failure of these slides (Lewis and Moore, 1989). After a successful drainage programme as remedial measure to these slides, the factor of safety, which was initially calculated at 1.00 before drainage took place, gave the value of 1.06 for the drained slope. The 6% improvement in safety factor was very significant to give safety to a larger extent in the slides.

A cohesion less soil (equation 6) will have the factor of safety given by:

$$F = \frac{\tan \phi}{\tan \beta} \quad (12)$$

The Engineering design of the US Army Corps:

Factors of safety for slopes should be selected consistent with the uncertainty involved in the parameters such as shear strength and pore water pressures that affect the calculated value of factor of safety and the consequences of failure. When the uncertainty and the consequences of failure are both small, it is acceptable to use small factors of safety, of the order of 1.3 or even smaller in some circumstances. When the uncertainties or the consequences of failure increase, larger factors of safety are necessary.

Large uncertainties coupled with large consequences of failure represent an unacceptable condition (Engineers Manual No. 1110-2-1902, 2003).

The factor of safety will be as effective as the accuracy in the data collection and data interpretation. An insight into the failure mechanisms of the site being investigated plays a pivotal role into determining the right parameters.

## **2.5 Probability of slope failure**

The probability of failure is a measure of the likelihood that a slope failure will take place. It takes into consideration all the empirical data that is available from field and laboratory investigations. This is an approach used for decision making during slope stability analysis.

The basis for the probabilistic design methods is the recognition that the factors which govern slope stability all exhibit some natural variation and ideally this variation should be accounted for in the design method.

In a probabilistic design method, the stochastic nature of the input parameters is included and the resulting chance, or probability, of failure is calculated. Dealing with probability of failures rather than safety factors means that one acknowledges that there is always a finite chance of failure, although it can be very small. This is more realistic than stating that a slope with a certain factor of safety is perfectly stable, notwithstanding other environmental factors that will affect this slope. Also, a quantitative description of the probability of failure can be used in a risk analysis and linked to economical decision criteria (Sjoberg, 2004).

If the load and the strength of a slope can be described by two probability density factors (PDF), the strength, or resistance of a slope is termed  $R$  and the load is denoted  $S$ . The respective mean and standard deviations of each distribution is denoted  $m_r$  and  $s_r$  for the resistance, and  $m_s$  and  $s_s$  for the load. It can be seen in Figure 2.6 that the two curves overlap, meaning that there exist values of the resistance which are lower than the load, thus implying the probability of failure.

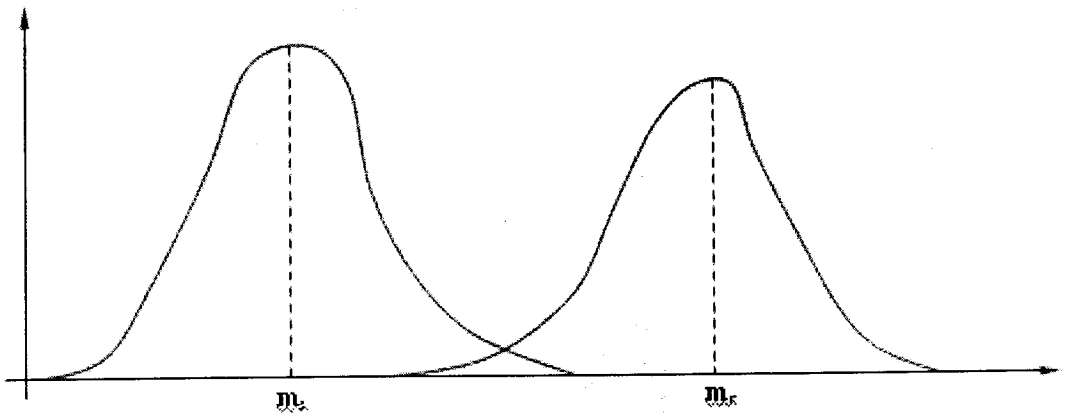


Figure 2. 6 Hypothetical distribution of the strength, resistance  $R$ , and the load  $S$  for a slope (after Sjoberg, 2004)

To be able to calculate the probability that the load exceeds the strength of the slope it is common to define a safety margin,  $SM$ , as:

$$SM = R - S \quad (13)$$

The safety margin is one type of performance function which is used to determine the probability of failure. The performance factor is often denoted  $G(X)$ , hence.

$$G(X) = R(X) - S(X) \quad (14)$$

Where  $X$  is the collection of all random input parameters (e.g. cohesion, friction angles, UCS, etc), which make up the resistance and the load distributions respectively (Sjoberg, 2004).

An alternative formulation of the performance function which is often used in geomechanics involves the use of a factor of safety,  $F_s$ . failure occurs when  $F_s$  is less than unity, and hence the performance function is defined as:

$$G(X) = F_s - 1 \quad (15)$$

Kirsten, (1982, quoted in Stacy, 2001) presents Table 2.1, which gives probabilities of failures for which open pit mine slopes should be designed.

**Table 2. 1 Comparative significance of probabilities of failure**

probability of failure %	Design criteria on basis of which probability of failure is established		
	Serviceable life	Minimum surveillance required	Frequency of evident slope failure
50 – 100	Effectively zero	Serves no purpose (excessive probability tantamount to failure)	Slope failure generally evident
20 – 50	Very short term (temporary open-pit mines – untenable risk of failure in temporary civil works)	Continuous intensive monitoring with sophisticated instruments	Significant number of unstable slopes
10 – 20	Very short term (quasi temporary slopes in open-pit mines – undesirable risk of failure in temporary civil works)	Continuous monitoring with sophisticated instruments	Some unstable slopes evident
5 – 10	Short term (semi temporary slopes in open-pit mines, quarries or civil works)	Continuous monitoring with simple/rudimentary instruments	Occasional unstable slope evident
0 – 5	Medium term (semi permanent slopes)	Conscious superficial monitoring	No ready evidence of unstable slopes

*After Stacy, 2001: pp 7*

## 2.6 Data input for Rock Slope Analysis

Rock slope stability analysis begins with information gathering for the eventual input into the geotechnical model for rock slope analysis. This information is gathered through site investigation which can include:

- Field observations (field mapping and geotechnical examinations)
- Study of geological maps and memoirs to indicate probable soil conditions
- Aerial photographs
- Geotechnical core logging
- In-situ and laboratory testing

As Evert Hoek (1994, cited in Palmstrom, 1997: 21 – 25) puts it:

I see almost no research efforts being devoted to the generation of the basic input data which we need for our faster and better models and our improved design techniques. These tools are rapidly reaching the point of being severely data limited.

The importance of data collection for any geotechnical analysis can therefore not be over-emphasised. Construction materials used in civil and mining engineering fields are mostly characterised by their strength, stress – strain behaviour under various conditions and these are the properties which are used for engineering design. In rock engineering rock characterisation ensures the rock properties in question are reviewed.

## **2.6.1 Field Investigation**

This is the genesis of almost all geotechnical works. Rock units occur naturally and cannot be treated in a unified manner. All rock units occur naturally differently with different mineral compositions. This gives different soil and rock masses unique properties that require careful characterisation.

### **2.6.1.1 Core Logging**

Drill core logging forms the primary source of geotechnical information from the on set of any mining project. It is through core logging that soil and rock properties are analysed for the first time well before any insight can be perceived in the design of the surface or underground mine.

Rock information obtainable from core logging ranges from geologic structures to rock quality designation, rock engineering properties to chemical analysis of the rock

core. Core logging also results into the information that is used to design the rock characterisation and preliminary design parameters.

Core logging can be done on oriented core, which will give orientations for major discontinuities, areas of weaknesses and weathering.

### **2.6.1.2 Geotechnical Mapping**

Field mapping provides information in-situ. This information is usually obtained to link the core logs to the right locations on site. In addition, this works as a re affirmation to the design information used for the preliminary construction of geotechnical models.

Geologic mapping has been a traditional method of field investigation by geotechnical and mining engineers for a long time. However new advancements in technological equipment have revolutionised the art of mapping to a simple operation of tools.

Currently the laser imaging system is used to determine geologic structures in a slope face. Feng, *et al.* 2004, recommends the laser technology mapping as one of the fastest ways of data capture as it combines the observation and descriptions of geological features of joints and rock masses as well as measure some geometrical parameters in these joints.

The limitations of these methods of high resolution laser beams is that without physically observing the discontinuities in the field, engineering judgment, which is the best compliment to slope stability analysis, is usually clouded. An advantage the laser imaging mapping equipment has is the ability to generate mapping data in highly inaccessible areas of the open pit.

### **2.6.1.3 Laboratory Testing**

Laboratory testing concretises the field investigation and core-shed data to information that can better be used for rock characterisation and geotechnical zoning. To make the best out of laboratory testing results, sampling should be done under the recommended methods. Structured sampling may be a good starting point but usually random sampling is advised. The recommended number of rock core samples per hole is the average of 10 cores.

## **2.7 Software for rock slope design and analysis.**

Advancement in computing power in recent years has helped to advance the ease with which slope analysis can be done through limiting equilibrium and numerical methods. Many iterative procedures in engineering design for built environment and mining were done manually. These procedures can now be modelled in computer programs there by reducing time for design and decision making for optimum productivity. For instance, modelling programs such as *Datamine* and *Surpac* are used for graphical modelling and presentation of digital terrain models. These models are useful as a design tool for mining and civil engineering procedures. *Rocscience Inc.* has provided interactive software programs for slope design and slope stability analysis such as *SWEDGE* and *UNWEDGE* for probabilistic and deterministic wedge failure analysis for surface and underground mining respectively. Other modelling programs are *SLIDE* for 2-D stability analysis of slopes, *DIPS*, etc. these programs make the work which otherwise may be difficult to accomplish a lot easier as some of the relationships that may be very important to review during a design project are incorporated in the modelling software.

## **2.8 Mechanics of Slope Failure**

### **2.8.1 Limit Equilibrium Analysis**

The aim of limit equilibrium studies is to analyse the stability of any mass of soil or rock assuming incipient failure along a potential slip surface. This approach often enables the solution of many problems by simple statics, provided some simplifying assumptions are made. In general a failure surface of simple shape is assumed and the material above this surface is considered to be a 'free body'. The disturbing and resisting forces above the assumed failure surface are estimated enabling the formulation of equations concerning force equilibrium or moment equilibrium (or both) of the potential sliding mass. The quantitative information obtained from the solutions of the equations give a picture of the stability of a slope; and is relevant only to the assumed slip surface. An iterative approach to these methods is therefore necessary to find the critical (potential) slip surface.

## 2.8.2 A Slip Surface

Limit equilibrium methods are generally not concerned with stress distribution at every point above or below the assumed slip surface nor do they seek to satisfy the equations of stress equilibrium at every point within the potential failure mass. The concept of a discontinuity in the form of a slip surface is a fundamental one for these methods and such a surface may be considered as a hypothetical one separating two rigid bodies.

It is important to analyse these slopes with a failure type in mind.

## 2.8.3 Plane Failure

Plane surfaces often occur when a soil deposit or embankment has a specific plane of weakness. However, a plane failure is a comparatively rare sight in rock slopes because it is only occasionally that all the geometrical conditions required to produce such a failure occur in an actual slope (Fig 2.7).

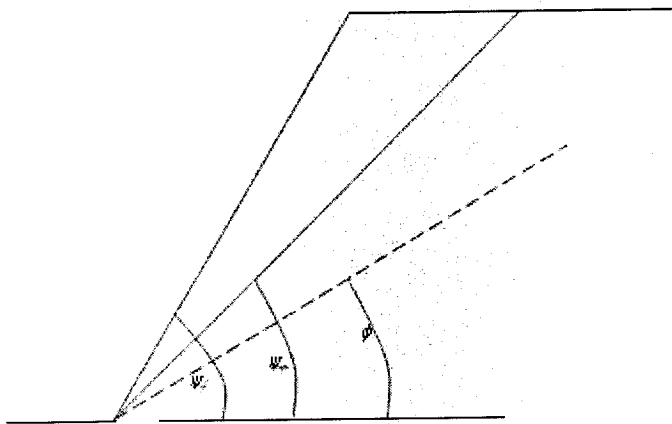


Figure 2. 7 Slope face showing conditions for sliding to occur, Hoek et al 1977

### 2.8.3.1 General conditions for plane failure

- I. The plane on which sliding occurs must strike parallel or nearly parallel (within approximately  $\pm 20^\circ$ ) to the slope face.
- II. The failure plane must “daylight” in the slope face. This means that its dip must be smaller than the dip of the slope face, i.e.  $\psi_f > \psi_p$ .

- III. The dip of the failure plane must be greater than the angle of friction of this plane, i.e.  $\psi_p > \phi$ .
- IV. 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.

### 2.8.3.2 Plane Failure Analysis

Two cases here must be considered:

- a. A slope having a tension crack in its upper surface
- b. A slope with a tension crack in its face.

The transition from one case to another occurs when the tension crack coincides with the slope crest, i.e. when

$$z/H = (1 - \cot \psi_f \cdot \tan \psi_p) \quad (16)$$

#### Assumptions

- a. Both sliding surface and tension crack strike parallel to the slope surface.
- b. The tension crack is vertical and is filled with water to a depth  $z_w$ .
- c. Water enters the sliding surface along the base of the tension crack and seeps along the sliding surface, escaping at atmospheric pressure where the sliding surface daylight in the slope face (Fig 2.8)
- d. The forces  $W$  (weight of sliding block),  $U$  (uplift force due to water pressure on the sliding surface) and  $V$  (force due to water pressure in the tension crack) all act through the centroid of the sliding mass. It is assumed that there are no moments which would tend to cause rotation of the block and hence failure is by sliding only.
- e. The shear strength of the sliding surface is defined by cohesion  $c$  and friction angle  $\phi$  which are related by equation (1) as discussed in chapter 2.

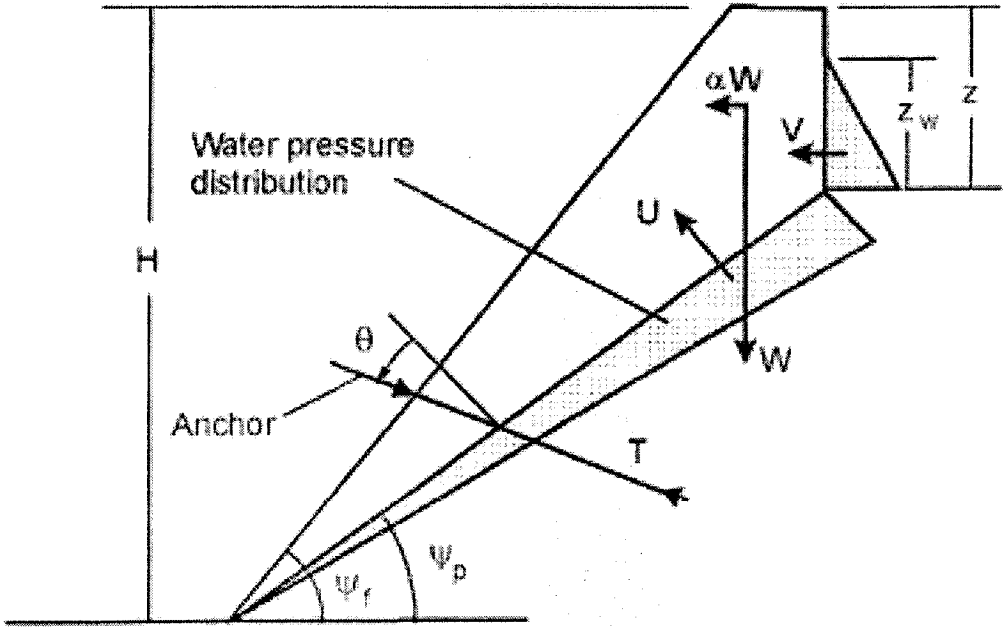


Figure 2. 8 Geometry of slope with tension crack in upper surface

In this case the factor of safety is calculated in the same way as that for the block on an inclined plane:

$$F = \frac{cA + (W \cos \psi_p - U - V \sin \psi_p) \tan \phi}{W \sin \psi_p + V \cos \psi_p} \quad (17)$$

Where

$$A = (H - z) \cdot \operatorname{cosec} \psi_p$$

$$U = \frac{1}{2} \gamma_w \cdot z_w (H - z) \cdot \operatorname{cosec} \psi_p \quad (18)$$

$$\gamma = \frac{1}{2} \gamma_w \cdot z_w^2$$

For the tension crack in the upper slope surface:

$$W = \frac{1}{2} \gamma H^2 \left\{ \left( 1 - (z/H)^2 \right) \cot \psi_p - \cot \psi_f \right\} \quad (19)$$

And, for the tension crack in the slope face:

$$W = \frac{1}{2} \gamma H^2 \left\{ (1 - z/H)^2 \cot \psi_p (\cot \psi_p \cdot \tan \psi_f - 1) \right\} \quad (20)$$

When the geometry of the slope and the depth of water in the tension crack are known, the calculation of the factor of safety is simple. It is however, necessary to compare a range of slope geometries, water depths and the influence of different shear strengths (Hoek and Bray, 1977). To simplify the calculations, Equation 17 can be rearranged to the following:

$$F = \frac{(2c/\gamma H) \cdot P + [Q \cdot \cot \psi_p - R \cdot (P + S)] \tan \phi}{Q + R \cdot S \cot \psi_p} \quad (21)$$

Where

$$P = (1 - z/H) \cdot \operatorname{cosec} \psi_p \quad (22)$$

When the tension crack is in the upper surface:

$$Q = \left\{ 1 - (z/H)^2 \right\} \cot \psi_p - \cot \psi_f \left\} \sin \psi_p \quad (23)$$

When the tension crack is in the slope face:

$$Q = \left\{ (1 - z/H)^2 \cos \psi_p (\cot \psi_p \cdot \tan \psi_f - 1) \right\} \quad (24)$$

$$R = \frac{\gamma_w}{\gamma} \cdot \frac{z_w}{w} \cdot \frac{z}{H} \quad (25)$$

$$S = \frac{z_w}{z} \cdot \frac{z}{H} \cdot \sin \psi_p$$

The ratios P, Q, R and S are all dimensionless which means that they depend upon the geometry but not upon the size of the slope. Hence, in cases where the cohesion  $c = 0$ , the factor of safety is independent of the size of the slope. The dimensionless ratios help in the generation failure charts.

#### **2.8.4 Influence of groundwater on stability**

In the preceding sections it has been assumed that it is only the water present in the tension crack and that along the failure surface which influences the stability of the slope. This is equivalent to assuming that the rest of the rock mass is impermeable. Consideration must be given to the water pressure distribution other than that upon which the analysis is based. It is however, difficult to precisely define the groundwater flow patterns in a rock mass. Consequently the only possibility open to the slope designer is to run iterative analyses to determine ranges of possible factors of safety and to assess the sensitivity of the slope to variations in groundwater conditions (Hoek and Bray, 1977).

#### **2.8.5 Wedge Failure and Analysis**

Wedge failures result when rock masses slide along two intersecting discontinuities both of which dip out of the cut slope at an oblique angle to the cut face, forming a wedge-shaped block. Commonly, these rock wedges are exposed by excavations that daylight the line of intersection that forms the axis of sliding, precipitating movement of the rock mass either along both planes simultaneously or along the steeper of the two planes in the direction of maximum dip.

Depending upon the ratio between peak and residual shear strengths, wedge failures can occur rapidly, within seconds or minutes, or over a much longer time frame, or on the order of several months. The size of a wedge failure can range from a few cubic meters to very large slides from which the potential for destruction can be enormous.

The formation and occurrence of wedge failures are dependent primarily on lithology and structure of the rock mass (Piteau, 1972). Rock masses with well-defined orthogonal joint sets or cleavages in addition to inclined bedding or foliation generally are favourable situations for wedge failure. Shale, thin-bedded siltstones, claystones, limestones, and slaty lithologies tend to be more prone to wedge failure development than other rock types. However, lithology alone does not control development of wedge failures.

A kinematic analysis for wedge failure is governed by the orientation of the line of intersection of the wedge-forming planes. Kinematic analyses determine whether sliding can occur and, if so, whether it will occur on only one of the planes or simultaneously on both planes, with movement in the direction of the line of intersection.

The necessary structural conditions for wedge failure are illustrated in Figure 2.8a and can be summarized as follows:

The trend of the line of intersection must approximate the dip direction of the slope face.

The plunge of the line of intersection must be less than the dip of the slope face. Under this condition, the line of intersection is said to daylight on the slope.

The plunge of the line of intersection must be greater than the angle of friction of the surface. If the angles of friction for the two planes are markedly different, an average angle of friction is applicable. This condition is also shown in Figure 2.9

Because the model represents a three dimensional shape, no assumptions of the lateral extent of the wedge are required. Stereographic analysis can also determine whether sliding will occur on both the wedge-forming planes or on only one of the two. This procedure is referred to as Markland's test (Hoek and Bray, 1981), which is described in Figure 2.9.

The presence of significant pore-water pressures along the failure planes can in some cases alter the possibility of kinematic wedge failures. For example, the introduction of water pressure may cause a failure even though the plunge of the intersection line is less than the average frictional strength of the planes.