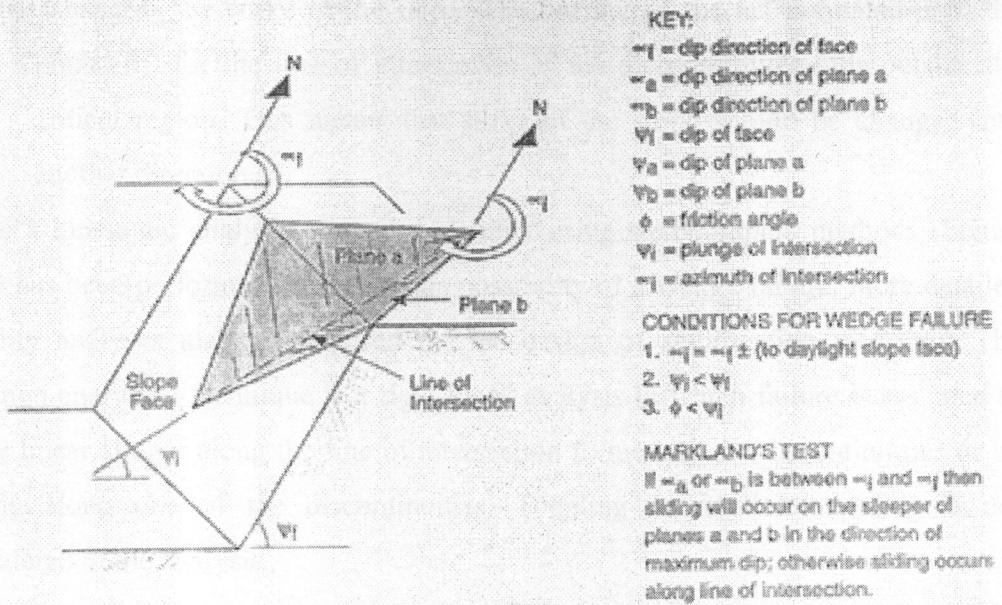
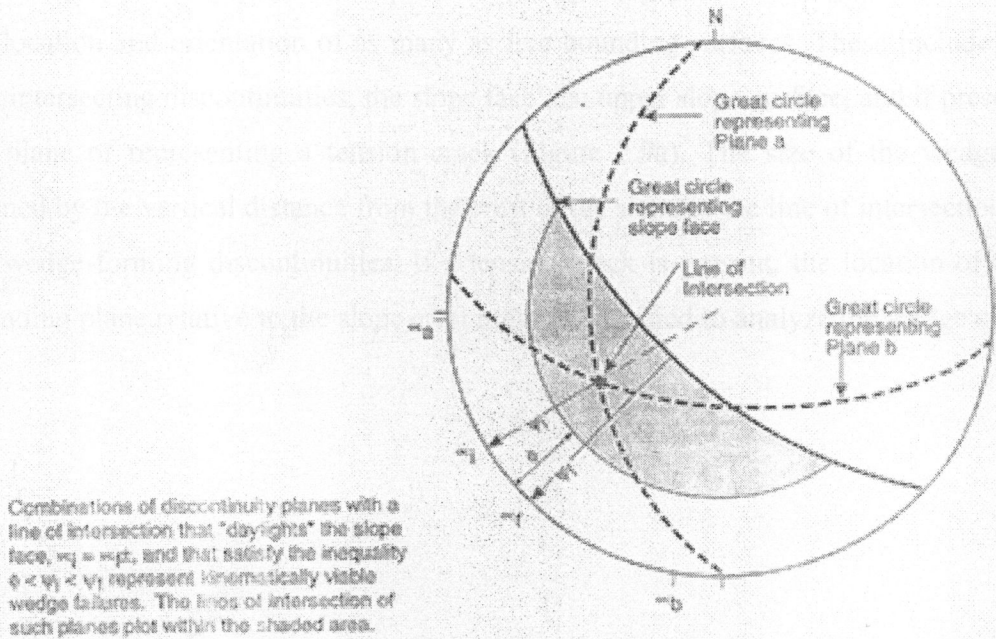




(a) Wedge Failure Model



(b) Great Circle Representation

Figure 2. 9 Kinematic analysis for wedge failure (Hoek and Bray, 1981)

There are two possible ways to prevent the wedge failure kinematically:

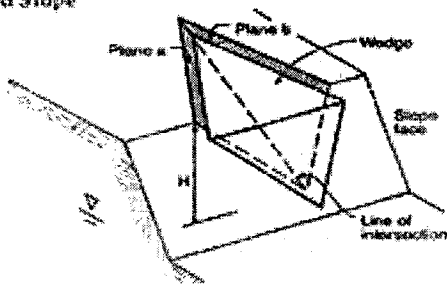
- (a) Flattening of slope: the great circle of the slope face is coincident with point (Iij) for the line of intersection of the discontinuities. Thus, this point falls behind the critical region.

- (b) Changing the strike of the slope: The overlay on the net is rotated until the point (Iij) for the line of intersection of the discontinuities falls outside the critical region. This means that strike of the slope should be changed into another direction.

Once a kinematic analysis of wedge stability using stereographic methods (Figure 2.9b) has been performed indicating the possibility of a wedge failure, more detailed stability analyses may be required for the design of stabilization measures. The common analytical technique is a rigid-block analysis in which failure is assumed to be by linear sliding along the line of intersection formed by the discontinuities or by sliding along one of the discontinuities. Toppling or rotational sliding is not considered in the analysis.

The analysis of wedge stability requires that the geometry of the wedge be defined by the location and orientation of as many as five bounding surfaces. These include the two intersecting discontinuities, the slope face, the upper slope surface, and if present, the plane of representing a tension crack (Figure 2.9a). The size of the wedge is defined by the vertical distance from the crest of the slope to the line of intersection of the wedge-forming discontinuities. If a tension crack is present, the location of this bounding plane relative to the slope crest must be specified to analyze the wedge size.

(c) Fully Drained Slope



(See Figure 15-14 for definition of angles)

$$FS = \frac{1}{\gamma H} (c_a X + c_b Y) + A \tan \phi_a + B \tan \phi_b$$

PARAMETERS:

c_a and c_b are the cohesive strengths of planes a and b
 ϕ_a and ϕ_b are the angles of friction on planes a and b
 γ is the unit weight of the rock
 H is the total height of the wedge
 X , Y , A , and B are dimensionless factors which depend upon the geometry of the wedge
 ψ_a and ψ_b are the dips of planes a and b
 ψ_i is the plunge of the line of intersection

$$X = \frac{\sin \theta_{a4}}{\sin \theta_{a5} + \cos \theta_{a5} + 1}$$

$$Y = \frac{\sin \theta_{b3}}{\sin \theta_{b5} + \cos \theta_{b5} + 1}$$

$$A = \frac{\cos \psi_a - \cos \psi_b + \cos \theta_{a5} \cdot \cos \theta_{b5}}{\sin \psi_i + \sin^2 \theta_{a5} + \theta_b}$$

$$B = \frac{\cos \psi_b - \cos \psi_a + \cos \theta_{b5} \cdot \cos \theta_{a5}}{\sin \psi_i + \sin^2 \theta_{b5} + \theta_a}$$

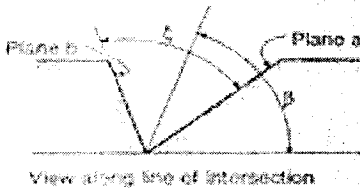
(d) Friction Only Shear Strengths

$$FS = A \tan \phi_a + B \tan \phi_b$$

(Parameters as above)

Note: Hoek and Bray (1981) have presented graphs to determine factors A and B.

(e) Friction Angle Same for Both Planes



$$FS = \frac{\sin \beta}{\sin (L/2)} + \frac{\tan \phi}{\tan \psi_i}$$

PARAMETERS:

ϕ = friction angle
 ψ_i = plunge of line of intersection
 β = see sketch
 L = angle between wedge-forming planes

Figure 2. 10 Stability equations for wedge failure

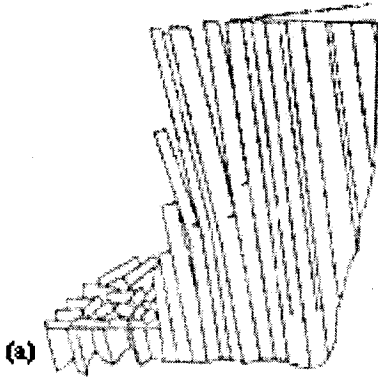
The stability of the wedge can be evaluated using the factor-of-safety concept by resolving the forces acting normal to the discontinuities and in the direction parallel to the line of intersection. These forces include the weight of the wedge, external forces such as foundation loads, seismic accelerations, tensioned reinforcing elements, forces generated by water pressures acting on the surfaces, and the shear strength developed along the sliding plane or planes.

The completely general formulation of wedge stability calculation requires adherence to a strict system of notation. Hoek and Bray (1981) presented the equations for the general analysis as well as a methodology to undertake the calculation in a systematic manner. Because the calculation is extended, this general analysis is best adapted to computer solution. However, in most cases, assumptions can be made that significantly simplify the controlling stability equations so that simple calculator or graphical methods can provide a good indication of the sensitivity of the wedge stability to alternative strength and load combinations. Figure 2.10(a-e) defines the calculation of wedge stability under various simplifying assumptions.

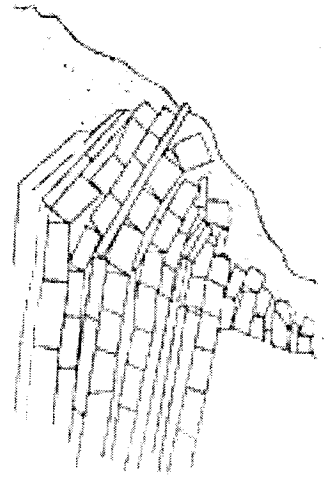
Some of the formulas given in Figure 3.4 contain dimensionless factors (X , Y , A , B) which depend upon the geometry of the wedge.

2.8.6 Toppling Failure and Analysis

Toppling failures most commonly occur in rock masses that are subdivided into a series of slabs or columns formed by a set of fractures that strike approximately parallel to the slope face and dip steeply into the face (Figure 2.10c). In a toppling failure the rock column or slab rotates about an essentially fixed point at or near the base of the slope at the same time that slippage occurs between the layers. A rare case of toppling is that of a single column defined by a unique discontinuity such as fault. Rock types most susceptible to this mode of failure are columnar basalts and sedimentary and metamorphic rocks with well-defined bedding or foliation planes (Figure 2.11c). There are several types of toppling failures, including flexural, block, or a combination of block and flexural toppling (Figure 2.11). Toppling can also occur as a secondary failure mode associated with other failure mechanisms such as block sliding. Examples of these various types of secondary toppling failure are shown in Figure 2.12



(a) Flexural toppling in hard rock slopes with well developed steeply dipping discontinuities.

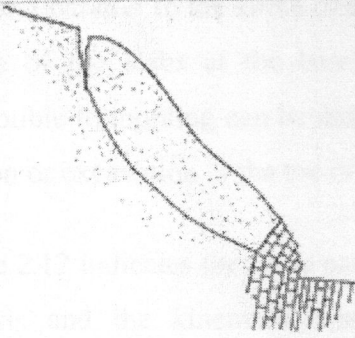


(c) Block flexure toppling characterized by pseudo-continuous flexure of long columns by accumulated motions along numerous cross joints.

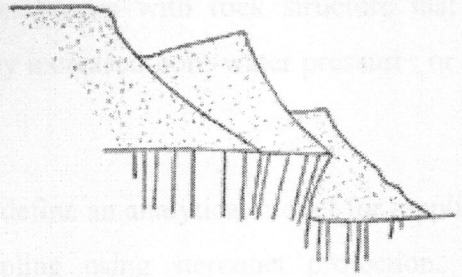


(b) Block toppling in a hard rock mass with widely spaced orthogonal joints

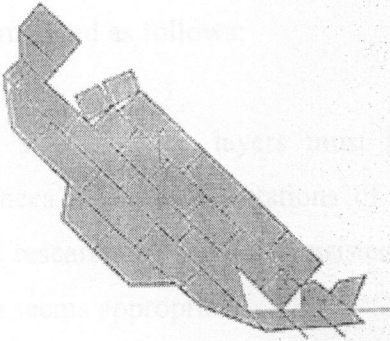
Figure 2. 11 Primary toppling modes (Hoek and Bray, 1981)



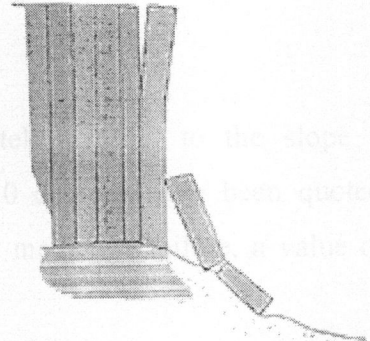
Slide toe toppling when steeply dipping beds of hard rock are loaded by instability higher up the slope.



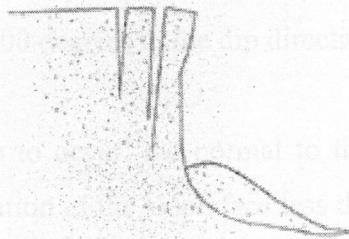
Slide base toppling when steeply dipping beds are dragged along by instability of overlying material.



Slide head toppling when movement lower in the slope frees block to topple.



Toppling and slumping of columnar rock by weathering of underlying material.



Tension crack toppling in cohesive materials.

Figure 2. 12 Secondary toppling modes

In order for toppling to occur, the center of gravity of the column or slab must fall outside the dimension of its base. Toppling failures are characterized by significant horizontal movements at the crest and very little movement at the toe. To accommodate this differential movement between the toe and crest, interlayer movement must occur. Thus, shear strength between layers is crucial to the stability of a slope that is structurally susceptible to toppling. Another characteristic of toppling movements is the antecedent development of major tension cracks behind the

crest and parallel to the strike of the layers. Failure does not occur until there is shear failure of the slabs at the base of the slope. Slopes with rock structure that is susceptible to toppling can be induced to fail by increased pore-water pressures or by erosion or excavation at the toe of the slope.

Figure 2.12 indicates the slope parameters that define an analytical model for toppling analysis and the kinematic analysis of toppling using stereonet projection. Of particular note is the presence of a stepped failure base assumed to develop along cross fractures between the columns. The necessary conditions for toppling failure can be summarized as follows:

1. The strike of the layers must be approximately parallel to the slope face. Differences in these orientations of between 15-30 degrees have been quoted by various researchers, but for consistency with other modes of failure, a value of 20 degrees seems appropriate.
2. The dip of the layers must be into the slope face. Using the dip direction convention, conditions 1 and 2 can be stated as follows: the dip direction of the layers must be between 160 and 200 degrees to the dip direction of the slope face.

In order for interlayer slip to occur, the normal to the toppling plane must have a plunge less than the inclination of the slope face less the friction angle of the surface. This condition can be formulated as follows:

$$90 - \psi_p = (\psi_f - \phi_p) \quad (26)$$

Where

ψ_p = dip of geologic layers (planes)

ψ_f = dip of slope face, and

ϕ_p = friction angle along planes

Analogous to planar failures, some limitation to the lateral extent of the toppling failure is a fourth condition for a kinematically possible failure. Because the analysis

is two dimensional, it is usually assumed that zero-strength lateral release surfaces are present or that the potential failure mass is defined by a convex slope in plan.

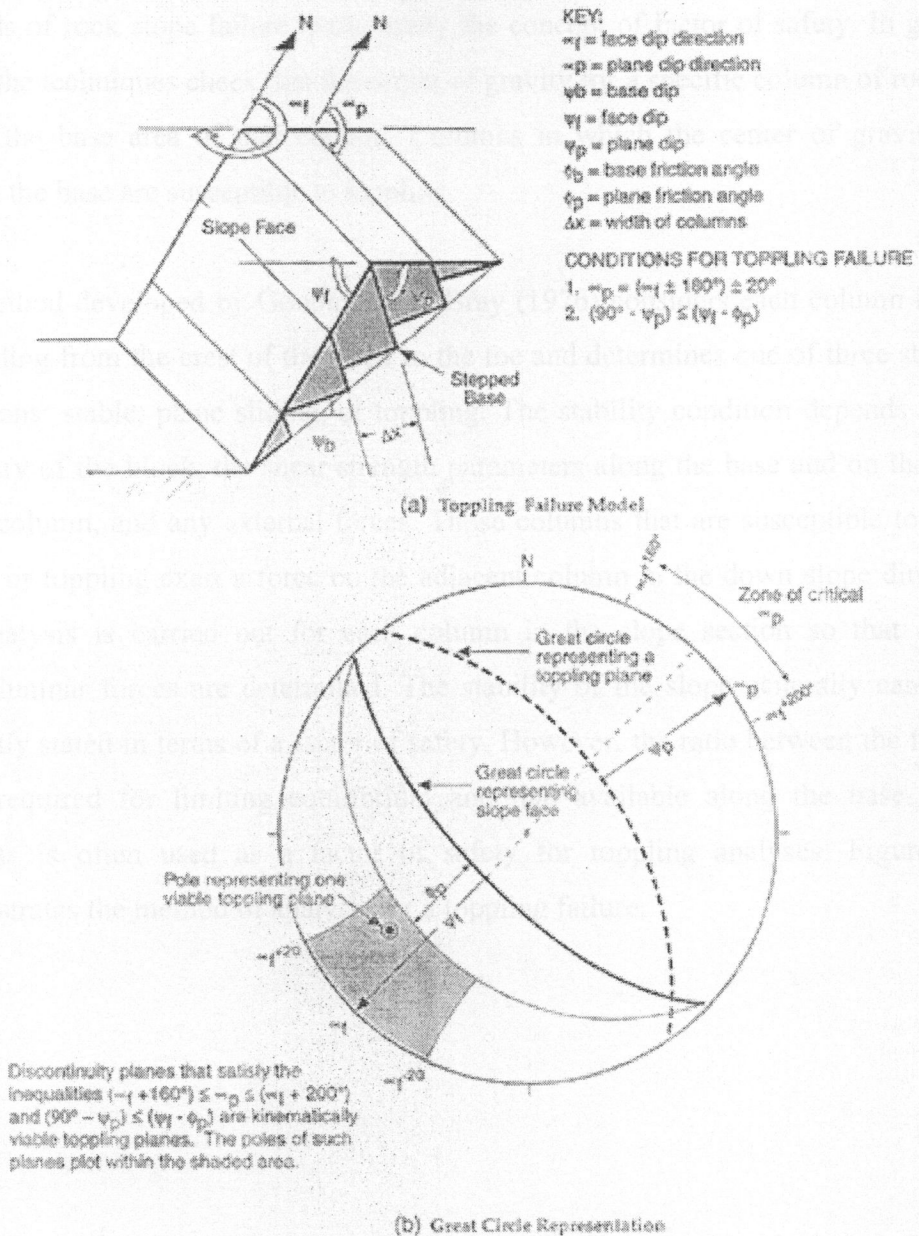


Figure 2. 13 Kinematic analysis for toppling failure (Hoek and Bray, 1981)

In order to prevent toppling kinematically, the procedure given below is followed:

- (a) Slope flattening: For this purpose, normal plot should be removed from the ruled region.
- (b) Rotation of slope orientation.

The analysis of toppling failures has been investigated by several researchers such as Goodman and Bray, (1976); Hittinger, (1978); Choquet and Tanon, (1985) cited in Hoek and Bray, (1981). The analytical procedures are not as clear-cut as for other methods of rock slope failure, particularly the concept of factor of safety. In general terms, the techniques check that the center of gravity for a specific column of rock lies within the base area of that column. Columns in which the center of gravity lies outside the base are susceptible to toppling.

The method developed by Goodman and Bray (1976) considers each column in turn proceeding from the crest of the slope to the toe and determines one of three stability conditions: stable, plane sliding, or toppling. The stability condition depends on the geometry of the block, the shear strength parameters along the base and on the sides of the column, and any external forces. Those columns that are susceptible to either sliding or toppling exert a force on the adjacent column in the down slope direction. The analysis is carried out for each column in the slope section so that all the intercolumnar forces are determined. The stability of the slope generally cannot be explicitly stated in terms of a factor of safety. However, the ratio between the friction value required for limiting equilibrium and that available along the base of the columns is often used as a factor of safety for toppling analyses. Figure 2.13 demonstrates the method of analysis for a toppling failure.

2. The base of each column forms a stepped failure base with an inclination assumed at +15 degrees ($\psi_b = 15$ degrees in Figure 3.7a)
3. No pore-water pressures are present within the slope.

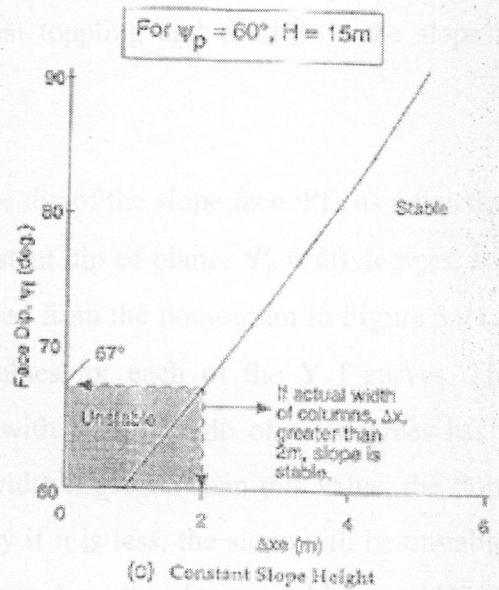
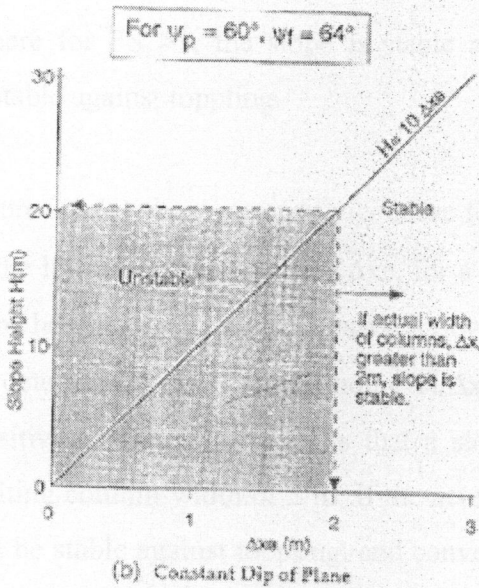
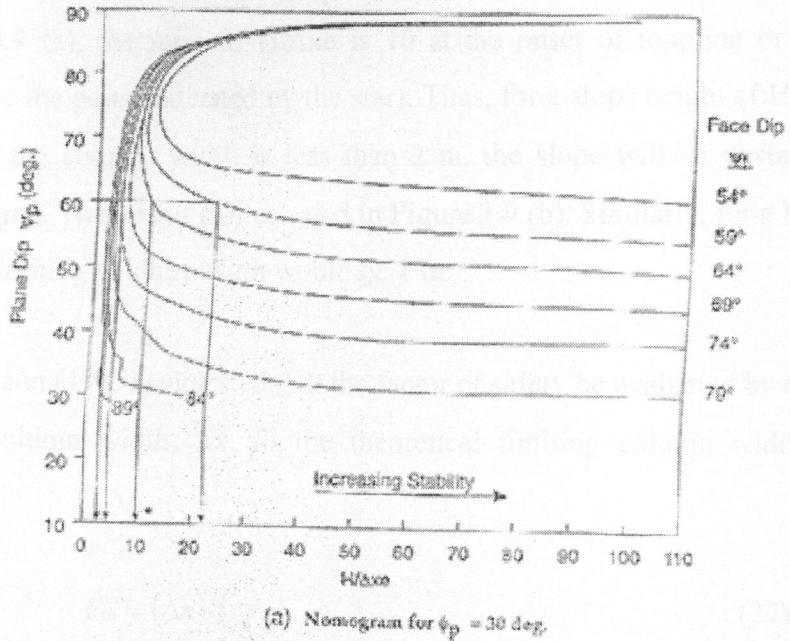


Figure 2. 15 Nomogram analysis for toppling failure (Hoek and Bray, 1981)

An example of the use of this method of analysis is illustrated in Figure 2.14. Note that the formulation yields the limiting column width, Δx_{se} , at which toppling failure occurs. The input variables for the examples are

Friction angle for plane = $\phi_p = 30$ degrees

Dip of face = $\Psi_f=64$ degrees

Dip of plane = $\Psi_p=60$ degrees

From Figure 3.9 (a), the ratio of $H/\Delta x_e$ is 10 at the onset of toppling or limiting equilibrium (see the point indicated by the star). Thus, for a slope height of $H = 20$ m, $\Delta x_e = 2$ m. If the column width is less than 2 m, the slope will be unstable with respect to toppling. This result is illustrated in Figure 3.9 (b). Similarly, for a height of $H = 10$ m, the limiting column width would be 1 m.

Choquet and Tanon (1985) suggested that the factor of safety be evaluated by the ratio of the actual column width, Δx , to the theoretical limiting column width, Δx_e , according to

$$FS = (\Delta x)/(\Delta x_e) \quad (27)$$

Where for $FS > 1$, the slope is stable against toppling and for $FS < 1$, the slope is unstable against toppling.

Figure 3.9 (c) shows a sensitivity curve for the dip of the slope face Ψ_f , as a function of the limiting column width, Δx_e , for a constant dip of plane, $\Psi_p = 60$ degrees, and slope height $H = 15$ m. This curve is developed from the nomogram in Figure 3.9 (a) by determining the corresponding $H/\Delta x_e$ values for each of the Ψ_f curves. The sensitivity curve demonstrates that a slope with a face angle of 67 degrees has a limiting column width of 2 m. If the actual width is greater than this value, the slope will be stable against toppling, and conversely if it is less, the slope will be unstable. By comparison, if the actual column width was 4 m, the slope would be stable at a face angle of 82 degrees.

Stabilization of toppling failures can be accomplished by reducing the aspect ratio of the columns in one of two ways: by reducing the slope height so that column height is reduced or by bolting layers together to increase the base width of the columns. Both of these methods change the column geometry so that the centers of gravity of the columns are within their bases.

2.9 Methods of Failure Analysis

2.9.1 Slicing Methods

The method of slices is one of the most useful methods of analysis for dealing with irregular slopes in non-homogeneous soils and rocks in which the values of c and ϕ are not essentially constant (Chowdhury, 1978). In all methods based on the slices procedure, the potential sliding mass is subdivided into a number of vertical elements or slices. These slices need not be equal in width. For example slice widths may be selected in such a way that as far as possible the base of each slice falls within one type of soil or rock mass in a non-homogeneous slope. The inclination of the base of each slice, the slice weight, the material properties and pore pressures at the base of each slice are expected to be known before an analysis can be carried out.

2.9.2 Ordinary Method of Slices (Fellenius Method)

In this method a trial slip circle with a sliding mass subdivided into a number of imaginary vertical slices. Each slice is acted upon by its own weight W and by inter-slice or sidewall forces which have both tangential components T and normal components E . the forces acting on the base of a slice of inclination α and length l are the shear resistance S and the normal force P .

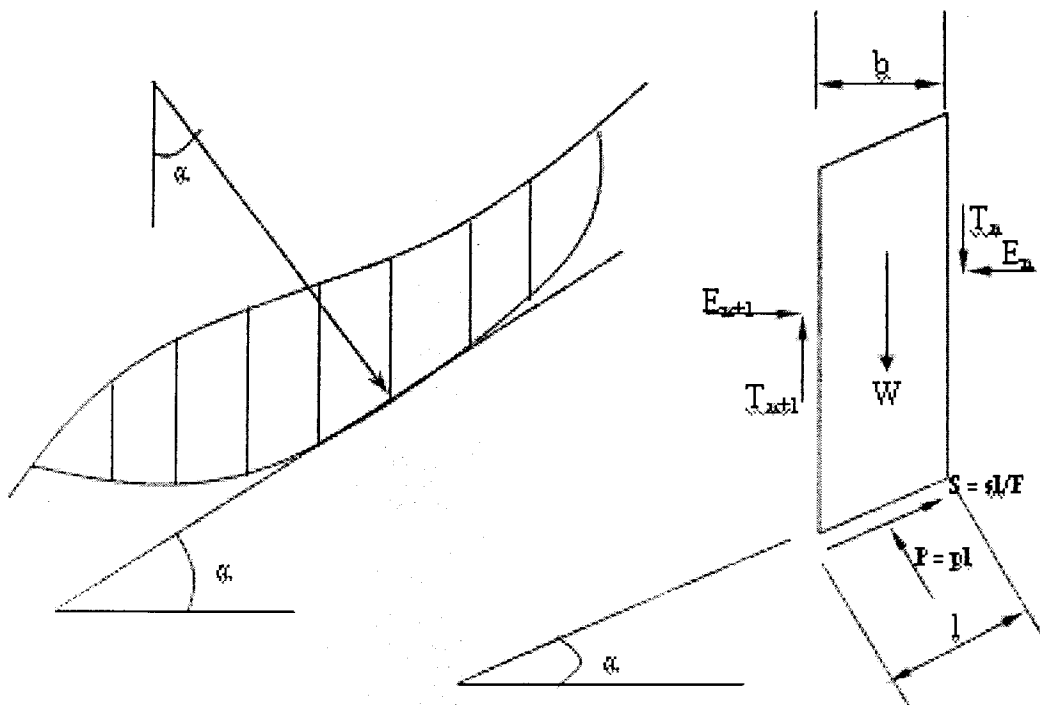


Figure 2.16 Division of sliding mass into slices and the forces acting on a typical slice

It is difficult to evaluate the forces T and E (Fig 2.15 (b)), which depend on a number of factors including the stress-strain and deformation characteristics of the material of the slope. In the simplest approach the total normal stress on the base of each slice is obtained by resolving all forces perpendicular and tangential to the base of a slice, after assuming the inter-slice forces T and E to be zero. Neglecting the inter-slice forces on each slice gives the following equilibrium conditions to the sliding mass, consisting of n slices:

$$\sum \tan \phi \{ (T_n - T_{n+1}) \cos \alpha - (E_n - E_{n+1}) \sin \alpha \} = 0 \quad (28)$$

Where the summation is over all the slices.

Such an assumption leads to an under estimate of the factor of safety. Therefore, the results are conservative and the error is on the safe side. The inaccuracy is large for deep critical circles with large variations in α . This corresponds to the arcs with large central angles. Inaccuracy also increases with increasing pore water pressures.

2.9.3 Bishop's Simplified Method of Slices

The ordinary method of slices satisfies only the overall moment equilibrium, neglects the moment equilibrium of the individual slice (by neglecting inter-slice forces), and only approximates the force equilibrium of each slice. Bishop formulated the problem rigorously by including the inter-slice forces in the equations of equilibrium of a typical slice. He demonstrated the solution of the correctly formulated problem by a process of iterative calculations in which assumed values of the inter-slice forces or sidewall forces are improved by successive approximations.

2.10 General Engineering Characteristics of Some Soil and Rock.

A list of soil types and their general engineering characteristics as presented in the following sections is not an all-inclusive list. It is intended to show that, with some experience, knowledge of the soil type and its history can provide preliminary information regarding the soils, general engineering characteristics. A description of the soil types in a given area is often available for many projects and can be obtained from geologic maps, soil maps, and reports of subsurface investigations made at adjacent sites.

- Loose sand is simply a sand deposit with a low density. Vibratory loads tend to densify these deposits. Loose sands present problems in high seismic risk areas, because seismic loading can induce liquefaction if the sand is saturated, as well as cause significant settlement.
- Loess is a deposit of relatively uniform, windblown silt. It has a relatively high vertical permeability but low horizontal permeability. This often causes problems with hydraulic structures such as canals and earth dams.
- Normally consolidated clays are clay soils that have never been subjected to a pressure greater than the existing pressure. These soils tend to be highly compressible, have a low ultimate bearing capacity, and, as with all clay soils, have a very low permeability
- Over consolidated clays are clays that in the past have been subjected to a pressure greater than the existing pressure. Highly over consolidated clays generally tend to have a rather high ultimate bearing capacity and are relatively incompressible.

- Bentonite is a highly plastic clay resulting from the decomposition of volcanic ash (Dunn et al, 1980)

2.10.1 Particle Size and Gradation

Brady and Weil (2004) define clay as soil particles smaller than 0.002 mm in diameter whereas Smithson et al (2002) also include secondary clay minerals which are synthesised from chemical weathering to this definition. Clay minerals are formed by the fusion of alumina and silica (aluminium silicates). The silica is in the form of a sheet of silica tetrahedra. The silicon atom (Si) at the centre of the tetrahedron is bound by four oxygen atoms.

The alumina consists of an aluminium atom (Al) equidistant from six oxygen (O) or hydroxyls (OH). The silicon-oxygen and aluminium-oxygen structural units are responsible for the sheets that result, thus giving the clay minerals the platy, crystalline structure (Smithson et al, 2002). In kaolinite (a type of clay) the silica and alumina sheets are held together by strong oxygen bonds that make the structure non-expanding. Illites and montmorillonites have a structure with holes in them and they have repeating sheets of alumina sandwiched between two silica sheets, held together by potassium ions

The small size of these particles results in them having a very large specific surface area, making them have a higher capacity for adsorption.

The size of soil particles range from boulders more than 1m in diameter to clay size particles less than 0.001 mm in diameter. Practically all the minerals with a sheeted structure which are encountered in the very fine soil fraction belong to a group known as the *clay minerals*. Most of the minerals of this group can be referred to one of three subgroups known as the *kaolinites*, the *illites*, and the *montmorillonites*. Each of these is characterised by an arrangement of atoms leading to a negative electrical charge on the flat surfaces of the crystals. Pure water consists primarily of molecules of H_2O , but a few of the molecules always dissociate into hydrogen ions H^+ and hydroxyl ions OH^- . If impurities are present such as acids or bases, the impurities also dissociate into positively charged cations and negatively charged anions. Since the plane surfaces of the clay minerals carry negative electrical charges, the cations including the H^+ furnished by the water itself are attracted toward the surfaces of the particle. Such a

cation is said to be adsorbed. Surrounding water ions are also affected in a way which makes an adsorption complex. This complex gives the clay soil, in general, the properties of swelling whenever pressure is removed from it. (Dunn et al, 1980).

Figure 2.17 shows the upper strata soil materials found at COP F, western wall. A high occurrence of limonite can be seen to overlay much of the strata and this is a source of concern as far as soil stability is concerned.

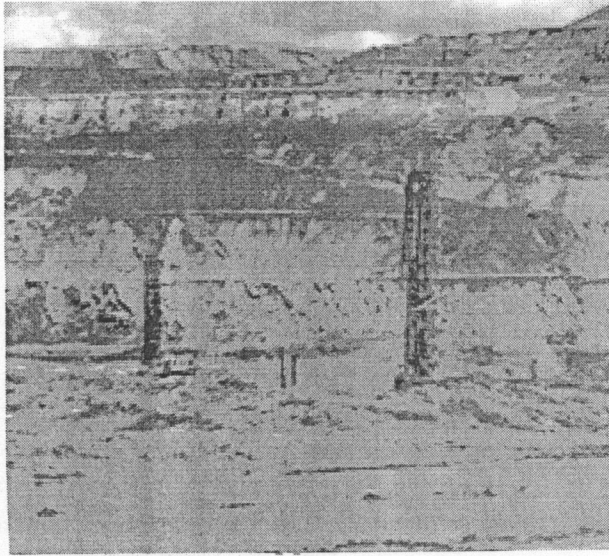


Figure 2. 17 Soil of the Nchanga Open Pit F (KCM, 2006)

The effective particle size of a given soil is defined as a particle size for which 10% (D_{10}) of the material by weight is less than that size. Other particle sizes are also frequently used in describing or classifying soils. The D_{50} of a soil is the median particle size.

The distinction between a uniform and a well-graded soil can be define numerically by the uniformity coefficient C_u and the coefficient of curvature C_z . the uniformity coefficient is defined by the ratio

$$C_u = \frac{D_{60}}{D_{10}} \quad (29)$$

The coefficient of curvature is defined as

$$C_z = \frac{D_{30}^2}{D_{10}D_{60}} \quad (30)$$

Soils with C_u less than 4 are said to be uniform, and soils with C_u greater than 4 (6 for sands) are well graded provided that the grain-size distribution curve is smooth and reasonably symmetrical (Dunn et al, 1980).

2.10.1.1 Weight-Volume Relationship

The relationships defined below are very useful in describing and evaluating the physical properties of soils. These relationships give an insight into the interactions of soil particles and ground water systems in the presence of stresses due to excavation or the super incumbent structural weight of the litho strata, in case of sedimentary or meta sedimentary formations (Dunn et al, 1980).

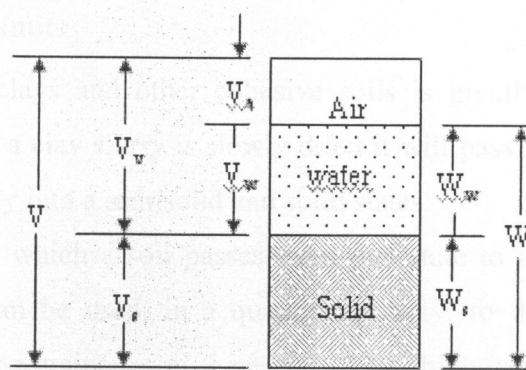


Figure 2. 18 Schematic representation of a soil element with volume relationships (Dunn et al, 1980)

V = total volume

W = total weight

V_v = volume of voids

W_w = weight of water

V_a = volume of air

W_s = weight of solids

V_w = volume of water

V_s = volume of solids

The total volume of the element of soil schematically represented in Fig. 2.18 is the sum of the volume of voids (V_v) and the volume of solids (V_s). The volume of voids is the sum of the volume of gas (usually air) (V_a) and the volume of liquid (usually water) (V_w). There are three useful relationships between the volumes of each phase.

- The void ratio (e) is the ratio between the volume of voids and the volume of the solids; it is always expressed as decimals. Void ratio is used extensively in soil mechanics to express various physical parameters as a function of the soil's density and it is given as:

$$e = \frac{V_v}{V_s} \quad (31)$$

- Porosity (n) is the ratio of the volume of the voids to the total volume; it is expressed either as a decimal or as a percentage, which is given as:

$$n = \frac{V_v}{V} \quad (32)$$

- Degree of saturation (S_r) is the ratio of the volume of the water to the volume of the voids; it is given by:

$$S_r = \frac{V_w}{V_v} \quad (33)$$

2.10.1.2 Atterberg Limits

The consistency of clays and other cohesive soils is greatly influenced by water content of the soil. If a clay slurry is slowly dried it will pass from a liquid state to a plastic state and finally into a semisolid and solid states.

The water content at which a soil passes from one state to another is different for different soils and can be used, in a quantitative way, to distinguish between, or classify, different fine-grained soil types. At very high water contents, the soil behaves as a viscous liquid in that it flows and will hold a specific shape.

2.10.1.2.1 Liquid Limit (LL)

This is the lowest content at which the soil is in liquid state. A specific test procedure has been developed to determine this water content. The test is performed by placing a soil pat in a cup and making a standard-sized groove in the pat. The cup is then dropped onto a hard surface from a height of 10 mm. the liquid limit is defined as the water content when the groove closes over 12.7 mm (1/2 in.) at 25 blows.

2.10.1.2.2 Plastic Limit (PL)

Soil is considered to be in a plastic state when it can be moulded or worked into a new shape without crumbling. The lowest water content at which the soil is considered to be in plastic state is the plastic limit of the soil. The test for plasticity is done by

rolling a pat of soil into a thread. When the thread begins to crumble at a diameter of 3.18 mm, the water content is the plastic limit.

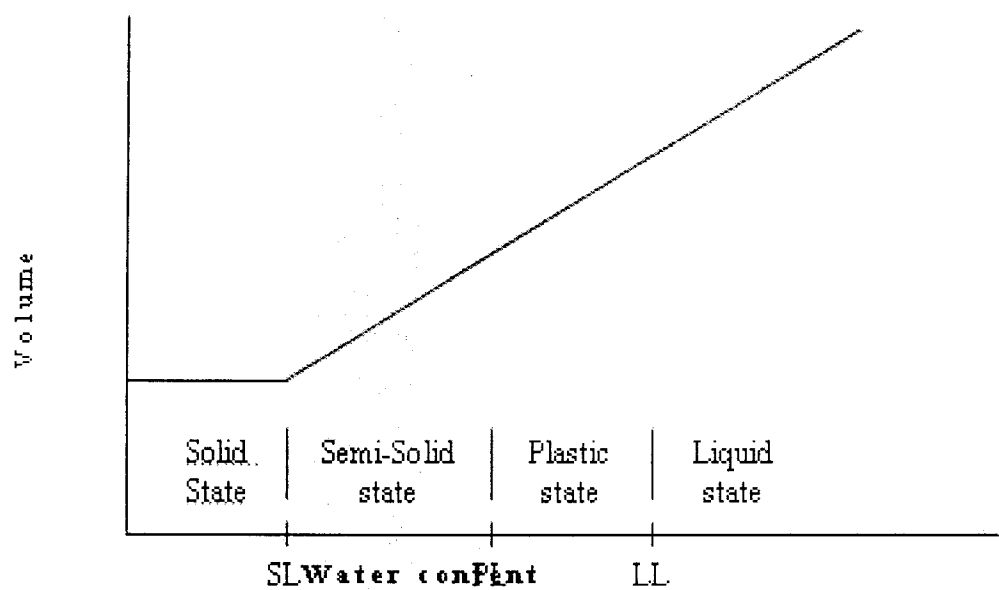


Figure 2. 19 States of soil consistency (Dunn et al, 1980)

The above soil states in Figure 2.19 are referred to as the Atterberg limits. It is necessary to review soil properties in this study as the interactions of groundwater, soil and rocks play a pivotal role in slope stability.

2.10.1.2.3 Plasticity Index (PI)

The plasticity index is the range of water content within which a soil is plastic; the finer the soil the greater the plasticity index, Smith, 2006.

$$\text{Plasticity index} = \text{Liquid limit} - \text{Plastic limit}$$

$$PI = LL - PL \tag{34}$$

2.10.2 Soil Classification

Just as it is important to classify rock masses in rock mechanics, soil classification is cardinal in soil mechanics for the simple reason of evaluating soils well before excavations and/or construction can begin.

Grain size seems to be an obvious method to classify soils, and most of the earliest efforts to establish classification systems were based on grain size. The common classification systems include:

- The MIT system.
- The AASHTO system.
- The Unified system

The most commonly used of these classifications is the Unified system. This system divides soils into three main groups: coarse-grained, fine grained, and highly organic. Coarse grained soils are those with more than 50 percent of the material retained on the No. 200 (75 microns) sieve. Coarse-grained soils are divided into gravels (G) and sands (S). The gravels and sands are grouped according to their gradation and silt or clay content, as well graded (W), poorly graded (P), containing silt material (M), or containing clay material (C). A typical classification thus might be GP for poorly graded gravel.

Fine-grained soils are those for which more than 50 percent of the material passes the No. 200 sieve. They are divided into silts (M), clays (C), and organics silts and clays (O), depending on how they plot on the plasticity chart. (Dunn et al, 1980; Hoek and Bray, 1977).

2.10.3 Rock Mass Classification.

Rock mass classifications have been used for rock characterisation and creation of geotechnical zones, for geotechnical analysis. Rock mass classification systems have been developed for over 100 years ago when Ritter in 1879 attempted to develop an empirical approach to tunnel design, particularly for support requirements (Hoek 2007).

2.10.3.1 Rock Mass Classification Systems

Many researchers presented rock mass classification systems that have been used over the time; in this section the most notable ones will be presented.

2.10.3.2 Terzaghi's Rock Mass classification

In the paper, Terzaghi, 1946; cited in Hoek et al, (1977), reference is made to the earliest use of rock mass classification for the design of tunnel support, in which rock loads are estimated on the basis of descriptive rock classification.

Terzaghi's description of the rock masses formulates a good example of the type of engineering geology information most useful for engineering design (Hoek, 2007).

As Stacy, (2001: 20) rightly puts it

Quantitative Classification of rock masses has become almost routine, since it provides a rapid means of quantifying the quality of a rock mass, comparing qualities and assessing support requirements (*for underground mining*).

Rock mass classification systems have been identified as engineering rock mass classifications and geomechanics rock mass classifications. Based on the scope of this study, emphasis was put on the geomechanics rock mass classification. This forms the basis of the rock characterisation and creation of geotechnical zones for a better understanding of the rock mass.

2.10.3.3 Geomechanics Rock Mass classification (After Bieniawski).

In 1976, Bieniawski first published the Geomechanics Classification System which he called the Rock Mass Rating (RMR) System. Over the years this system has been refined to include more case records to the 1989 classification (Bieniawski, 1989), of the geomechanics classification on which the following description will be based. This system derives a rock mass rating (RMR), obtained by summing up five parameter values and adjusting this total by taking into account the joint orientations. The parameters are presented in Table 2.2.



2.10.3.3.1 Rock Quality Designation (RQD)

The rock quality designation index (RQD), (Table 3.1), was developed by Deer (Deer *et al* 1967, cited in Hoek, 2007; Palmstrom, 2005), to give an estimate of the rock mass quality from rock drill cores logs. RQD is defined as “the percentage of intact core pieces longer than 100mm in the total length of core”. The core diameter should be at least of NX size (54.7mm) (Palmstrom, 2005).

$$RQD(\%) = \frac{\sum \text{corepieces} > 100\text{mm}}{\text{coreLength}} \quad (35)$$

The most important use of RQD is as a component of the RMR and Q rock mass classifications. RQD gives an average estimation of the degree of jointing along the core run.

2.10.3.3.2 Limitations of the RQD

The RQD system has limitations, some which are worth mentioning in this study. For example, $RQD = 0$ where the joint intercept (distance between the joints in the rock cores) is 10cm or less, while $RQD = 100$ where the distance is 11cm or more, see Fig. 2.20.

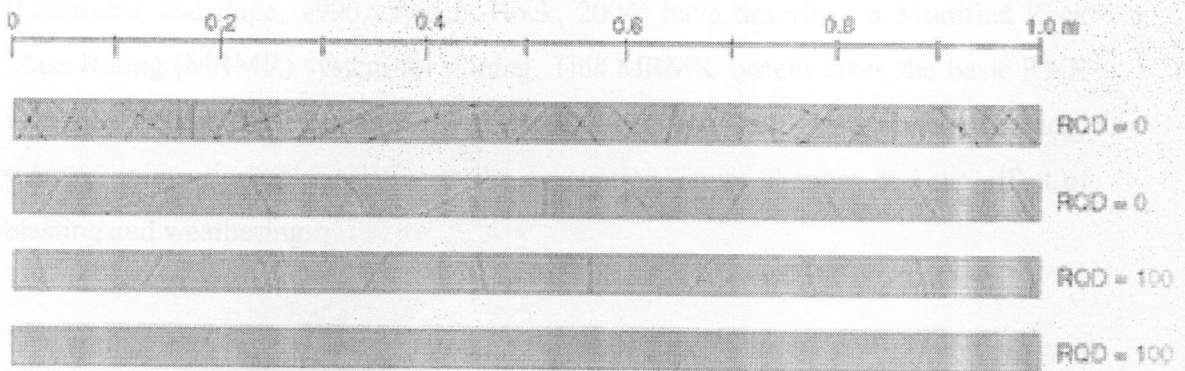


Figure 2. 20 Examples of maximum and minimum values for various joint densities along drill cores (Palmstrom, 2005). Figure not to scale

However, RQD values are acceptable when double coring method is used, when NX size of the core is under discussion and given that the method of drilling is rotary diamond drilling.

Another drawback is that the RQD gives no information on the remaining core pieces which are less than 10cm, i.e. it does not matter whether the discarded pieces are earth-like materials or fresh rock pieces up to 10cm length.

Palmstrom (2005) suggests that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for rock masses is:

$$RQD = 115 - 3.3J_v \quad (36)$$

Where J_v is the sum of the number of joints per unit length for all joint (discontinuity) sets, known as the *volumetric joint count*.

2.10.3.4 Modification to RMR for Mining

Bieniawski's Rock mass Rating (RMR) system was originally based upon case histories drawn from civil engineering. Consequently, the mining industry tended to regard the classification as somewhat conservative and several modifications have been proposed in order to make the classification more relevant to mining application.

(Laubscher and Page, 1990, cited in Hoek, 2004) have described a Modified Rock Mass Rating (MRMR) system for mining. This MRMR system takes the basic RMR value, as defined by Bieniawski, and adjusts it to account for in-situ and induced stresses, joint orientation relative to the excavation, stress changes and the effect of blasting and weathering.

The mining rock mass classification treats the joint conditions and ground water a little differently from the Bieniawski 1989 rating system as shown Table 2.3.

Table 2. 3 Adjustments for joint conditions (JC) and groundwater ratings

Parameter	Description		Dry Condition	Wet Conditions		
				Moist	Moderate Pressure 25-125 l/mm	Severe Pressure >125 l/mm
A Joint Expression (Large Scale Irregularities)	Wavy	Multi Directional	100	100	100	95
		Unidirectional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint Expression (Small Scale Irregularities or Roughness)	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall Alteration Zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint Filling	No fill-Surface staining only		100	100	100	100
	Non Softening and sheared material (clay or talc free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (e.g. Talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness <amplitude of irregularities		40	30	10	
	Gouge thickness >amplitude of irregularities		20	10	Flowing material 5	

Source: Stacy, (2001)

In the Mining Rock Mass Rating, a weathering adjustment is relevant when the rock types occur which are susceptible to deterioration over time. The adjustment percentages are given in Table 2.4.

Table 2. 4 Weathering adjustments

Description of weathering extent	Rate of weathering and adjustment (%)				
	6 months	1 year	2 years	3 years	4+ years
Fresh	100	100	100	100	100
Slightly	88	90	92	94	96
Moderately	82	84	86	88	90
Highly	70	72	74	76	78
Completely	54	56	58	60	62
Residual soil	30	32	34	36	38

Source: Stacy, (2001)

In explanation of the adjustments given in this table, if the rock will weather to a residual soil in 6 months, the adjustment is 30%; if it will be only slightly weathered after 3 years, the adjustment is 94%.

The orientation of the joints, and whether or not the bases of blocks formed by the joints are exposed, have a significant effect on excavation stability. The joint orientation adjustment depends on the orientations of the joints with respect to the vertical axis of the block. See Table 2.5.

Table 2. 5 Adjustments due to joint orientation

Number of joints defining the block	Adjustment (%)				
	Number of faces inclined away from the vertical				
	70	75	80	85	90
3	3	-	2	-	-
4	4	3	-	2	-
5	5	4	3	2	1
6	6	5	4	3	2 or 1

Source: Stacy, (2001)

The quality of blasting has an influence on the fracturing and loosening of the rock mass. Adjustments are as shown in Table 3.5.

Table 2. 6 Adjustment for blasting impacts

Excavation Technique	Adjustment (%)
Boring	100
Smooth wall blasting	97
Good conventional blasting	94
Poor blasting	80

Source: Stacy, (2001)

2.10.3.5 The Q system.

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al, (1974) of the Norwegian Geotechnical Institute, proposed a Tunnelling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (37)$$

Where;

RQD is the Rock Quality Designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor.

In explaining the meaning of the parameters used to determine the value of Q, Barton et al (1974, cited in Hoek, 2007) offer the following comments:

The first quotient (RQD/J_n), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5) and (10/20) differing by a factor of 400. If the quotient is interpreted in the units of centimetres, the extreme 'particle size' of 200 to 0.5 are seen to be crude but fairly realistic approximations.

(J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact.

(J_w/SRF) consists of two stress parameters. SRF is a measure of: 1) loosening load in the case of an excavation through shear zones and clay bearing rock, 2) rock stress in competent rock, and 3) squeezing loads in plastic incompetent rocks. It can be regarded as the total stress parameter. J_w is a measure of water pressure, which has an adverse effect on the shear strength of the joints due to a reduction in effective normal stress. In addition, water may cause softening and possible out-wash in the case of clay-filled joints. (cited in Hoek, 2007)

Bieniawski (1989), developed a relationship between the Q-system and the RMR from empirical analysis as give below which has been found quite satisfactory:

$$RMR = 9 \ln Q + 44 \quad (38)$$

2.10.3.6 The Geologic Strength Index (GSI)

Marino, et al, (2005), described the Geologic Strength Index as a system of rock mass characterisation that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks.

The geological character of rock material, together with the visual assessment of the mass it forms, is used as a direct input to the selection of parameters relevant for the prediction of rock mass strength and deformability. This approach enables a rock mass to be considered as a mechanical continuum without losing the influence geology has on its mechanical properties.

Because the RMR and the Q system were heavily dependent on the RQD classification introduced by Deer (1964), and that the RQD in most of the weak rock masses is essentially zero or meaningless, it became necessary to consider an alternative classification system (Marinos, et al, 2005).

The GSI was developed not to replace the Rock mass rating System, but to fill the gaps on information left in describing rock masses that could not be invariably described in the RMR system. It became an appropriate system for rock slope stability as it placed greater emphasis on basic geological observations of rock mass characteristics, reflected the material, its structure and its geological history. This index has now been incorporated as a basic system for the Hoek and Brown failure criterion, which is the basis for the slope stability analysis software called *RocLab*. See Appendix I.

CHAPTER THREE

FIELD DATA ACQUISITION

The approach in this study has included field, laboratory and desk analyses. In addition, field observations of pit slopes and the trends of smaller failures provided helpful information on the general characteristics and behaviour of the rock slopes. The methodology presented below was aimed specifically to achieve the objectives given in chapter one.

3.1 Site Investigations (visits and monitoring), COP F/D

Site investigations formed the largest part of the research activities as raw data was collected on which this whole study depended. Care was taken in data collection as to collect all the important information leading to the achievement of the objectives mentioned in the sections above. In site investigation, data was collected in:

3.1.1 Cell Mapping.

This was conducted to obtain information on the geologic structures characterising the site under investigation. Mapping was carried out in cells (Appendix 2). This is where the slope faces in the pit were demarcated and the coordinates were picked for areas of maximum horizontal distances of 30m. Pictures of these cells were taken for later analysis. Physical investigations of the area were then carried out by inspecting the ground for various geological structures, with emphasis on the discontinuities. The information was obtained on the following:

3.1.1.1 Discontinuity Mapping.

The discontinuities were investigated and recorded. These were either the bedding³ or joints. The analysis of these structures contributes to the prediction of failure mechanisms. For these structures the parameters below were collected to help describe and analyse the face in a detailed manner:

³ The Nchanga Mining Area's geology is that of meta-sediments, underlain by granite intrusion.

- **Discontinuity Spacing**

This describes the direction of the joint spacing. This is defined as the perpendicular distance between two joints of the same set. The spacing direction traces the path of the joint system and it can be vertical, horizontal, wavy, curved, undulating or even straight.

- **Joint Infilling**

When discontinuities are formed in a rock mass, this becomes a probable plane for failure to occur. The strength of a joint plane is controlled by the presence of the joint infill and its characteristics. The friction generated by the discontinuity plane must overcome the forces that induce failure⁴. When there is no joint infill, the strength of the joint surfaces will depend on the strength of the joint walls.

Hencher, (1995, cited in Sjöberg, 1996), presents the factors which contribute to shear strength of a discontinuity as, (figure 3.1):

1. Adhesion bonding
2. Interlocking of surface asperities and ploughing through asperities
3. Overriding of surface asperities
4. Shearing of rock bridges and locked asperities

Adhesion bonding is quite significant for metals but not so much for the rocks. while interlocking of minor asperities and the damage to these increases the basic friction angle for the rock units. The third factor, overriding of surface asperities occurs on a large scale on discontinuity surfaces and adds significant dilation to the shear behaviour.

⁴ This is the definition of the factor of safety.

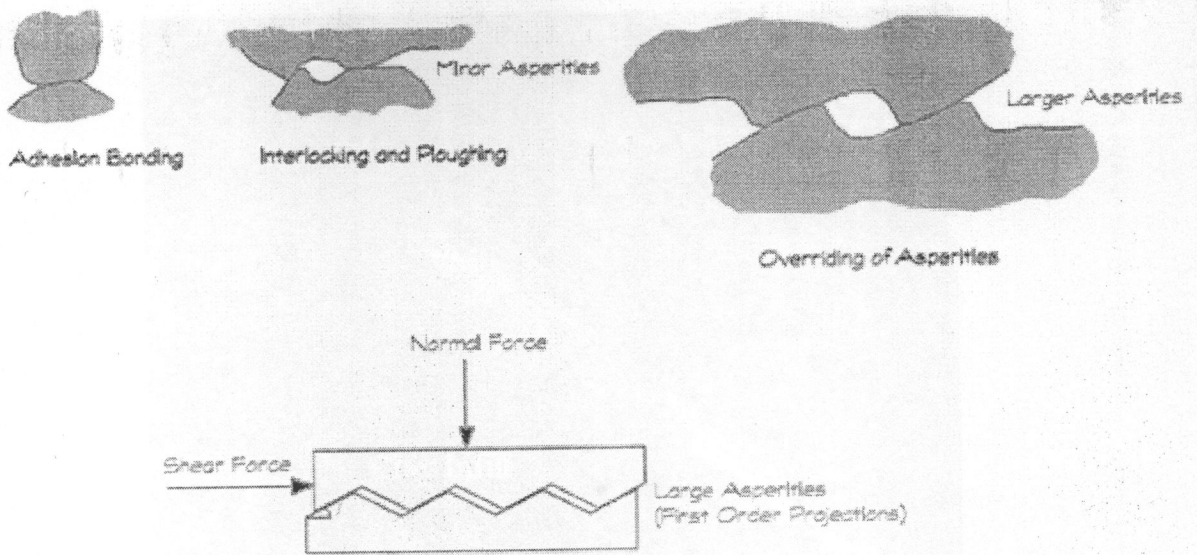


Figure 3.1 Factors contributing to the shear strength of discontinuities and an illustration of joint asperity angles (Sjoberg, 1996)

Sometimes the discontinuities are filled by clay minerals. This drastically reduces the shear resistance that these discontinuities can offer during failure. This is usually the source of most plane failure surface kinetics. When water soaks the clay minerals, the pore pressure created reduces the normal force holding the two sides of the discontinuity together and the upper part will easily slide off to cause failure, (figure 3.2), as clay minerals have this tendency to swell leaving the joint walls lubricated.

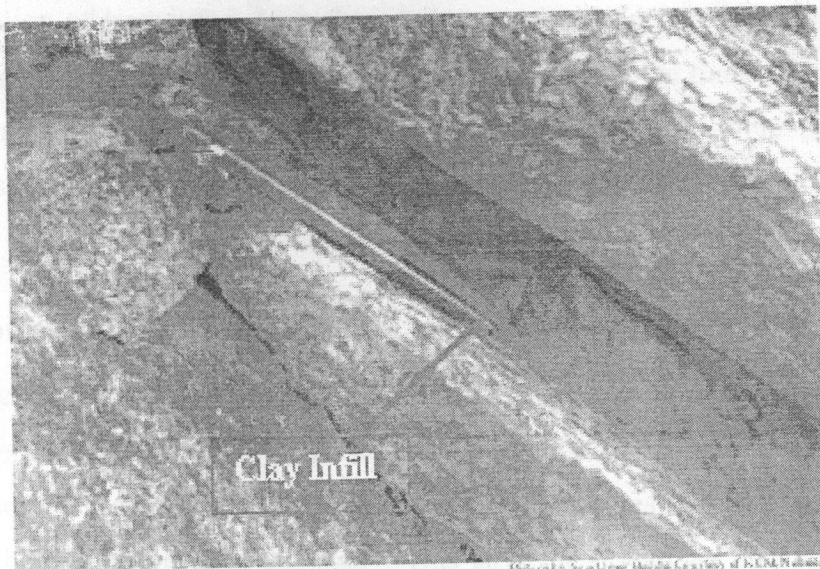


Figure 3. 2 Discontinuity filled with clay mineral. COP F/D, field data, 2006)

In cell mapping the discontinuities were classified according to whether the infilling was present and if so the condition of the infilling. way the infilling was found and they were rated towards the strength of in-situ conditions.

Finally field estimations of uniaxial compressive strengths for in situ rock were collected using the geologic hammer. With this hammer, observations were made when the rock scratched, given a blow and determining how many blows would break the rock into pieces.

The mapping information was recorded in mapping sheets specifically designed to capture as much information as possible on the pit slope faces. From this designed database, initial rock characterisation was done to get rock mass parameters for rock classification under Bieniawski⁸⁹ and Laubscher⁹⁰. For the sake of uniformity, field estimations were used in all calculations for the rock mass rating.

3.1.1.2 Ground Water

One important aspect of site investigation in this study was the observation and review of groundwater. COP F/D is characterised by a high water table. A pumping programme was initiated which was required to pump out 10 000 m³/hr of water from

a combination of bore holes. More bore holes were sunk in the COP F/D phase III area (study site), preceding the operational phases I and II at the time of this research. Observations were also made on the effect of the high water table on the rock formations (weathering and leaching), and the aquifer system of the area.

3.1.2 Geotechnical Core Logging

Geotechnical core logging was the first activity in data collection method for this study. Physical inspections of the rock cores were done at the time of drilling up to the core storage shade. Logging was also conducted 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.

Sampling of the rock cores was carried out for laboratory testing to determine strength factors of the rock formations. The logging exercise emphasised on the following geotechnical parameters:

- RQD length; determined by measuring the length of the core pieces that were more than 10 cm, divided by the run length of the core. A high RQD indicated a degree of reliability in the rock (intactness, strength, hardness, etc), while a lower RQD signified that the rock would have stability problems, highly fragmented ground; and requiring further geomechanical analysis. For drill cores, extra care was taken in ensuring that only natural breaks were considered as discontinuities from which the RQD was calculated, as there is a tendency in logging methodologies to mistake drill breaks for consideration into the calculation of RQD.
- Joint set number: determined by considering all possible jointing systems on the core run based on the observations of the orientations and characteristics (joint aperture, etc) of the discontinuities. The joint set number shows how blocky the rock mass is and it gives an insight into the possible failure mechanisms.
- Rock formations were also a critical input parameter in to the rock characterisation process. Observations were made to determine the geologic

formations and their contacts. Close geotechnical investigations were conducted in the contact areas of these geologic formations, as they often pose stability problems.

- Joint conditions were physically observed to determine the weathering, identify the joint infilling (if any) and to feel the roughness of the joint walls. The type of infill had an impact on the shear strength of the discontinuity. Kaolinitic infills (clayey) are usually a source of stability worry for geotechnical engineers in the presence of ground water or rainfall as the effective normal stress is reduced on the plane created by the discontinuity. This comes as a result of swelling of the clay infilling in the joints. A favourable situation is when the joints are not filled and the joint walls are rough.

The intactness of the rock was also predicted from the core recoveries. High recoveries were observed (as the case should be) on formations with high intact rock strengths. As in field mapping, intact rock strengths were estimated on-site with use of the geologic hammer.

The rock characteristics analysed in geotechnical cell mapping were compared to those determined in geotechnical core logging. This helped to close up gaps of incongruence and reduce descriptive errors.

Other information was collected by observational visits in the field. 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.

3.1.3 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.

I. Observation of soil/rock mass patterns.

The COP F/D phase II mining area is similar in both rock formation (geology) and groundwater regime to Phase III. Inspection of soils and rock masses at depths (new ground from mining) was conducted as mining progressed.

II. Crack monitoring

Tension cracks presented a geotechnical challenge in slope management. Site inspections were conducted at almost all slopes to identify and locate tension cracks. Extensometers were used to monitor the progress of these cracks. Electrical triggers in the extensometers were helpful in giving early warning signs for the evacuation of workers in case of a slip failure. Also other cracks were buried to prevent soaking the infill (especially kaolinitic) which would reduce the shear strength of the discontinuities.

III. Observation of potential slip surfaces

The orientations of bedding and jointing can be very worrisome to geotechnical engineer, especially if the two geologic structures are trending almost in the same way. Field inspections were made in the pits on the footwall and hanging wall to determine the presence of slip surfaces.

These inspections generally helped in filling up the gaps that may have been left out during earlier research.

3.2 Data Analysis (computer aided designing)

All the data that was collected had to be processed by entering it into the existing database or saved for later use within the study. This study also helped in the gathering of information for routine use with the section. Daily slope monitoring for example was a routine exercise that this study took advantage of in the field inspections.

Desk analysis also involved the analysis and reviewing of all the information obtained from data collection tools and software. The computer software used to analyse information in this study is described below:

3.2.1 DIPS

DIPS was used to analyse information on discontinuities that were observed and reviewed during cell mapping. Stereographs were generated from the field data which were used to analyse the dips and trends of the discontinuities in comparison to other planes like the slopes to determine potential areas of slope failure.

This software was also used to determine bedding orientations in relation to slope faces. See Appendix 2.

3.2.2 RocLab

RocLab is a software programme for determining rock mass strength parameters, based on the generalized Hoek-Brown failure criterion. The generalized Hoek-Brown failure criterion is expressed as:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (39)$$

Where:

- σ'_1 and σ'_3 = major and minor effective principal stresses at failure
- σ_{ci} = uniaxial compressive strength of the intact rock material
- m_b and s = material constants
- s = 1, for intact rock

$$\text{generally } s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \quad (40)$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right), \text{ and} \quad (41)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right), \quad (42)$$

D is the disturbance factor. It measures the degree of disturbance to which the rock mass has been subjected blast damage or stress relaxation. $D = 1$, for undisturbed in situ rock masses and $D = 0$, for very disturbed rock masses (*after Hoek, et al, 2002*).

Appendix 3 presents the explanation for the disturbance factor D .

Geotechnical information collected from core logging and mapping was analysed with the use of a software package called *RockLab*, for the derivation of shear parameters such as cohesion and frictional angles for the COP FD formations. The shear parameters were to be used in the slope stability analysis and suggestion of the pit designs (benches and face angles)

To derive these shear parameters in *RockLab* the Hoek-Brown failure criterion was used and five factors were analysed for the formations:

(1) The Uniaxial Compressive Strength (UCS)

This is the compressive strength of the intact rock material. From geotechnical information in logging and mapping these values were estimated. In this case the values were compared to the laboratory results of UCS.

Table 3.1 shows the field estimations used in *RockLab* for the analysis of rock mass characteristics in as far as the uniaxial compressive strength is concerned.

Table 3. 1 Analysis of Uniaxial compressive Strength with RocLab

Field Estimate of Strength	Examples	Strength Range [MPa]
Specimen can only be chipped with a geological hammer.	Fresh basalt, gneiss, chert, diabase, granite quartz	> 250
Specimen requires many blows of the geological hammer to fracture it.	Amphibolite, sandstone, basalt, gabbro, granodiorite, gneiss, limestone, marble, rhyolite, tuff	100 – 250
Specimen requires more than one blow of the geological hammer to fracture it.	Limestone, marble, phyllite, sandstone, schist, shale	50 – 100
Cannot be scrapped or peeled with a pocket knife, specimen can be fractures with a single blow from a geological hammer.	Claystone, coal, schist, concrete, shale, siltstone.	25 – 50
Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer.	Chalk, rocksalt, potash	5 – 25
Crumbles under firm blows with point of a geological hammer, can be peeled with a pocket knife	Highly weathered or altered rock	1 – 5
Indented by thumbnail	Stiff fault gauge	0.25 – 1

Source: Hoek-Brown Criterion, 2002

(2)The Geologic Strength Index

This index is determined from the descriptions of the rock mass under investigation. The geologic strength index in this study was determined from field mapping for all

the stacks. For instance a very good rock with good surface conditions gave the rating of this index in the range 78 to 100.

- i. The Material Constants (Mi Values): These are values that are different for different rock materials. Quartzites have mi values of 20 ± 3 with igneous and metamorphic rock materials exhibiting high mi values.
- ii. The Disturbance Factor: This factor indicates the amount of disturbance blasting would cause to surface conditions of the rock materials. It can be applied to tunnels and slopes. For large scale open pits whose slopes suffer from significant disturbance due production blasting this value is designated, 1.00 and for excavation production the value is 0.70.

3.3.3 SLIDE

This software was used in this study to analyse the rock mass parameters determined in RocLab for stability. It is a limit equilibrium based technique for safety factors. In this research, advantage was taken of the relative compatibility of the various software, being programs from the same distributors, Rocscience Inc, to arrive at the design parameters for the Chingola Open Pit F and D. SLIDE analyses are presented in chapter five.

3.3.4 SWEDGE

SWEDGE is a quick and interactive software for investigating stability of surface wedges in a slope. The wedge in this software is defined by two intersecting planes of and the slope surface (e.g. slope face). Another discontinuity would be that of tension cracking. The other features addressed in *SWEDGE* for modelling are water pressure, external and seismic forces and support from shotcreting and active or passive rock bolts.

Analysis with *SWEDGE* can be deterministic (safety factor computation) or probabilistic (probability of failure analysis). For a deterministic analysis SWEDGE computes the safety factor for a wedge of known orientation, while for a probabilistic analysis, statistical input data is entered to account for the uncertainty in joint

orientation and strength values. It results in a safety factor distribution, from which a probability of failure is calculated.

Different scenarios will be presented once the *SWEDGE* programme is:

- A wedge will form only if the two planes of discontinuity are intersecting such that this intersection daylight in a slope surface. This wedge may be stable, symbolised by high safety factor values or unstable depending on the material parameters like cohesion and frictional angle.
- A wedge may not be formed due to either the “non-intersectability” of the planes, or the slope surface.

Table 3.2 shows the computer programs that were used in the study and how useful they are given the differing circumstances and the objectives of the study.

Table 3. 2 Computer programs used in this study and their functions

Software	Data input	Key variables	Expected data output	Comments
<i>Datamine</i>	Bore hole ID (e.g. x,y,z coordinates). Geological data Optional info. (e.g. weathered zones, weak spots, etc)	Survey data (hole identifiers) Geological formations and contact points/depths	Colour coded 3-D models of 'desurveyed' bore holes, digital terrain models (DTMs), zones of influence	Very useful in graphical visualisation of important aspects of the pit during design stages.
<i>DIPS</i>	Discontinuity setting, type, material descriptions (e.g. strength, weathering, etc)	Dip angles Dip direction/ trends Coordinates and major and controlling discontinuities	Stereographs, discontinuity density and their trends dips Comparison of various planes	Is useful in itself for plane failure analysis.
<i>RocLab</i>	Field cell mapping data Observed blasting effects Rock mass characteristics (e.g. field strength values and surface qualities)	Rock strength values Rock quality designational data (e.g. surface quality, cracks and jointing) Petrologic data.	Shear strength data (cohesion, frictional angles, hoek brown failure criteria parameters)	Useful in determination of field estimated design parameters especially in the absence of triaxial test data.
<i>SLIDE</i>	x,y,z coordinated of sectional slices Shear/material properties from <i>RocLab</i> Cracks and ground water models.	Cohesion and frictional angles from <i>RocLab</i> Orientation and dip of tension cracks	Safety factors Indication of slip surfaces	Analysis of failure along a failure plane.
<i>SWEDGE</i>	Orientations of discontinuities from <i>DIPS</i> (mapping) Slope heights and shear parameters from <i>RocLab</i>	Cohesion and frictional angles Dips and trends of joints and slope surfaces	Safety factor and its distribution for deterministic and probability of failure analyses for wedges Orientation of the line of intersection of the planes	Useful for modelling wedges at various locations Useful also in a risk analysis (probabilities)

3.3.5 The Geotechnical Database

From the literature review it was found that the Nchanga Open Pits geotechnical database existed in the form of Excel programmable data. Field investigation and core logging information was entered into this database and either analysed there and then using the structure equations entered into the database or stored for later use. An evaluation of the geotechnical database was conducted and it was replaced by a corrected database that was accessible to all parties involved in planning, geology and surveying.

The geotechnical database was the store house of all the geotechnical data relevant for the monitoring of slopes and other rock mass characteristics.

3.3.6 3-D Visualisation Models

Visualisation models were developed from the data stored in the geotechnical database. The visualisation models showed important parameters for easy understanding by the various sections accessing the geotechnical data (e.g. planning section).

Datamine™, a special software developed for mining and geological graphic design systems, was used to develop these models. Figure 3.6 shows a model created using field data for this study with the help of Datamine™.

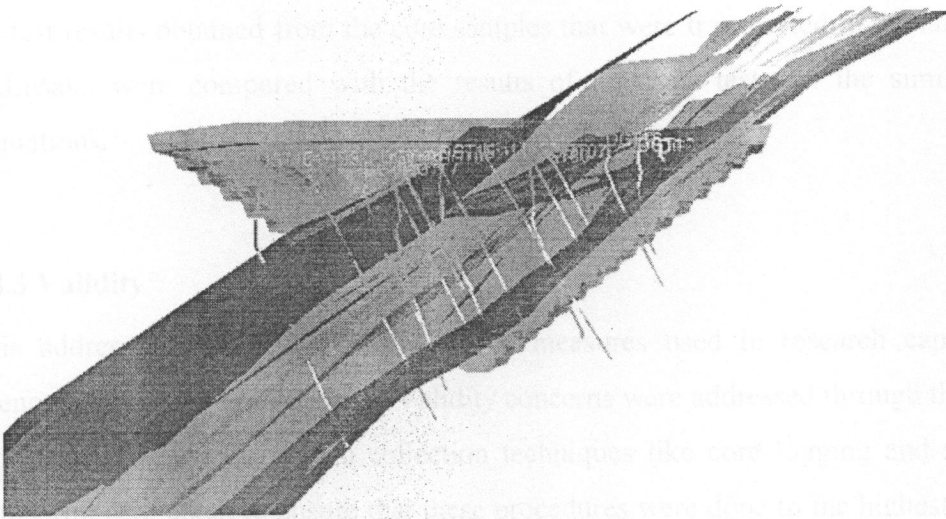


Figure 3. 3 Optimised Pit boundaries for COP F/D Phase III, (Nchanga, 2006)

These visual models helped in the characterisation of areas of important geotechnical structures. Areas of similar parameter values were joined and a set trend was identified.

3.4 Representivity, Reliability and Validity of Data

3.4.1 Representivity

Representivity describes the level of data-representation across the study area. Data was collected on a total number of 23 geotechnical bore holes distributed over an 800 by 700 square metre area. The holes had been drilled as infill holes for the geologic

sectional holes that were used for exploration and as such they traced intermediary sections. In the study about 90% of sectional boundaries were covered through mapping, logging and field visits.

3.4.2 Reliability

This is the consistency of results across different measurements. This study depended much on engineering judgement for geotechnical logging and mapping as well field observations of slopes. Reliability errors could have arisen through systematic errors in field estimations of rock characteristics. For instance, the measuring of RQD lengths, intact rock strength estimations, determination of joint conditions, etc; all these were factors dependent on the engineer's perceptions. In addition, drill cores were collected for testing and transported over a long distance (over 500 km from the site to the labs). This could have affected the core through micro-cracking. Therefore, the test results obtained from the core samples that were transported to the laboratory in Lusaka were compared with the results of previous tests on the similar rock formations.

3.4.3 Validity

This addresses the adequacy with which measures used in research capture the phenomenon under investigation. Validity concerns were addressed through the use of time tested, standardised data collection techniques like core logging and mapping with extra care taken to ensure that these procedures were done to the highest level of standards. The samples were prepared to standard e.g. Length- Diameter ratios and their correction factors.

CHAPTER FOUR

RESULTS, ANALYSIS AND DISCUSSION

This chapter is dedicated to the presentation of the results, their analysis and discussion of the findings. The presentation will be done based on objectives. The results, data analysis and discussion for objective 1 will be presented in the first section, followed by other objectives and subsequent sections. This approach has been adopted to aid the reader in following the findings and discussion of one objective without disruptions. A brief summary of the major findings are then presented and the chapter ends with a short reflection on the data analysis tools for the study.

4.1 Slope Design Parameters (cohesion and internal friction angles)

The results of the study on slope design parameters for COP FD Phase III are presented in this section. These results were determined from various stages of data collection and are analysed concurrently.

Methods of data collection such as drill core logging and geotechnical mapping were given special attention because they have a great bearing on the quality of designs that they will give, which stems from the quality of data collection.

4.1.1 Geotechnical Core Logging

Detailed core logging of geotechnical drill holes was carried out to determine rock mass characteristics which were critical factors for the geotechnical zones of the study site. The Nchanga Open Pits uses the rock mass classification system devised by Laubscher (1990) while the Bieniawski classification is used as a reference system to check the accuracy of the Laubscher classification. The difference between these two classifications is that the Laubscher classification includes adjustments for weathering of the rock material, joint orientations in relation to the excavation, mining-induced stresses and blasting effects. The nature of core logging is somewhat dependent on the logger. Changing hands introduces errors in the values as some of the information obtained during logging is based on estimates and engineering judgments, for

instance, the calculation of RQD requires the measurements of the recovered core length when core logging, in addition to measuring the pieces of recovered core that are 100 mm and above. However, a problem arises in defining these pieces. It is sometimes argued that RQD is supposed to be calculated on the basis of breakage either natural or drill break. However in this study RQD was calculated based on the pieces that were separated by natural breaks as opposed to drill breaks. This is in line with the suggestion presented by Palmstrom, (2005), that RQD gives an average measurement of the degree of jointing along the actual section (core run).

Table 4.1 shows the rock mass ratings that were determined from drill core logging. These values were determined by embedding graphical relationships for various inputs to RMR in Microsoft Excel for calculation (Appendix 4) It can be seen from the table that the Laubscher rating is conservative (lower values) as compared to the Bieniawski rating. This is due to the adjustments that have been included in the Laubscher rating for mining.

Table 4. 1 RMR for Bieniawski and Laubscher as determined from geotechnical core logging. Nchanga, 2006

Rock Formation	In-situ Density [MN/m ³]	Rock Mass Rating		Classification		RQD [%]
		Bieniawski	Laubscher	Bieniawski RMR Class	Laubscher RMR Class	
URD	0.019	30	28	4	4	13.3
SWG	0.028	41	33	3	4	33.9
CDOL	0.028	35	30	4	4	21.0
DOLSCH	0.028	42	37	3	4	26.1
UBS	0.027	34	29	4	4	12.1
TFQ	0.025	46	42	3	3	32.1
TFQT	0.025	38	35	4	4	15.5
BSSU	0.023	37	35	4	4	23.4
PQ/SM	0.023	45	40	3	4	25.7
BSSL	0.023	33	29	4	4	13.8
LBS	0.026	33	31	4	4	13.8
TR	0.027	46	45	3	3	41.1
ARK	0.027	57	52	3	3	58.8
BAS	0.027	57	53	3	3	51.9

(Field Data, 2007)

4.1.2 Field Mapping.

Cell mapping was carried out at COP F/D phase II, where in-depth information on the rock units was obtained. Face mapping was carried out on the western wall based on benches in Phase II adjacent to Phase III. As in core logging, mapping was a source of design information used for rock mass characterization and geotechnical zones identification.

4.1.2.1 Face Mapping of 105 meter Bench West Wall

Face mapping was carried out in benches starting from the newly mined 105mB. This bench was found to be characterised by The Feldspathic Quartzites (TFQ) to some extent and Arkose (ARK) to Lower Banded Shales (LBS) to a larger extent. The mineralisation at this depth is of high grade copper. The rock mass in this locality is leached and highly weathered due to a shallow water table. Table 4.2 shows the face mapping information collected at this bench.

Table 4. 2 The 105 metre-bench mapping results. COP FD Phase II, 2007

Discontinuity Type	Dip	Dip Direction	Comments
JS ₁	76	147	HW, hard, Leached, moist, smooth and stepped joints. Highly Broken, dark grey, micaceous, flaky.
JS ₂	74	60	
B	55	260	

(Field Data, 2007)

Fig 4.1 indicates areas where the information in the above table was picked from which stereo plots were constructed.

Figure 4.1 shows at least two joint sets while Figure 4.2 shows the possibility of wage failure and indicating the presence of at least two joint sets. The bedding here dips into the face thereby improving stability.

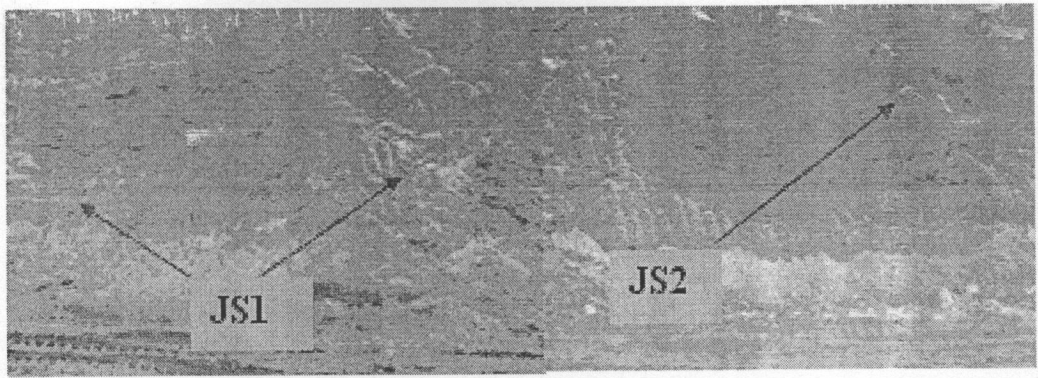
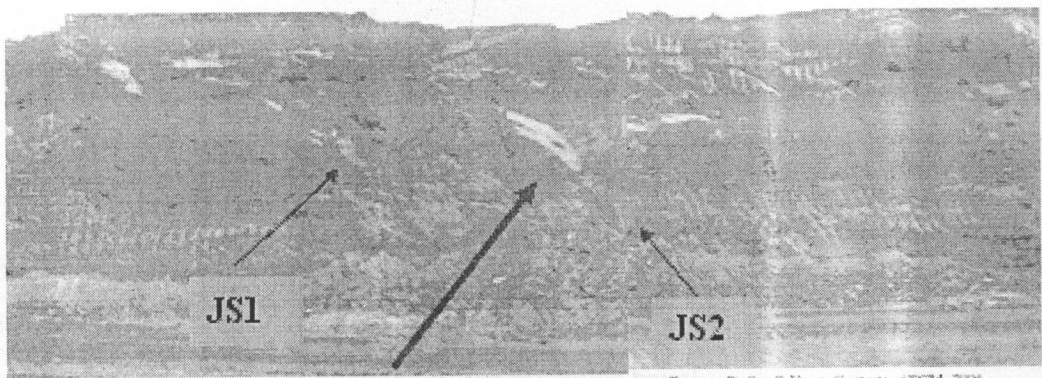


Figure 4. 1 COP F/D face mapping.



Wedge Failure Planes.

Figure 4. 2 the 105 meter-bench pictorial view of the west wall of the mapped slopeface (Source: Field Data, 2006)

Fig. 4.3 is a stereographic projection showing both bedding and joint orientations. The planes presented are the slope face, bedding and joint planes. For the 105mB, the slope ace designed was to plunge at 70 degree eastwards (i.e. 70 deg dip, 90 deg dip direction).

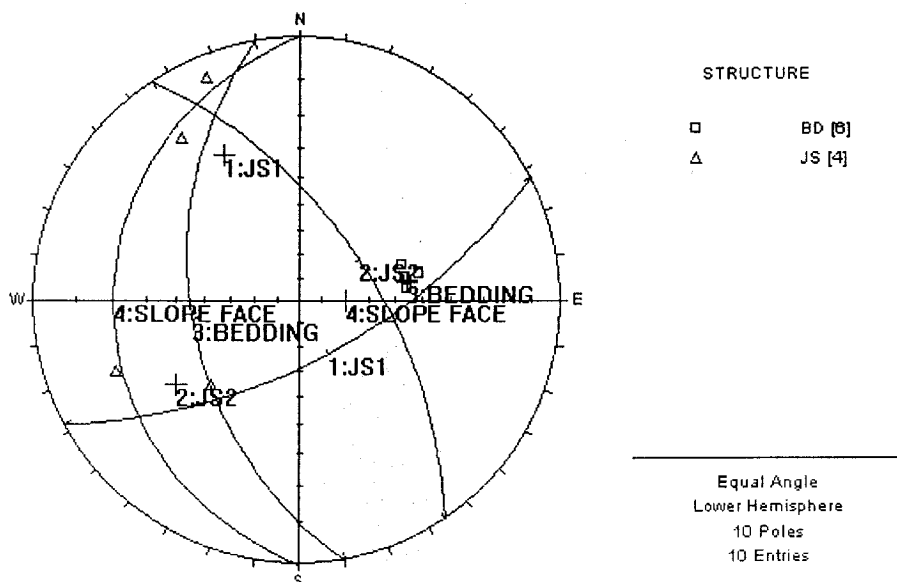


Figure 4. 3 Stereoplots of the three planes at 105 meter-bench. (Source: Field data, 2006)

4.1.2.2 90 meter Bench Mapping

The 90 meter bench presented a challenge for mapping and taking photographs because access areas had been mined out before mapping. However, Table 4.3 presents information from mapping of the 90 meter-bench

Table 4. 3 The 90 metre-bench cell mapping, COP FD Phase II, 2007

Discontinuity Type	Dip	Dip Direction	Comments
JS ₁	60	70	Highly persistent jointing. hard in-filling in some places. highly weathered generally. Groundwater conditions determined from an independent study of the hydrology of the area.
JS ₂	78	127	
JS ₃	88	175	
B	53	258	

(Field Data, 2007)

The 90mB had various geotechnical structures. There were several planes of discontinuities oriented in different directions. However, the overall conditions of the rock mass (generally weak and highly weathered, leached and sheared) gave it a different rock mass behaviour in failure propagation which was inclined towards planar or semi-circular failure.

The 90mB showed a marked presence of The Feldspathic Quartzites (TFQ) as shown in Figure 4.4; the strength of which varied at different locations.

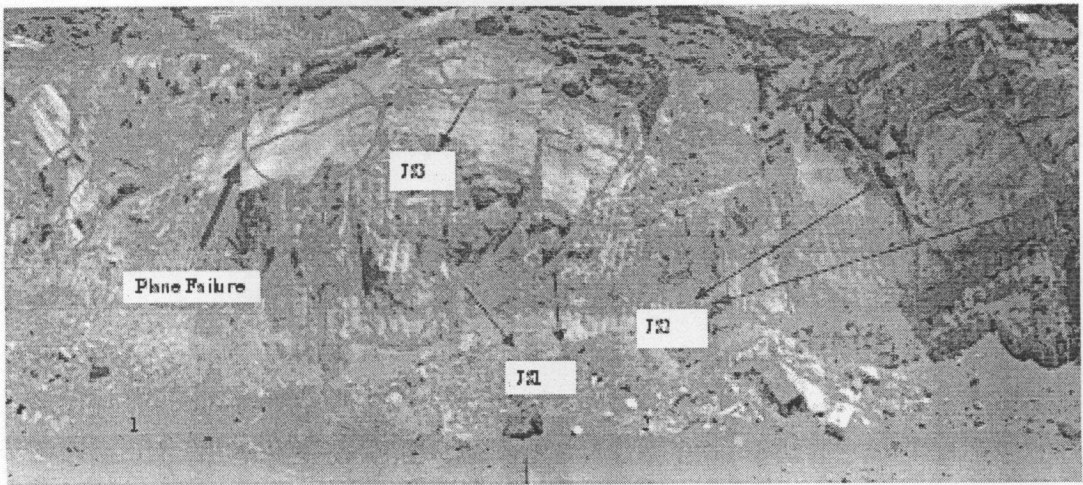


Figure 4. 4 The 90 meter-bench west wall showing major joint sets and failure planes

Fig. 4.5 shows the stereographic projections for the planes of discontinuities discussed above. With the design face angle of seventy degrees (70°) in the eastern general direction, the bedding and jointing give the idea of the interaction they have with the slope face.

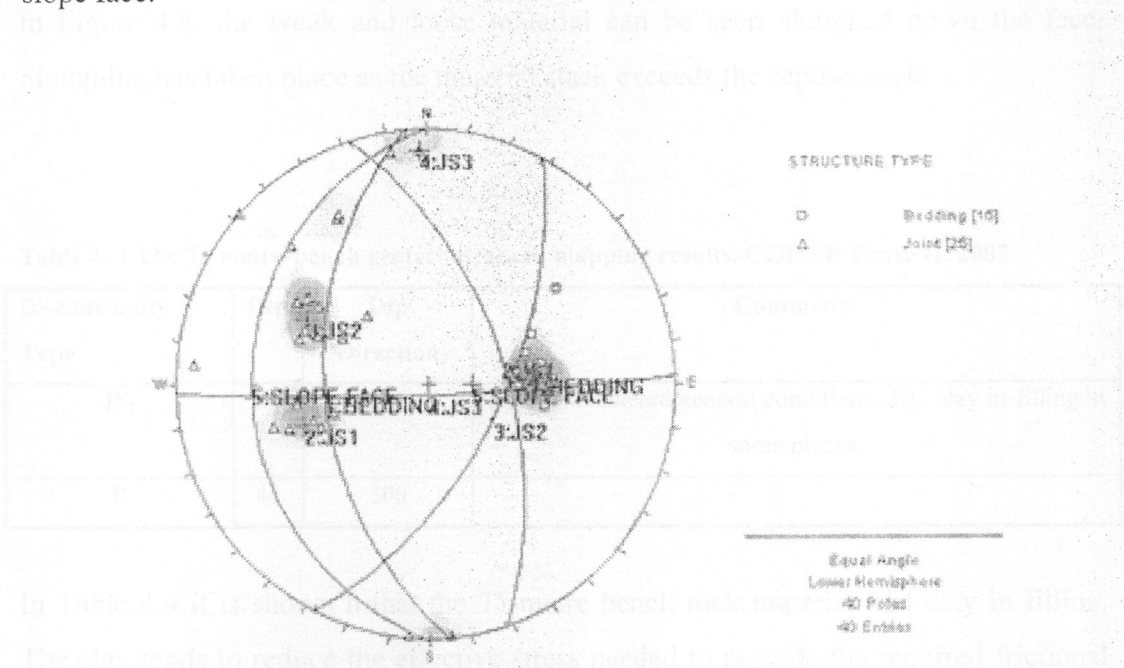


Figure 4. 5 90 meter-bench stereo projection. Nchanga, 2006.

4.1.2.3 75 meter Bench Mapping

The 75mB slope faces were also mapped with the same techniques (cell mapping). Stratigraphically this level is nearer to the surface. It shows clear orientations with an overturned folding system. The geologic material is predominantly soft rock and soil mass. This face is characterised by weathering due to atmospheric conditions and it is highly broken due to shearing.

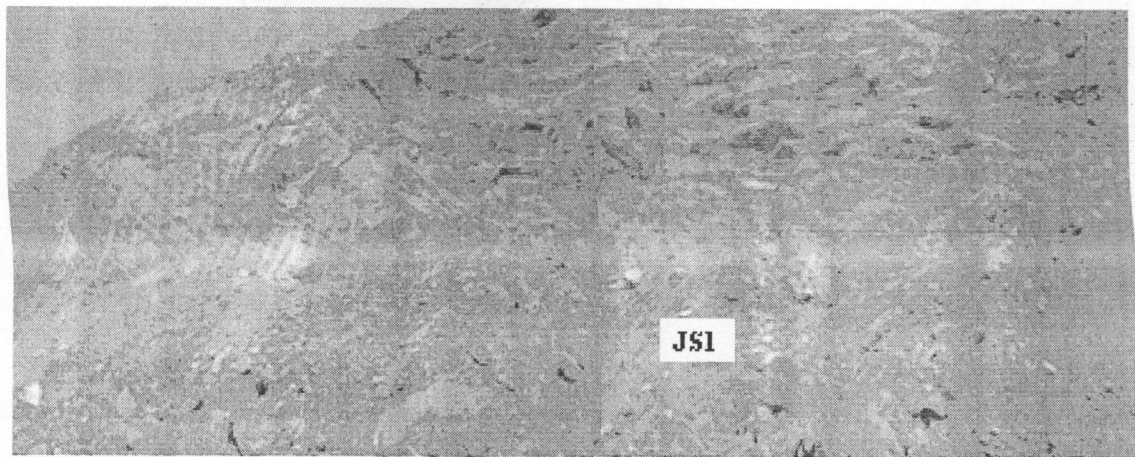


Figure 4. 6 The 75 meter-bench wall face

In Figure 4.6, the weak and loose material can be seen sloughed down the face. Sloughing has taken place as the material stack exceeds the repose angle.

Table 4. 4 The 75 metre-bench geotechnical cell mapping results. COP FD Phase II, 2007

Discontinuity Type	Dip	Dip Direction	Comments
JS ₁	64	165	Highly weathered general conditions, dry, clay in-filling in some places.
B	44	300	

In Table 4.4 it is shown h that the 75 metre bench rock material had clay in filling. The clay tends to reduce the effective stress needed to provide the required frictional forces to prevent sliding of the rock strata. Figure 4.7 shows a kinematic diagram of these discontinuities.

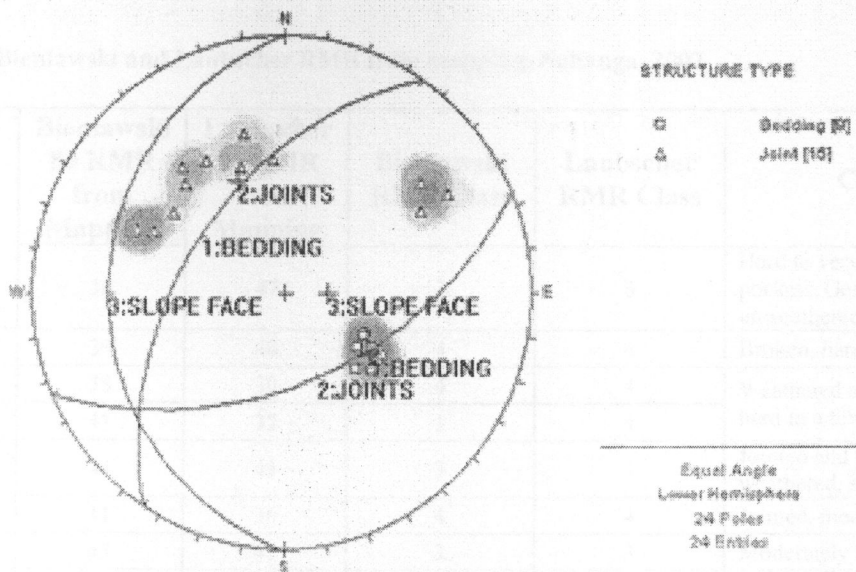


Figure 4. 7 The 75 meter-bench streoplot

In Table 4.5, summary data of rock mass classification for both Bieniawski and Laubscher are presented. As in core logging, these values were calculated from various geotechnical inputs from the field and embedded in Microsoft Excel by using graphical relationships for RQD, UCS, etc in the calculation of RMR.

This classification compares very well with Table 4.1 whose rock mass classification values are from core logging. The reason for comparisons is to ensure similar information as most of the drill holes logged were from the same area where mapping took place.

Table 4. 5 Bieniawski and Laubscher RMR from mapping. Nchanga, 2007

Rock Formation	Bieniawski 89 RMR from Mapping	Laubscher 90 RMR from Mapping	Bieniawski RMR Class	Laubscher RMR Class	Comments
ARK	50	47	3	3	Hard to very hard rock. Soft pockets. Generally competent and unweathered.
TR	39	40	4	4	Broken, hard and soft pockets
BSSL	38	30	4	4	Weathered and soft. Hard to very hard in a few places
BSSU	41	32	3	4	
TFQ	44	43	3	3	Jointed and slightly to moderately weathered, soft pockets present
UBS	31	36	4	4	Jointed, moderately weathered
DOLSCH	43	41	3	3	Moderately to highly weathered
CDOL	32	32	4	4	Highly weathered
SWG/URD/OV	-	-	-	-	Very highly weathered and ground material

(Field data, 2007)

The mapping and core logging data collected helped in the formation of a design model for COP FD Phase III. It must be noted here that the footwall slopes for Phase III will mostly be the same as the previous phases but those for the hanging wall will change in this Phase with the changing depth of lithologies.

4.1.3 Comparisons of COP FD rock classifications based on RMR₈₉ and RMR₉₀ from core logging and cell mapping.

Agreements between core logging and cell mapping data was analysed statistically for the sole purpose of comparing the two data sets. In Table 4.6, all parameters are arranged (Bieniawski vs. Laubscher) for both mapping and logging.

Table 4. 6 RMR Comparison between core logging and cell mapping procedures

Rock Formation	Rock Mass Rating (RMR)				Class	
	Bieniawski Rating		Laubscher Rating			
	Core Logging	Cell Mapping	Core Logging	Cell Mapping	Core Logging	Cell Mapping
ARK	57	50	52	47	3	3
TR	46	39	45	40	3	4
BSSL	33	38	29	30	4	4
BSSU	37	41	35	32	4	4
TFQ	46	44	42	43	3	3
UBS	34	31	29	36	4	4
DOLSCH	42	43	37	41	3.5	3
CDOL	35	32	30	32	4	4
SWG/URD/OV		-		-		-

(Field data, 2007)

Variations found in the RMR values in mapping and core logging represent particular spots of interest. Some of the reasons for these differences may be as follows:

1. Soft pockets of materials which characterised mapping, due to the fact the core logging data represented geotechnical information obtained from a narrower perspective of the drill hole trace.
2. Mapping has a nature of large scale parameter determination in situ. This tended to spread the numerical values over a wide spectrum around the pit.
3. The drill core was un-oriented. The information on joint orientation is ignored in this type of core and no good perspective can be analysed to determine the effects of the joint system on the slope surfaces in a broader sense. However, care was taken to record possible joint sets by inspecting joint characteristics like infilling, roughness, etc.

Two-sample t-tests were run on the rock mass ratings for Bieniawski and Laubscher systems: (Table 4.6) and it was found that there was no significant difference in the core logging values ($t_{\text{calc}, df=14} = 0.933$, $p = 0.3665$). In addition, the mapping values also showed high agreement ($t_{\text{calc}, df=14} = 0.69$, $p = 0.502$). The p-values obtained in these tests, with the level of confidence of 95%, they are acceptable and reveal that there are no significant differences in the data obtained from logging and that from mapping (Appendix 5). In fact, the two methods of soil investigation affirm the accuracy employed during the whole procedure of data collection.

4.2 Groundwater

Groundwater accounts for a very influential role in the stability of a slope. The Chingola Open Pits F and D have been characterized by a high water table as shown by piezometer values in Appendix 6. Many such values were interpreted and graphical representations as shown in Figure 4.8 were modeled.

Figure 4.9 shows a model of the COP FD with drill holes for dewatering. A program was started for dewatering in this area (Phase II and III) as can be seen by the holes outside the pit in the model.

Flowrates were measured for the determination of water being pumped out of the pit through the pipes in Figure 4.10. selected results for this rate is shown below.

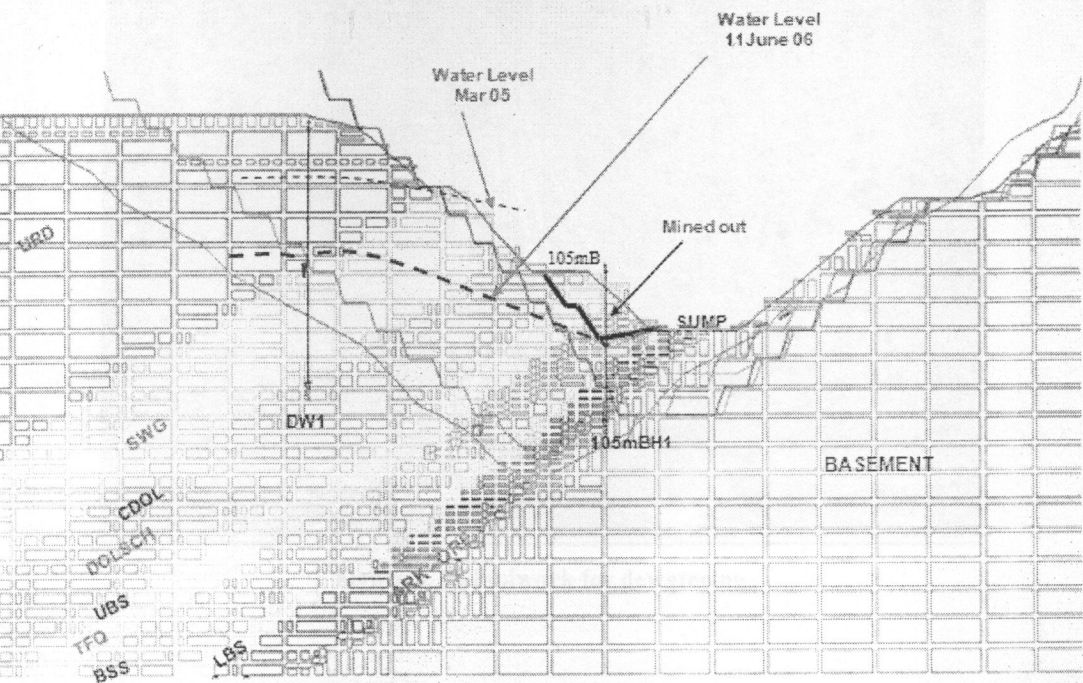


Figure 4. 8 Graphica representation of the interaction between rock strata and watere levels

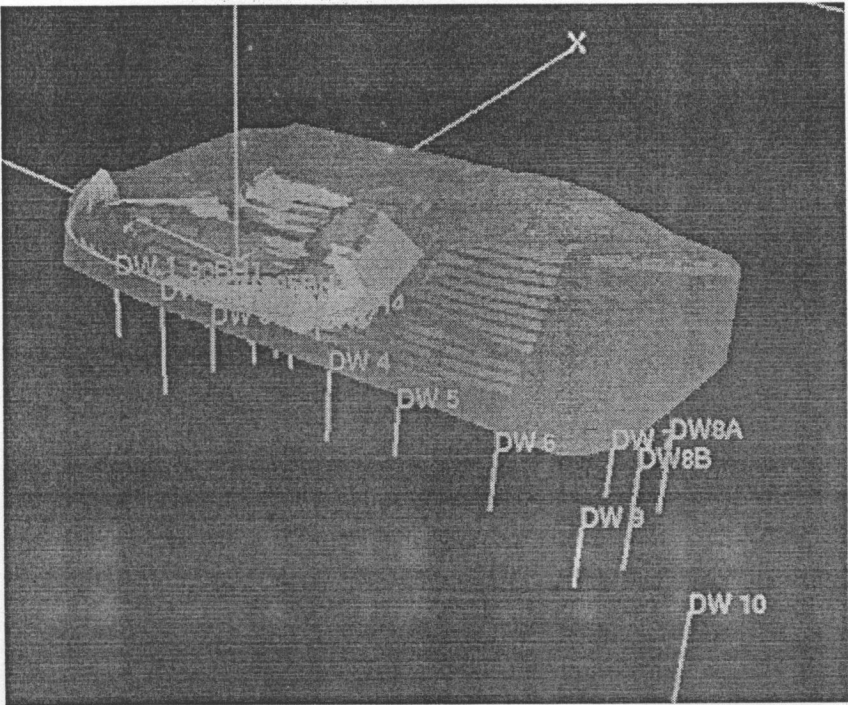


Figure 4. 9 Solid model of pit with dewatering holes

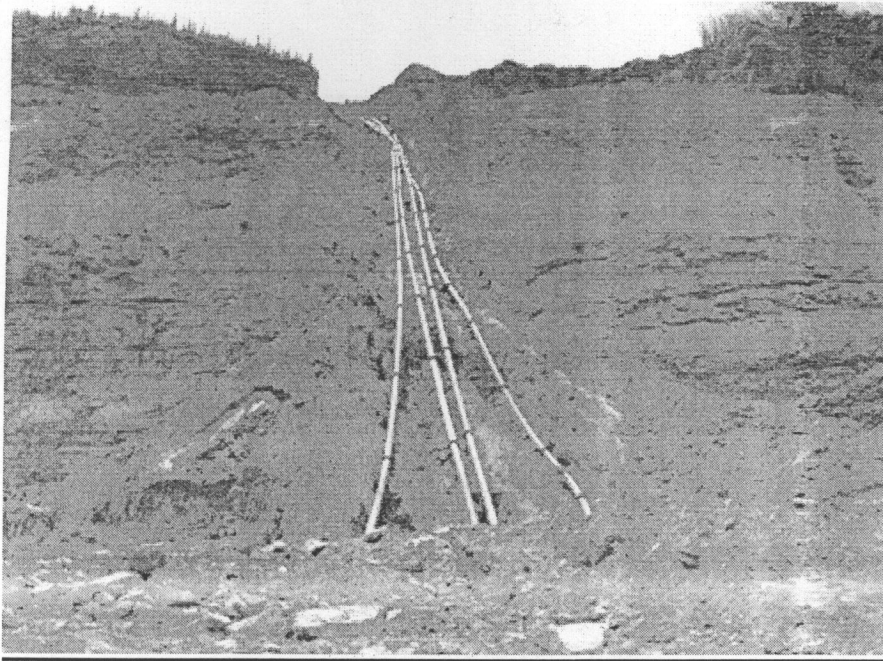


Figure 4. 10 Piplines for dewatering

4.2.1 COP F/D Dewatering Boreholes

Flow rates as at 11 / 05 / 05

(1) DW 1

Results of the water measurements taken on DW1 using a 210 L Container:

Time 1	13.93 seconds
Time 2	12.86 seconds
Time 3	11.93 seconds

Average time was time at **12.91** seconds

$$\begin{aligned}
 \text{Hence flow rate} &= 210 / 12.91 \\
 &= 16.27 \text{ l/s} \\
 &= 1,405 \text{ cu.m / day}
 \end{aligned}$$

(2) **DW 2**

Results of the water measurements taken on DW 2 using a 210 L Container:

Time 1	43.46 seconds
Time 2	51.49 seconds
Time 3	50.20 seconds

Average time was time at **48.40** seconds

Hence flow rate = 210 / 48.40

 = **4.33 l/s**

 = **375 cu.m / day**

(3) **DW 3**

Results of the water measurements taken on DW 2 using a 210 L Container:

Time 1	32.09 seconds
Time 2	29.51 seconds
Time 3	30.55 seconds

Average time was time at **30.72** seconds

Hence flow rate = 210 / 30.72

 = **6.83 l/s**

 = **590 cu.m / day**

(4) **DW 4**

Results of the water measurements taken on DW 2 using a 210 L Container:

Time 1	22.72 seconds
Time 2	21.78 seconds
Time 3	21.76 seconds

Average time was time at **22.10** seconds

$$\begin{aligned}
 \text{Hence flow rate} &= 210 / 22.10 \\
 &= \mathbf{9.50 \text{ l/s}} \\
 &= \mathbf{821 \text{ cu.m / day}}
 \end{aligned}$$

4.3 COP F/D Rock Characterisation

In the study area rock masses were characterized on the basis of rock mass ratings and their general behaviour in situ. The rock mass in this area was classified as *POOR TO FAIR* (Table 4.7). Most of the rock units were found to be highly leached, weathered and decomposed. In a study carried out in Phase II on the water regime of COP FD, Nkhuwa et al. (2004) suggested that the water table in this area was very high. This water is believed to be responsible for the weathering and decomposition of most the rock units in Phase III.

Three geotechnical zones called stacks were arrived at based on all the factors that were used in data collection. These geotechnical zones are presented below and they form a framework for the calculation of design parameters.

Table 4. 7 COP FD Rock characterisation. Nchanga, 2007

Rock Formation	Description	Remarks
ARK	Fair Rock	Whitish grey, fine grained, hard and competent. slightly weathered, talcose in quartzite matrix, pitted in some places due to leaching
TR	Fair Rock	Grey medium grained, slightly hard, sheared
BSSL	Poor Rock	Reddish brown, medium to coarse grained, very weak. micaceous
BSSU	Poor Rock	Light brown, fine grained, weak to very weak. pitted. silicified
TFQ	Fair Rock	Whitish to pinkish-brown, medium grained, weak to moderately hard to hard, feldspathic quartzite matrix. sporadic mineralization, slightly weathered and highly weathered in places.
UBS	Poor Rock	Grey fine grained, highly jointed, very low RQD, weak to moderately hard, weathered
DOLSCH	Fair Rock	Grey to brown, medium grained, highly weathered, highly jointed, silicified, weak to moderately hard.
CDOL	Poor Rock	Brown medium grained, weak to very weak, sheared and silicified.
SWG/URD/OV		Generally grey to brown. Very weak, remoldable.

(Field Data, 2007)

The three stacks that were identified are:

- ***Upper Hangingwall Stack:***

Comprised of *URD, SWG, CDOL, DOLSCH* and *UBS*.[♦] They have rock mass ratings ranging from 25 to 40; and height ranging between 60 and 200 metres. This is due to the thickness of the URD and SWG.

- ***Lower Hangingwall Stack:***

It consists of *TFQ, TFQT, BSSU, BSSL, PQ/SM* and *LBS* with rock mass ratings ranging from 30 to 45, and height ranging between 30 and 70 m.

- ***Footwall Stack:***

Consists of *ARK* and *BAS* with RMR over 50 and height of more than 40 and 80 m.

The stacks grouped above represent rock formations that have similar behaviour under specific stress-strain conditions. A major parameter of classification of these formations into stacks is the rock mass rating (RMR_{90})⁵. In Table 4.8 design parameters averaged for each stack have been shown. These parameters have been determined for the rock masses and soils in which these slopes are expected to be mined.

Dealing with naturally occurring rocks needs an understanding of the environmental factors that affect the rock. Most notable of these factors would be seasonal rainfall, atmospheric conditions that enhance weathering to a lesser degree and ground water. In this project, field observations have enabled engineering judgements to be used extensively.

[♦] Abbreviations of these rock formations have been presented above.

⁵ After Laubscher, 1990.

Table 4. 8 Design parameters for individual stacks

Stack	Rock Formation	In-situ Density [MN/m ³]	RMR Bieniawski	RMR Laubscher	RQD [%]	UCS from Logging [MPa]	UCS from Mapping [MPa]	Cohesion [Mpa]	Frictional Angle [Deg]
Upper Hangingwall Stack	URD	0.019	30	28	13.3	0-1	0-5		
	SWG	0.028	41	33	33.9	1-20	0-20		
	CDOL	0.028	35	30	21	10-50	10-50		
	DOLSCH	0.028	42	37	26.1	0-30	0-40		
		0.02575	37	32	23.575	3-50	2-60	0.226	32.46
Lower Hangingwall Stack	60 – 200								
	UBS	0.027	34	29	12.1	5-60	10-70		
	TFQ	0.025	46	42	32.1	20-100	40-100		
	TFQT	0.025	38	35	15.5	5-20	5-10		
	BSSU	0.023	37	35	23.4	10-70	5-20		
	PQ/SM	0.023	45	40	25.7	10-60	5-50		
	BSSL	0.023	33	29	13.8	0-20	0-10		
	LBS	0.026	33	31	13.8	5-100	1-60		
	TR	0.027	46	45	41.1	10-80	5-50		
		0.024875	39	35.75	22.1875	0-80	0-75	0.290	40.01
Stack Average Stack Height [m]	30 – 70								
Footwall Stack	ARK	0.027	57	52	58.8	30-150	50-100		
	BAS	0.027	57	53	51.9	80-200	60-120		
Stack Average Stack Height [m]	40 – 80								
		0.027	57	52.5	55.35	30-200	40-120	0.416	41.91

Laboratory values of UCS are incorporated. (Field Data, 2007)

CHAPTER FIVE DISCUSSION OF RESULTS

The design parameters (cohesion and internal friction angles) presented in chapter four were determined using *RocLab*, and as such tests were done to determine the stability offered by the angles. These angles must agree with the general conditions for slope failure presented in Chapter 2. A testing criteria used in this study was that of using *SLIDE*, a slope stability analysis software used in the calculation of safety factors. In all calculations for the safety factors, dry slope conditions were assumed for ease of calculations.

5.1 Safety Factor for the Upper Hanging Wall Stack

Comprising mostly of soft to moderately hard material, this stack causes the most worry in as far slope stability is concerned due to its high porosity, high content of clayey minerals and its susceptibility to seasonal rainfall the combination of which affects the effective stress values through pore water pressure.

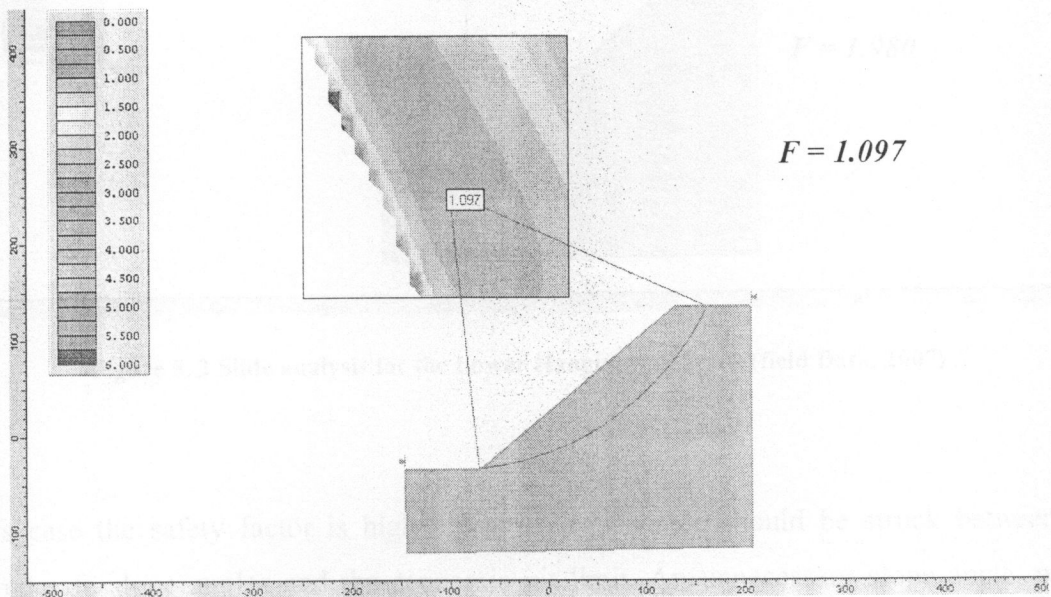


Figure 5. 1 Slide Analysis of the Upper Hangingwall Stack

The factor of safety F in the stack shown in Figure 5.1 is 1.097. This factor of safety indicates the stability of this material at the friction calculated from *RockLab*. It is lower than the require 1.2. This means that, on selected sections of the pit, the slope angle must be

maintained 33 degrees. Using iterative runs of slope testing a range of slope angles would be arrived at for this part of the pit.

5.2 Safety Factor for the Lower Hanging Wall Stack

The lower hangingwall stack exhibits trends of strength values more than those in the upper hangingwall stack. Figure 5.2 shows the factor of safety for this stack from Bishop's simplified method.

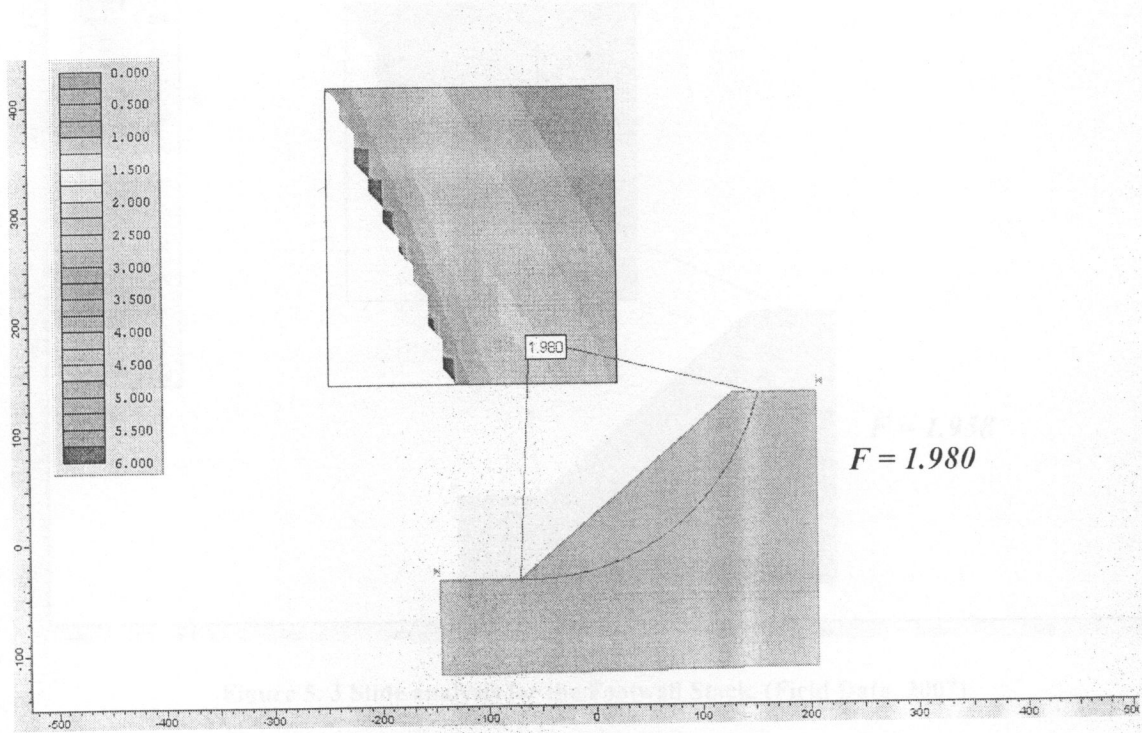


Figure 5. 2 Slide analysis for the Lower Hangingwall Stack (field Data, 2007)

In this case the safety factor is higher than 1.2, a balance should be struck between the required safe slope angles and the economic pit limit. An unnecessary slope angle, which would produce a more than needed safety slope, will impact negatively on stripping ratios. Also iterative entrances into this model require arriving at the right slope angle. Slope angles of less than 40 degrees can be used depending of designs for particular sections.

5.3 Safety Factors for the Footwall Stack

This stack is comprised of Arkose and Basement Schist (BAS). The thickness value of this stack is far bigger than the rest of the stacks in this analysis because of the Nchanga Granites that underlay the arkoses. The material in this stack is hard to very hard rock, weathered in some places due to leaching by ground water, sheared and jointed in other places. Figure 5.3 shows the safety factor from *Slide* for this stack.

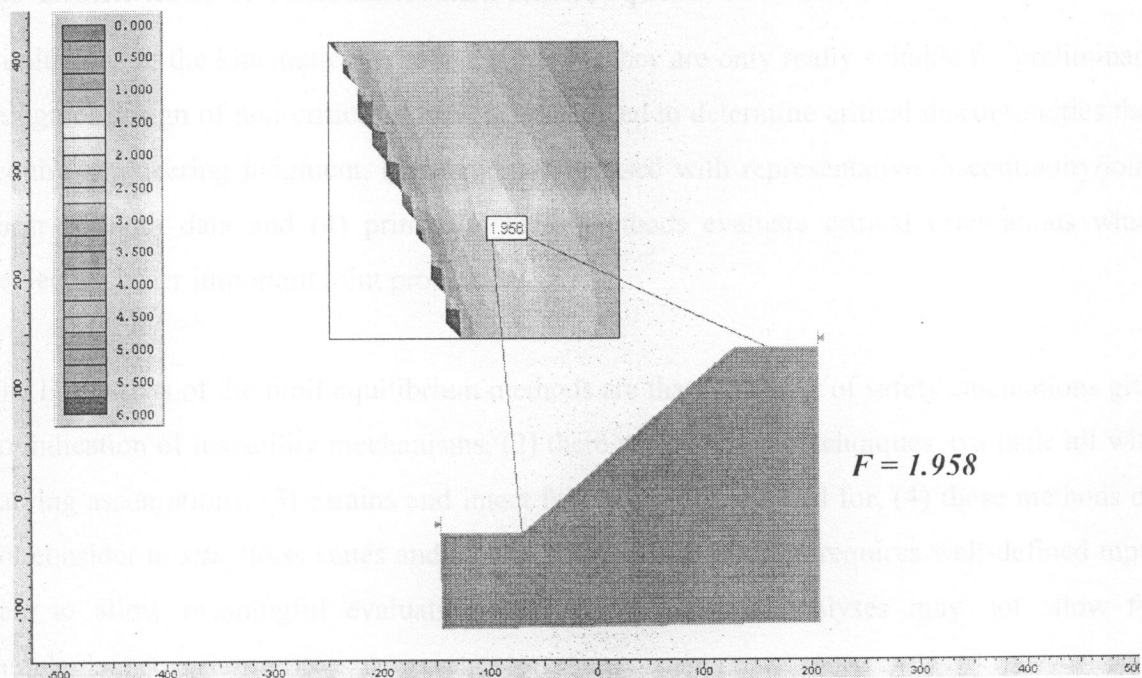


Figure 5. 3 Slide analysis for the Footwall Stack. (Field Data, 2007)

5.4 Analysis with SWEDGE

Investigations for possible wedges and their subsequent failure were carried out using SWEDGE. These revealed that there were no possible wedges to be formed on the upper formations (in the areas where cell mapping was conducted). These were the 75 and 90 metre-benches. However a wedge was projected when the 105 metre-bench was analysed and this wedge gave a combined safety factor of approximately 290.

This means that even though the wedge was formed, there was a very small chance of this wedge failing. This is attributed to the orientations of the planes of discontinuities and the material strength parameters. See Appendix 7.

All the analysis and determination of results were conducted either using the kinematic or the limit equilibrium analysis methods. Kinematic analyses were used when analysing the discontinuities using the *DIPS* program and limit equilibrium methods were used in *SLIDE* and *SWEDGE* analysis for the calculation of safety factors.

5.5 Limitations of Kinematic and Limit equilibrium methods

Limitations of the kinematic methods are that (1) they are only really suitable for preliminary design or design of non-critical slopes, (2) they need to determine critical discontinuities that require engineering judgment, (3) they must be used with representative discontinuity/joint shear strength data and (4) primarily, these methods evaluate critical orientations while neglecting other important joint properties.

The limitations of the limit equilibrium methods are that (1) factor of safety calculations give no indication of instability mechanisms, (2) there are numerous techniques available all with varying assumptions, (3) Strains and intact failure are not allowed for, (4) these methods do not consider *in situ* stress states and (5) the probabilistic analysis requires well-defined input data to allow meaningful evaluation. Simple probabilistic analyses may not allow for sample/data covariance (Dolezalova et al, 2006).

5.6 Limitations in Data Collection

Limitations to data collection came in different forms. The most notable areas which posed challenges in data collection were logging, mapping and shear strength laboratory testing equipment. In core logging, for instance, some of the drill cores were drilled earlier than the time this study was conducted. Some sections of core could not be accounted for, or had been totally taken for sampling by the Mineral Reserves section of geology department for assaying. Some of the core had weathered further because of prolonged exposure to the atmospheric conditions at the core shed.

In mapping: some areas could not be accessed. Mapping, in this study, required physical presence at the face for a close up observation of the rock mass.

A major limitation in this study was the non availability of the shear strength testing equipment for triaxial (direct shear) tests, in this case, the Hoek Cell.⁶

5.7 Mitigations for the above limitations

In core logging careful analysis and replacement of core was done to reduce the information gaps in the data. This was done by replacing the entire drill hole by one that did not miss sections, but within the same drilling programme. Core logging information was also obtained from the mineral reserves section to fill in the gaps.

Photographic analysis was used to mitigate for the inaccessible areas in mapping. Pictures were also taken for accessible areas for the large scale analysis of discontinuities and for information storage. Face mapping was done strategically and information captured before the areas were mines out.

The direct shear values were not used in the study. An approach which used the software *RockLab* was used. This was used in combination with *Slide*⁷ for the calculation of safety factors and for the running of the model in simulation analyses. *RockLab*, based on the Hoek-Brown failure criterion analysis was used to determine the shear parameters for the various rock formations in the study area.

The main methods for data collection were core logging and field mapping. These estimations were also used to determine rock mass classifications for the various rock units.

5.8 Costs associated with the change to the new slope angles

The design parameters so determined will affect the economic pit limits of phase three in comparison to phases II and I. Changes in slope angle have an effect on the cost aspect of mining. At the Chingola Open Pit (Phase II) overall slope angles used for the hangingwall and footwall are 38 – 39° and 41° respectively. In this research however, the hangingwall angles, given in stacks, have been given as 30 – 35° for the upper hangingwall stack, 37 – 42°

⁶ At the time of writing this thesis, the researcher had information that the study site proprietors (KCM) have acquired a tri-axial testing kit for direct shear tests (the Hoek Cell).

⁷ *RockLab* and *Slide* are Limit equilibrium analysis software for slope stability analysis.

for the lower hangingwall stack and 39 – 45° for the footwall stack. The upper hangingwall stack is considered for the initial cost analysis as presented in figure 5.4.

With the current cost of mining material pegged at **US\$ 0.77/t⁸**, an incremental cut of material mined will be considered.

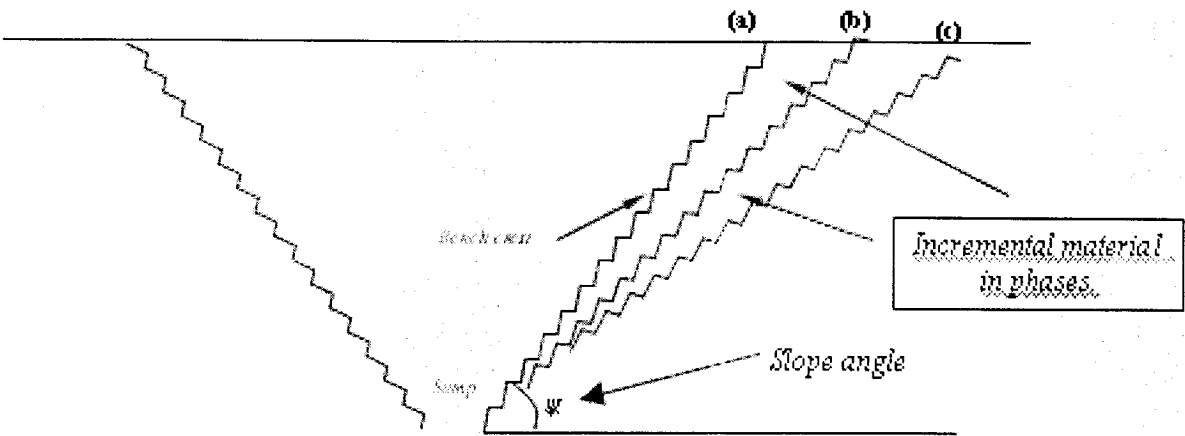


Figure 5. 4 Open Pit slope angles (a, b and c) showing different slopes in the hangingwall, (Nchanga 2007)

For the upper hangingwall incremental mining the following assumptions are made for a cut into the next phase, whose initial phase II slope angles are 38°:

Cut:		
Width of cut	=	60 m
Length of cut	=	100 m
Angle of cut	=	30-35°, 37-42°, 39-45°

By using basic *trigonometry* on the slopes a, b and c; cost values may be arrived at for mining material in hypothetical cut. Table 5.9 summarises the costs each stack angle (max and min slope angles) will produce as compared to an initial stage where similar angles from phase II are used.

⁸ Cost per tonne by the mid 2007 at Konkola Copper Mines PLC .

The upper hangingwall stack minimum angle of 30° increased the cost of mining in the adjacent phase III by 35%, while the maximum angle of 35 increased this cost by 11%. In the lower hangingwall stack the minimum stack angle of 37° increased the cost by less than 0.4% while the maximum angle of 42 reduced these costs by 1%. Lastly in the footwall an increase in the cost was by 3% against a reduction of 5% by the minimum and maximum angles of 39° and 45° respectively.

Table 5. 1 Cost analysis for mining at various pit slope angles (Field Data, 2006)

Stack	Density (MN/m ³)	Stack Depth (m)	Slope Angle (deg)	Material Moved (t)	Cost of Material moved @ 77c/t (\$)	Percentage increment (%)
Upper Hangingwall stack	0.026	150	38	3874525.4	2983384.6	
	0.026	150	30	5222487.5	4021315	35
	0.026	150	35	4316326.8	3323572	11
Lower Hangingwall stack	0.026	50	37	3890018.7	2995314	0.4
	0.026	50	42	3818313.6	2940101	-1
Footwall stack	0.027	60	41	1510316.5	1162944	
	0.027	60	39	1552191.4	1195187	3
	0.027	60	45	1435816.7	1105579	-5

(Field Data, 2007)

5.9 Material tonnage determination

Comparisons in the above cost calculations assume a continued use of phase II overall slope angles (i.e. 38° and 41° for the hangingwall and footwall respectively). Table 5.10 shows the increase and decrease of material moved as a result of changing slope angles from the previously used ones in phase III to those suggested by this research.

Table 5. 2 Amount of material determination using the incremental analysis for the mined material
(Source: Field data)

Stack/Area	Original angle (phase II)	Suggested angle	Material mined [t]	Incremental material [t]
Hangingwall	38	-	3874525.4	
Upper hangingwall		30	5222487.5	1347962.1
		35	4316326.8	441801.4
Lower hangingwall		37	3890018.7	15493.3
		42	3818313.6	-56211.8
Footwall	41	-	1510316.5	
		39	1552191.4	41874.9
		45	1435816.7	-74499.8

(Field Data, 2007)

The shaded cells with negative values indicate suggested steeper slopes which are economically suitable in as far as mining stripping ratios are concerned.

5.10 Slope Management Strategies

Once a rock slope has been designed, constant checks are required during the time of mining to examine areas of geotechnical concerns. Management of rock slopes is not just a responsibility of geotechnical engineers. Inputs from planning and survey sections are very vital as well as those from the production crews.

More often than not, men in the production line will be able to identify signs of an unstable block in the slope face, where there is potential for failure. One of the functions of the geotechnical engineering section is to establish measures for slope monitoring. In this section, some of the methods of slope management practiced at the Nchanga Open Pits (NOP) have been reviewed.

5.10.1 Slope over-cutting

Sometimes a slope will fail because the material that makes up the slope will no longer support the force induced by the adjacent upper block. To reduce the amount of force exerted on this slope, the upper slope can be cut-out. By so doing the excessive force is reduced and bringing up the safety factor to acceptable standards. The standards of the safety factor will

vary from place to place, depending on the rock mass structural properties. For material with both friction and cohesion, slope over-cutting reduces and stabilises the slope against rotational shear failure.

5.10.2 Slope movement monitoring

Before failure occurs, there are measurable signs to look for that would indicate the imminent failure. These can be sloughing of material from the rock slope, miniature movement of the slope and finally the large scale failure and of the entire wall.

5.10.3 Survey monitoring points.

The geotechnical engineering section, in collaboration with the survey departments, establish the monitoring points. These are points selected due to the stability conditions existing at a given time.

The procedure involves the installation of pegs (survey targets) to be sited using survey instrumentation like the total station. The targets are sighted at given times and the average vertical movements of these targets are recorded. This gives relative movement trends of a wall. The source of error in this procedure comes in when the point where the total station is mounted records some movement. So care should be exercised when selecting the survey stations. The station must be established on a relatively stable ground that will not move itself.

5.10.4 Slope Stability Radar (SSR)

The slope stability radar is an electro-optical measuring device that uses wave technology and a camera, which scans a set area in the slope for any movements. This is the state-of-the-art development for monitoring slope movement in open pit mines. It offers unprecedented sub-millimetre precision and broad area coverage of wall movements through rain, dust and smoke. The real-time display of the movements of mine walls has allowed continuous management of the risk of slope instability at a mine operations level.

This monitoring occurs without the need for mounted reflectors or equipment on the wall and the radar waves adequately penetrate through rain, dust and smoke, 24 hours a day (Harries et al, 2006)

5.11 Tension Cracks

Tension cracks are a genesis of most slope failures. When they are filled with water, the water forces generated can induce very sudden failures. There is need to monitor the progression of these cracks. Walking surveys may prove to be very helpful in locating tension cracks which will give an insight into areas of high tension with the pits. It is important to prevent water from entering the cracks by sealing them with a flexible impermeable material such as clay (Hoek and Bray, 1977)

5.12 Slope Draining

It is very important to drain out water from the slopes. The pressure that ground water builds reduces the effective shear stresses of the slope materials. Drainage should be positioned so that it reduces the water pressure in the immediate vicinity of the slope.

5.13 Rock Falls

One of the dangers associated with rock slopes is that of falls of loose rocks and boulders from the face of the slope. Such boulders can fall, bounce or roll down a slope (Hoek and Bray, 1977).

A ditch at the foot of the slope will contain much of the energy of the fall and a chain-link fence on the shoulder of the trench will prevent the rock from bouncing on to the road way.

5.14 Rock Bolts

Some slopes can be reinforced with rock bolts. The use of the rock bolts would increase the factor of safety of the rock slope. Calculations must be done to get the right number of rock bolts to create the desired force that will increase the safety factor to the required level. Mostly, slopes reinforced with rock are intended for temporary or semi temporary especially if they are associated with surface structures such as treatment plants. At the Nchanga Open Pits, extensive use of rock bolts has been made at the *East Mill* section where the rock

material underneath this structure was believed to be unstable. This East Mill stands up to date.

5.15 Slope Management Strategies

Continuous field observations will ensure early identification of problem area in the slopes. Activities such as field mapping should be carried out on regular basis to confirm geological structures of the slopes, together with ground water and tension cracks.

Piezometers must be installed to continuously monitor ground water. There is constant need to monitor the water table even as the dewatering program progresses.

Horizontal drain systems, such as manifold drain pipes, are very effective in reducing the ground water table of the mining area and everything should be done to stream water from entering the pit as this defeats the essence of dewatering.

A reduction in slope height can be effective in the increase of the safety factor. In essence this reduction means a reduction in the weight of the block with the potential of sliding. Also when reducing a slope height, some tension cracks in progress are disrupted, sealing off the cracks from filling with water.

Alarm systems can be devised to indicate movements that should raise the concern of the relevant personnel. Extensometers can be fitted with alarm triggering electrodes that would alert the rock slope engineers in case of larger crack separation.

Many of the slope management practices reviewed above are currently being undertaken at the Nchanga open pits. However, it is suggested that the frequency and procedures of monitoring must be increased

5.16 Risk Assessment

At the time of research risks associated with phase III were largely predicted from the slopes designated in various stacks. Assessment of risks was comprehensively done in phase II where production was taking place. These risks were a result of many factors including

undercutting of slopes, likelihood of failure due to persistent tension cracks and the effects of ground water on the slopes. However, these risks were either economic or safety risks. Economic risks were classified for the effects on the cost of remedying them while the safety risks classified for their potential to injure workers and even their fatality rates. Table 5.3 shows a summary of some of the risk areas that were identified during research.

Table 5. 3 Risk assessment performed in selected areas (Source: Field data)

SLOPE AREA	RISK LEVEL – SAFETY/ECONOMIC	MITIGATION	SUMMARY
<i>Ramp Slope, Western Wall, COP F-D Phase III</i>	E: Needs constant monitoring to prevent undercutting of ramp.	- Detailed Monitoring in place – to be maintained - Anchor load testing on a regular basis	Acceptable low risk, but implications of failure that would destroy ramp are high.
<i>Upper Slope Western Wall</i>	S: Upper slope – low. Latter stages, mining may undercut BSS. E: Moderate to high depending upon accepted design.	- Requires detailed design and crack monitoring - Detailed monitoring if BSS is to be undercut in lower slope	- Potential high risk in lower slope. - Detailed design/review required before mining design can be completed. - Possible revised mining plan.
<i>Eastern Wall OB 5 (dump area)</i>	S: High on OB (dump) situated on footwall. Probable local mobilization of failure debris. E: Low provided geotechnical design accepted	- Geotechnical control of mining operations essential - Strict compliance with dewatering/depressurization requirements. - Detailed monitoring of movement and water pressures, at least weekly	- Acceptable (moderate to low) risk levels provided mining carefully controlled and detailed monitoring in place. - Field modification to design to be expected.

General risks of the COP FD Phase III at this time are theoretical. However, risk assessment of the COP FD Phase II brought out issues that would relate to Phase III once mining begins.

5.17 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.

5.17.1 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*.⁹ 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 (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 τ , applied to the

⁹ 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 while the

5.18 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

5.18.1 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.

$$W \sin \psi + V = cA + (W \cos \psi - U) \tan \phi \quad (43)$$

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:

$$\tau = c' + (\sigma_n - U_a) \tan \phi' + (U_a - U_w) \tan \phi^b \quad (44)$$

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).

5.18.2 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 β to the plane as shown in figure 2.4:

5.19 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:

$$F = \frac{cA + (W \cos \psi - U + T \sin \beta) \tan \phi}{W \sin \psi + V - T \cos \beta} \quad (45)$$

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:

$$F = \frac{\tan \phi}{\tan \beta} \quad (46)$$

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.

In summary, Table 5.4 shows stack information obtained from the design exercise. The stacks have been presented and all the information on their characteristics has been shown. It is necessary to consider these units in each stack a single unit due to their similar characteristics.

Table 5. 4 Summary of design parameters for COP FD Phase III. Nchanga, 2007

Rock	Rock formations	Stack Height [m]	Stack RMR		Density [MN/m ³]	Internal friction angle [Deg]	Cohesion (c) [MPa]	Overall slope angle [Deg]	Pit Design	
			Bieniawski	Laubsche					Berm width [m]	Slope angle [Deg]
Overburden rock	URD, SWG, CDOL, DOLSCH	60 – 200	37	32	0.026	32.46	0.226	30-35	10	
Overburden rock	UBS, TFQ, TFQT, BSSU, PQ/SM, BSSL, LBS TR	30 – 70	39	36	0.025	40.01	0.29	37-42	10	
Wall rock	ARK, BAS	40 – 80	57	53	0.027	41.91	0.419	39 - 45	10	

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

Drill core examined during core logging comprised of newly drilled core and that left from previous analyses and activities. Logging of the previously drilled core provided the 'missing' information for various areas of the study site. A total of 3000 m of core was logged in this exercise, which formed an important basis for information collection for the research.

Geotechnical mapping, done in cells, was successfully carried out with over 13000 m² of the area mapped. A challenge arose in mapping inaccessible areas, a few of these areas had been mapped before and so a simple photography-assisted mapping was done. The information from mapping and core logging provided a vast array of information for the geotechnical zoning of COP FD. These zones were designed by considering rock mass behaviour and properties (such as RMR) exhibited in various conditions and areas. Three geotechnical zones called *stacks* were identified using this method: (1) the upper hangingwall stack; with RMR ranging from 25 – 40, stack height of 60 – 200 m, factor of safety of 1.1 and the design angle (stack angle) of 30 – 35°; (2) the lower hangingwall stack; with RMR range of 30 – 45, stack height of 30 – 70 m, factor of safety of 1.98 and stack angle of 37 – 42° and (3) the footwall stack; with RMR range of 40 – 55, stack height of 40 – 80 m, factor of safety of 1.96 and stack angle of 39 – 45°.

The general rock mass characterisation in this area based on the RMR is **FAIR** to **POOR** rock, based on the international rating systems. This meant the rock was generally of low geotechnical strength and extra care should be taken when making slope designs.

The geotechnical models designed in this research were useful in understanding rock and soil properties, trends and their extents. These models were graphical representation of the drill holes, geological contacts, weak zones as well as topographies.

The stability analysis of this research is largely based on the limit equilibrium methods of safety factor calculation, probability of failure analysis and design parameter determination. This research has extensively used software which is based on these principles and, to a lesser degree, on numerical methods approach.

Slope management, an important aspect of slope mining, was instituted at the research site. Most of the management procedures are existent at COP FD. It is however the accuracy, consistency and methods of implementing these procedures that have been emphasised upon.

The slope designs in this research have been compared with those used in phases I and II. Two geotechnical zones have been suggested in this study for the hangingwall in phase III while only one zone exists in the hangingwall for phase II. An increase of up to 35% in the cost of mining at the slope of 30° in the hangingwall and 3% in the footwall at the angle of 39°.

A risk assessment was carried out and three risk areas were identified (during this research)¹⁰ that needed extra attention from geotechnical section personnel during routine inspections and monitoring. These risk were either economic or safety. Risks were compounded by seasonal rainfall which was responsible for major slope instability of pit slopes. With annual rainfall of 1,194 mm, many slopes were found to be highly saturated. Tension cracks were weakened and sliding was unavoidable in weaker formations.

It is envisaged that further research could be undertaken in determining the extent of slope stability in the research site using numerical modelling methods, e.g. finite element methods of modelling.

¹⁰ These risks may have been removed at the time of compiling this report as they were in an active phase of the pit (COP FD Phase II)

6.1 Recommendations

After determining the geomechanical and geotechnical aspects of the pit, it must be understood that the actual slope angles used in the pit will depend upon (i) the presence of haul roads, or ramps, necessary for the transportation of the blasted ore from the pit, (ii) possible blast damage, (iii) ore grades, and (iv) economical constraints. These factors require that there is adequate manoeuvring room for the hauling and loading equipment during production at the lowest elevation in the pit. Appendix 8 illustrates the proposed phase II and phase III pit designs in relation to the geology and the existing phase II designs. Currently the Nchanga Open Pits maintain mining radius of not less than 30 m due to the equipment in use.

6.1.1 Pit Design Tips

Pit design should consider the intersections with the rock formation. For stability reasons, weaker and weathered formations should not be cut in to the slope face as this would provide the genesis for sloughing and eventual slope failure. In this study it recommended that berm widths be maintained at 10 m, slope faces at 70 degrees and ramps of 40 m. This of course will take into consideration the overall slope angles determined earlier in the study.

The design parameters obtained in this research are to be used as guiding parameters at every level for design purposes. However, engineering judgment must play a pivotal role in ensuring the right slope angles for the right rock formations are planned. Most geotechnical engineers will use previous information and alongside the new information to determine the variations of design parameters.

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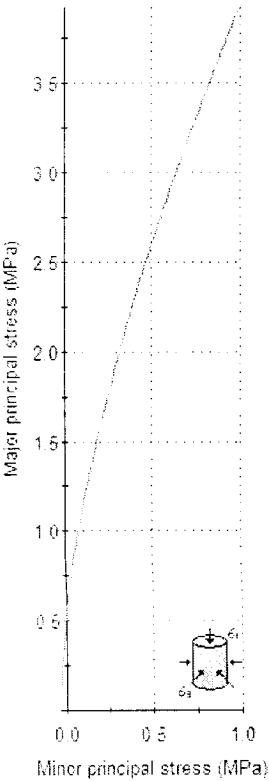
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APPENDICES

APPENDIX 1 Determination of Design Parameters using RocLab™

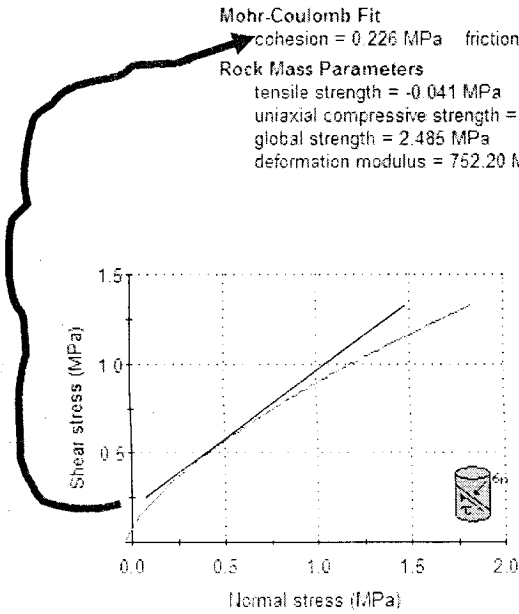
UPPER HANGINGWALL STACK



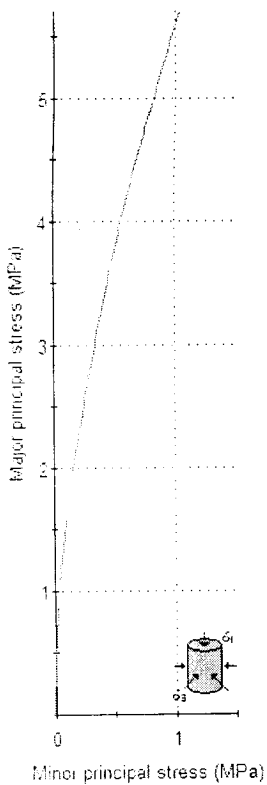
Hoek-Brown Classification
intact uniaxial comp strength (σ_{ci}) = 42 MPa
GSI = 30 m_i = 4 Disturbance factor (D) = 0.2
intact modulus (E_i) = 12000 MPa
Hoek-Brown Criterion
 m_b = 0.249 s = 0.0002 a = 0.622

Mohr-Coulomb Fit
cohesion = 0.226 MPa friction angle = 32.46 deg

Rock Mass Parameters
tensile strength = -0.041 MPa
uniaxial compressive strength = 0.541 MPa
global strength = 2.485 MPa
deformation modulus = 752.20 MPa



LOWER HANGINGWALL STACK



Hoek-Brown Classification

intact uniaxial comp. strength (σ_{ci}) = 44 MPa
GSI = 39 m_i = 15 Disturbance factor (D) = 0.7
intact modulus (E_i) = 12000 MPa

Hoek-Brown Criterion

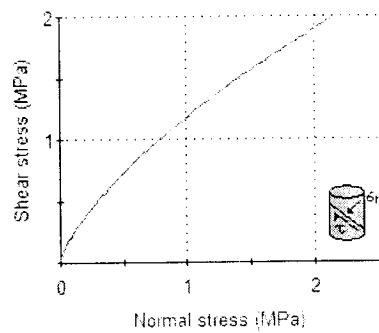
m_b = 0.525 s = 0.0001 a = 0.512

Mohr-Coulomb Fit

cohesion = 0.290 MPa friction angle = 40.01 deg

Rock Mass Parameters

tensile strength = -0.012 MPa
uniaxial compressive strength = 0.475 MPa
global strength = 4.012 MPa
deformation modulus = 661.05 MPa



FOOTWALL STACK

Hoek-Brown Classification

intact uniaxial comp. strength (σ_{ci}) = 80 MPa
 GSI = 53 m_i = 11 Disturbance factor (D) = 1
 intact modulus (E_i) = 12000 MPa

Hoek-Brown Criterion

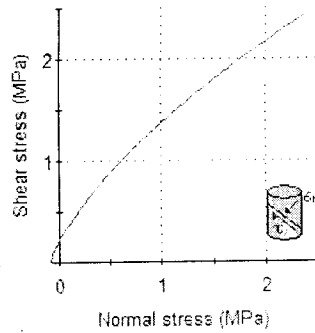
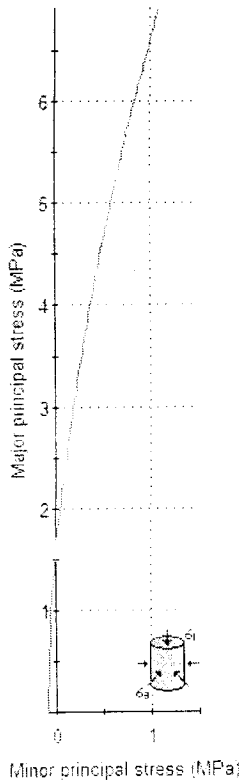
m_b = 0.383 s = 0.0004 a = 0.505

Mohr-Coulomb Fit

cohesion = 0.416 MPa friction angle = 41.91 deg

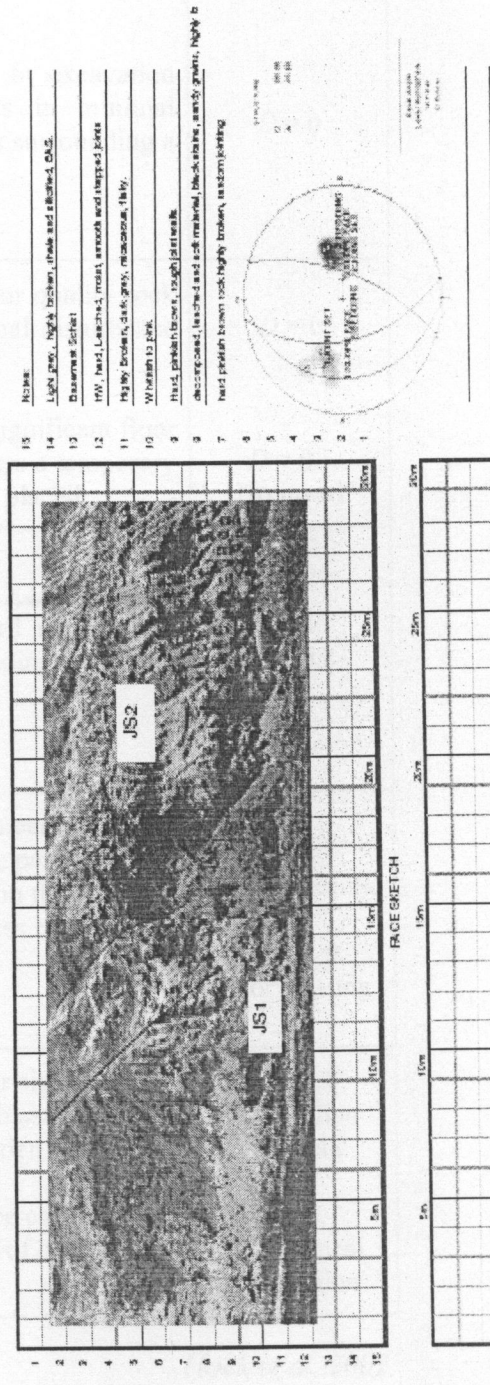
Rock Mass Parameters

tensile strength = -0.083 MPa
 uniaxial compressive strength = 1.536 MPa
 global strength = 6.533 MPa
 deformation modulus = 955.22 MPa

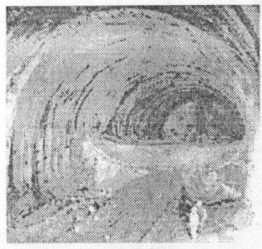
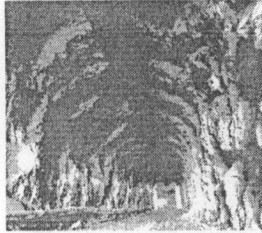
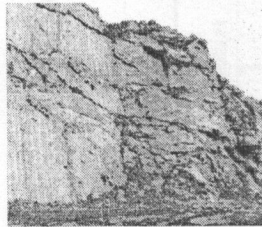
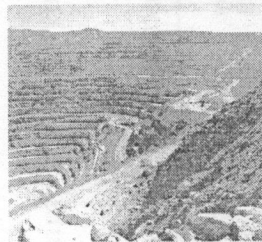


Si on cherche à appuyer sur le bouton, l'écoulement de l'air est interrompu.

Bench: 103mB										Section		Date		Mapcity		SRM/CRACK		Date Entered		DN		SRM/CRACK		Cell No.		SRM/CRACK	
SRM		CCP MD		PRJ-SE 2		Station		Cell No.		WI		H		Reck Type		Fdb		Type		Type		Type		Type		Type	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
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C		D		E		F		G		H		I		J		K		L		M		N		O		P	
C		D		E		F		G		H		I		J		K		L		M		N		O		P	
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C		D		E																							



APPENDIX 3 Determination of the Disturbance factor D , of blasting production

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from over burden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

Hoek et al, 2002

APPENDIX 4 Sample Table for core logging data collection

Hole ID:	DGT1 5	Site: COP F	Coor d	X =6053.282	Y = 15811.018			Z =1314.946		Inclin: 54	Bearng: 117							
From	To	Rock Type	RQD	Bieniawski 1989	Intact Rock Strength (MPa)	Rec %	No. of Break(s)		Disc Type(s)	Joint Conditions				Ground water	Notes/Comments	Run Length	Core Size	Reco erec Core Leng
							Natur al	Drill		FF/m	Joint Separ ation (mm)	Joint Rough	Joint Infill					
0.00	3.68	LAT	0	15	15	0	33	0	0	0.8					Reddish brown, coarse grained, soft soil, HW, easily molded with fingers.	3.68	HQ	1.21
3.68	5.83	LAT	0	15	15	0	28	0	0	1.7						2.15	HQ	0.60
5.83	8.83	LAT	0	15	15	0	44	0	0	0.8					Red, fine to medium grained, HW, very weak to soft soil, easily molded with fingers.	3.00	HQ	1.31
8.83	11.83	LAT	0	15	15	3	55	0	0	0.6						3.00	HQ	1.66
11.83	14.83	SWG	20	33	30	3	29	4	2	4.7	BD	2			Red, fine grained, HW, very weak tyo soft, easily molded with fingers, color change to whitish grey from 17.4m, foliated, talcose, schistosity	3.00	HQ	0.80
14.83	17.83	SWG	20	35	31	10	39	5	4	4.3	BD	2				3.00	HQ	1.16
17.83	20.83	SWG	42	40	33	10	53	4	6	2.5	BD	2			Whittish brown, fine grained, HW, very weak to soft, talcose, foliated, laminated, wek along foliations, micaceous, thinly bedded	3.00	HQ	1.60
20.83	23.83	SWG	34	36	25	15	97	20	9	6.8	BD	2				3.00	HQ	2.90
23.83	26.83	SWG	61	44	34	20	94	8	5	2.8	BD	2			greyish brown to green, fine grained, HW, very weak to mod hard, weak along BD directions, foliated, laminated, talcose	3.00	HQ	2.80
26.83	29.83	SWG	32	37	31	20	60	8	5	4.4	BD	2				3.00	HQ	1.80
29.83	32.83	SWG	18	30	23	2	43	12	6	9.2	BD	2			Brown to greyish green, fine grained, HW, very weak to soft, talcose, micaceous, weak along foliations, highly broken, laminated, smooth joint planes, thickly bedded.	3.00	HQ	1.30
32.83	35.83	SWG	25	32	21	10	53	15	5	9.5	BD	2				3.00	HQ	1.50
35.83	38.83	SWG	64	42	24	8	99	16	9	5.4	BD	2				3.00	HQ	2.90
38.83	41.83	SWG	59	41	27	10	93	17	6	6.1	BD	1				3.00	HQ	2.70
41.83	44.83	SWG	15	33	26	50	51	17	2	11.1	BD	2			Greyish brown, fine grained, HW, very weak to soft rock when wet, foliated, banded,	3.00	HQ	1.50
44.83	47.83	SWG	18	32	23	30	46	17	3	12.2	BD	2				3.00	HQ	1.30

47.83	50.83	SWG	10	30	24	40	43	17	2	13.1	BD	2
50.83	53.83	CDOL	19	34	30	40	53	11	7	6.9	BD	2
53.83	56.83	CDOL	23	33	22	40	80	29	7	12.1	BD	2
56.83	59.83	CDOL	31	35	25	20	69	18	7	8.7	BD	2
59.83	62.83	CDOL	32	35	23	30	64	20	4	10.4	BD	2
62.83	65.83	DOLSC H	13	31	23	30	46	17	3	12.2	BD	2
65.83	68.83	DOLSC H	20	33	22	40	65	21	3	10.7	BD	2
68.83	71.83	DOLSC H	33	36	24	35	63	17	3	8.9	BD	2
71.83	74.83	DOLSC H	12	27	20	0.5	32	11	2	11.5	BD	2
74.83	77.83	DOLSC H	17	29	20	2	39	13	3	11.2	BD	2
77.83	80.83	DOLSC H	16	30	21	0.5	32	8	4	8.2	BD	2
80.83	83.83	DOLSC H	8	29	24	0.5	18	4	1	7.5	BD	2
83.83	86.55	UBS	0	27	27	5	11	2	1	6.9	BD	2
86.55	89.55	UBS	11	27	21	5	24	10	3	13.9	BD	2
89.55	92.55	UBS	0	24	19	5	21	11	3	17.2	BD	2
92.55	95.55	TFQ	10	37	38	60	35	13	2	12.4	BD	2
95.55	98.55	TFQ	53	49	41	80	95	21	4	7.3	BD	2
98.55	101.5 ₅	TFQ	34	49	42	80	71	19	3	8.9	BD	2

None

Rough
Planar

Rough
Undulating

		talcosc, bedded, HW weak and soft zone from 44m-47.83m, highly broken zone from 47.83m-50.83m with quartz pebbles.	3.00	HQ	1
		Brown to light grey, fine grained, HW, mod hard to hard, talcosc, highly broken from 50.83m-53.83m and 55m-56.83m, weak along bedding directions, foliated, smooth joints, pitted, banded and weathered weak zone 56.83m-59.83m, schistosity	3.00	HQ	1
		Brown to light grey, fine grained, HW, mod hard, highly broken, weak along foliations, banded, talcosc, foliated weathered.	3.00	HQ	1
		Brownish grey to green, fine to medium grained, weak to mod hard, foliated, schistosity, gritty bands, weak along laminations and bedding directions	3.00	HQ	1
		Darkish brown to greyish green, fine grained, HW, very weak to soft, micaceous, talcosc, slippery, weak along foliations, HW weak soft zone from 72m-83.83m, schistosity present.	3.00	HQ	1
		Grey, fine grained, HW, laminated, pitted, white infilling, interbedded with feldspar quartz bands, weak and soft	2.72	NQ	1
			3.00	NQ	1
			3.00	NQ	1
		Brown, coarse grained, HW, feldspathic, mod hard, thickly bedded, sugary grains, pitted, highly broken from 92.55m-96.4m, color change to whitish brown at 95.55m, foliated.	3.00	NQ	1
		Brown, coarse grained, MW, foliated, silicified joints, highly broken zone from 99.1m-	3.00	NQ	1

Moderately
Weather

101.5 5	104.5 5	TFQ	6	42	45	80	31	12	4	12.9	BD	2	Rough Undulating	ed	99.78m, broken from 101.55m-103m	3.00	NQ
104.5 5	107.5 5	ARK	19	46	48	150	32	9	3	9.4	BD	2			Pinkish white to brown, coarse grained, MW, hard, bedded, weak along BD planes, foliated, pitted.	3.00	NQ
107.5 5	110.5 5	ARK	13	39	37	50	18	4	7	7.4	BD	2			Whittish grey, coarse grained, HW, leached, mod. hard, bedded, highly broken, quartz bands from 111.15m-113.55m glassy and hard band, highly broken from 111.15m-113.65m	3.00	NQ
110.5 5	113.5 5	ARK	4	43	48	200	51	17	5	11.0	BD	2			Brown, coarse grained, HW, weak, bedded, foliated, weak along Bd directions, highly broken from 114m-116.7m, hard pockets	3.00	NQ
113.5 5	116.5 5	ARK	17	31	20	10	41	13	1	10.5	BD	2		Highly Weathered		3.00	NQ
116.5 5	119.5 5	ARK	38	39	30	25	51	9	4	5.9	BD	2				3.00	NQ
119.5 5	122.5 5	ARK	9	32	23	40	40	14	4	11.6	BD	2			Brown, medium grained, HW, mod. weak, foliated, silicified, black foliations, bedded, weak along BD planes,	3.00	NQ
122.5 5	125.5 5	ARK	5	46	51	250	33	11	2	11.0	BD	2		Moderately Weathered		3.00	NQ
125.5 5	128.5 5	ARK	11	48	53	250	22	5	2	7.6	BD	2				3.00	NQ
128.5 5	131.5 5	ARK	4	47	53	250	17	4	3	7.8	BD	2				3.00	NQ
131.5 5	134.5 5	ARK	24	39	31	20	99	37	7	12.4	BD	2			Brown, coarse grained, HW, mod. hard, weak zone from 131.55m-132m, sandy grains, bedded, color change to grey at 134m	3.00	NQ
134.5 5	137.5 5	ARK	71	56	42	150	98	15	3	5.1	JN	1				3.00	NQ
137.5 5	140.5 5	ARK	61	60	63	200	99	14	10	4.7	JN	2				3.00	NQ
140.5 5	143.5 5	ARK	81	57	68	250	100	11	3	3.7	JN	2				3.00	NQ
143.5 5	146.5 5	ARK	70	64	59	250	98	16	6	5.5	BD	2				3.00	NQ
146.5 5	149.5 5	ARK	95	72	85	250	99	7	2	2.4	BD	1				3.00	NQ
149.5 5	152.5 5	ARK	85	63	67	300	100	17	4	5.7	BD	1				3.00	NQ
152.5 5	155.5 5	ARK	85	65	64	300	100	19	1	6.3	BD	2				3.00	NQ

APPENDIX 5 Statistical comparisons for RMR of Bieniawski and Laubscher

Two-Sample T-Test : Bieniawski Core Logging vs. Laubscher Core Logging

Two-sample T for BienCoreLog vs. LaubCoreLog

	N	Mean	StDev	SE Mean
BienCoreLog	8	41.25	8.21	2.9
LaubCoreLog	8	37.38	8.40	3.0

Difference = μ (BienCoreLog) - μ (LaubCoreLog)

Estimate for difference: 3.88

95% CI for difference: (-5.03, 12.78)

T-Test of difference = 0 (vs. not =): T-Value = 0.93 P-Value = 0.366 DF = 14

Both use Pooled StDev = 8.3039

Two-Sample T-Test : Bieniawski Mapping vs. Laubscher Mapping

Two-sample T for BienMapping vs. LaubMapping

	N	Mean	StDev	SE Mean
BienMapping	8	39.75	6.27	2.2
LaubMapping	8	37.63	6.07	2.1

Difference = μ (BienMapping) - μ (LaubMapping)

Estimate for difference: 2.13

95% CI for difference: (-4.49, 8.74)

T-Test of difference = 0 (vs. not =): T-Value = 0.69 P-Value = 0.502 DF = 14

Both use Pooled StDev = 6.1724

Two-Sample T-Test : RMR Classification ; Core Logging vs. Mapping

Two-sample T for ClassCoreLog vs. ClassMapping

	N	Mean	StDev	SE Mean
ClassCoreLog	8	3.563	0.496	0.18
ClassMapping	8	3.563	0.496	0.18

Difference = μ (ClassCoreLog) - μ (ClassMapping)

Estimate for difference: 0.000

95% CI for difference: (-0.531, 0.531)

T-Test of difference = 0 (vs. not =): T-Value = 0.00 P-Value = 1.000 DF = 14

Both use Pooled StDev = 0.4955

APPENDIX 6 Piezometric Values of water levels measured at Konkola Copper Mines COP FD Phase III

The piezometer values for water levels were measured to determine the water level at various places so that information for pumping water from the pit and the surrounding areas can be obtained. The table shows selected values of piezometer measurements.

<i>WATER LEVELS AT SELECTED POINTS FOR COP FD PHASE III</i>			
BHID	Water Level	Comment	
DW8	87		
DW9	74	high water level	
DW10	88		
DW11	89		
DW12	76		

APPENDIX 7 Wedge analysis using SWEDGE

Swedge Analysis Information

Document Name:

105 mB wedge failure analysis

Job Title:

SWEDGE - 105 meter bench

Analysis Results:

Analysis type=Deterministic
Safety Factor=286.412
Wedge height (on slope) =1.39501 m
Wedge width (on upper face) =10 m
Wedge volume=2.55062 m³
Wedge weight=6.88668 tonnes
Wedge area (joint1) =7.78897 m²
Wedge area (joint2) =6.33782 m²
Wedge area (slope) =0.955546 m²
Wedge area (upper face) =6.37062 m²
Normal force (joint1) =7.18963 tonnes
Normal force (joint2) =6.14543 tonnes

Failure Mode:

Sliding on intersection line (joints 1&2)

Joint Sets 1&2 line of Intersection:

Plunge =18.048 deg, trend=183.189 deg
Length =10.9644 m

Trace Lengths:

Joint1 on slope face=1.75206 m
Joint2 on slope face=1.54456 m
Joint1 on upper face=10.0402 m
Joint2 on upper face=10.0071 m

Maximum Persistence:

Joint1=10.9644 m
Joint2=10.9644 m

Intersection Angles:

J1&J2 on slope face = 44.9266 deg
J1&Crest on slope face = 58.8808 deg
J1&Crest on upper face = 84.8725 deg
J2&Crest on slope face = 76.1926 deg
J2&Crest on upper face = 87.8421 deg
J1&2 on upper face = 7.28543 deg

Joint Set 1 Data:

Dip =55 deg, dip direction=260 deg
Cohesion =42.41 tonnes/m², friction angle=41.91 deg

Joint Set 2 Data:

Dip =70 deg, dip direction=100 deg
Cohesion =42.41 tonnes/m², friction angle=41.91 deg

Slope Data:

Dip =65 deg, dip direction=185 deg
Slope height=100 meters
Bench width=10 meters
Rock unit weight=2.7 tonnes/m³
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

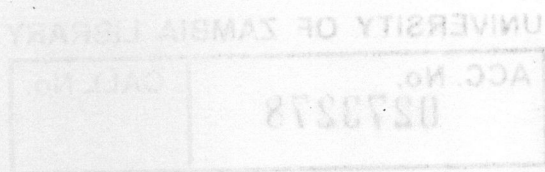
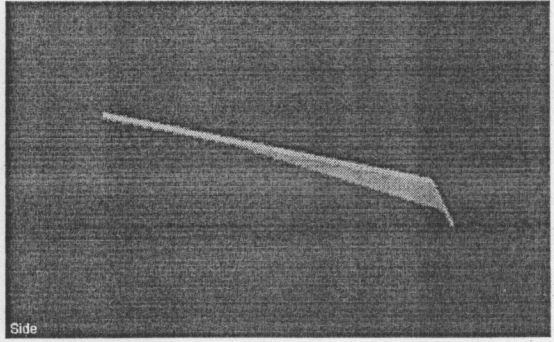
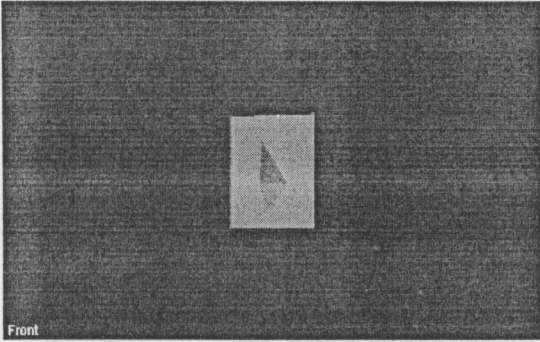
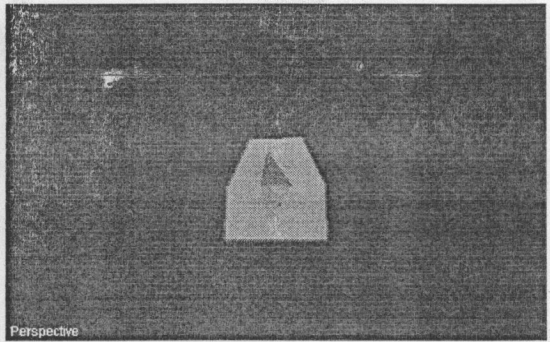
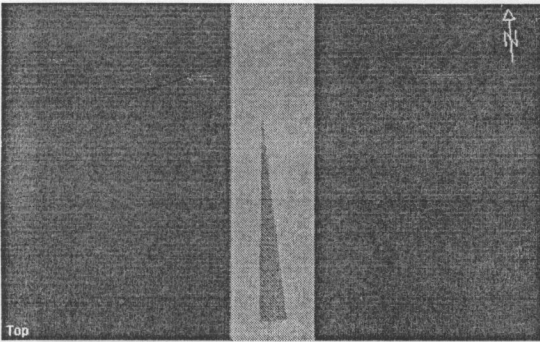
Upper Face Data:

Dip =12 deg, dip direction=195 deg

Wedge Vertices:

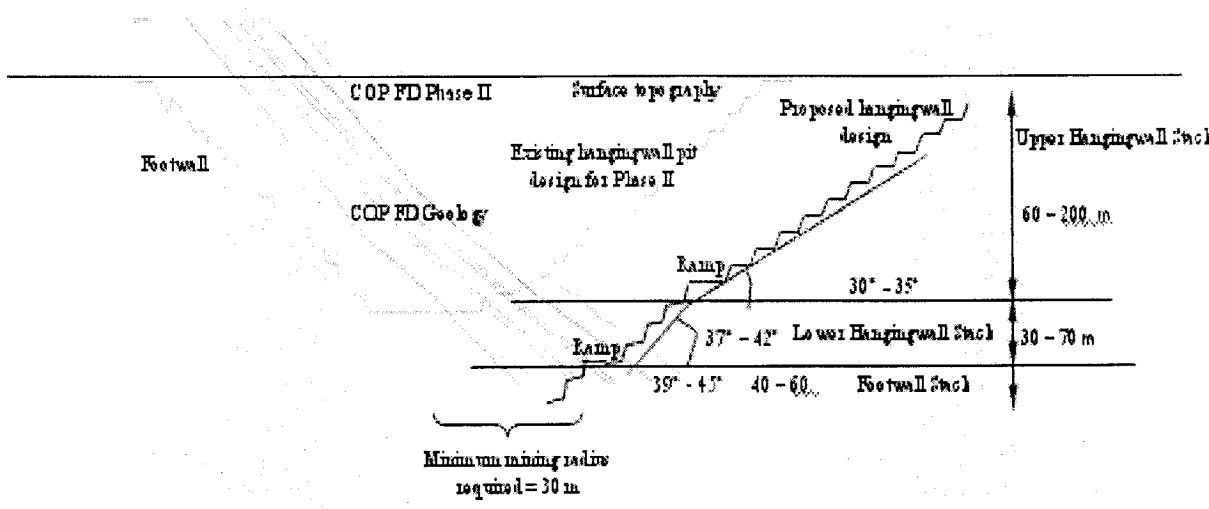
Coordinates in Easting, Northing, Up Format
1=Joint1, 2=Joint2, 3=Upper Face, 4=Slope
Point 124: 0, 0, 0
Point 134: 0.89, 0.575, 1.4
Point 234: -0.38, 0.662, 1.34
Point 123: 0.58, 10.4, 3.4

The figure below shows the various projections of the wedge which was formed at the 105 meter bench. This was found to be a stable wedge. Comparing this wedge with the shear parameters provided information on its stability. Further investigation showed that the wedge was very stable under the prevailing circumstances (i.e. dry conditions)



APPENDIX 8

Proposed pit design slopes. Note varying overall slope angles for the stacks.
(source: field Data, 2007)



It is worthwhile to note that the footwall structure remains the same for the whole slope of the pit. However, the hangingwall strata differ especially with the increasing depth of the pit.