Probabilistic Stability Assessment of a High Slope in Variable Strength Rocks

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Summary A probabilistic stability assessment is presented for a high slope in a relatively complex geological setting with variable geotechnical conditions. Consideration of the basic mechanics indicated a critical failure mechanism involving combined defect and rock mass failure. Rock mass strength properties were predicted from either triaxial shear strength testing or applying the Hoek-Brown failure criterion using material constants estimated from Geological Strength Index values derived from borehole logs. Defect shear strengths were estimated from direct shear strength testing. Back analysis of previous failures provided field verification of selected strength parameters. Deterministic analyses were initially used to determine the critical slope segment for probabilistic analysis. The probabilistic method considered both probability of failure through the rock mass in all units and probability of failure given that adversely oriented and located defects occur in one or more units. Recommendations are made for refinement of the method for future use.

1. INTRODUCTION

Slope design in soils and rocks inherently involves dealing with uncertainty in the geological model and variability in the governing parameters such as strength and groundwater. Methods for quantifying variability and uncertainty are referred to as probabilistic analyses.

This paper summarises a probabilistic stability assessment carried out for a 380m high final wall in a large open pit gold mine. This example is considered to be of interest because of the relative complexity of the geological setting, the variable nature of the geotechnical conditions and the probabilistic model and method adopted.

The methods used to model rock mass and defect strengths are presented together with assumptions made on the model geometry and groundwater conditions. The probability of failure considered defect controlled and rock mass failure and its determination is outlined.

2. GEOTECHNICAL DESCRIPTION

The geological setting of the high wall comprises a major thrust fault which dips into the slope towards the north-northwest at about 50° (Figure 1) and approximately divides the wall in half. The upper half consists of an older sedimentary sequence of mainly medium strength interlaminated carbonaceous siltstone and fine grained sandstone of good rock mass quality. The lower half contains a younger tuff unit which is of high strength and good rock mass quality.

In addition to the sedimentary sequence, the upper half of the wall also exposes diatreme breccia, rhyolite and basalt. The diatreme breccia, which is associated with a diatreme pipe centred outside the pit limits, consists of fine to coarse grained angular clasts of sediments, tuff and rhyolite in a matrix of black mud derived from reworked or "milled" sediments originating from the phreatomagmatic eruption which created the diatreme. The breccia is of low to medium strength and fair rock mass quality.

The rhyolite body has been intruded discordant to the diatreme boundary and is a high strength rock of fair rock mass quality. The rhyolite forms a topographic high up to 115m above the surrounding ground surface.

Basalt occurs as dykes intruded into the thrust fault sequence and as flows and laharic deposits capping ridges. The dykes are up to a few metres thick and are fresh, high strength and of very good quality. In contrast the flows have been weathered and are a very stiff clay to low strength rock to a maximum thickness of 50m.

The thrust fault can be divided into two separate brecciated zones 35 to 50m thick both containing brecciated sediments with relatively narrow bands of crush breccia. These are interlayered with variable but generally minor amounts of brecciated tuff and

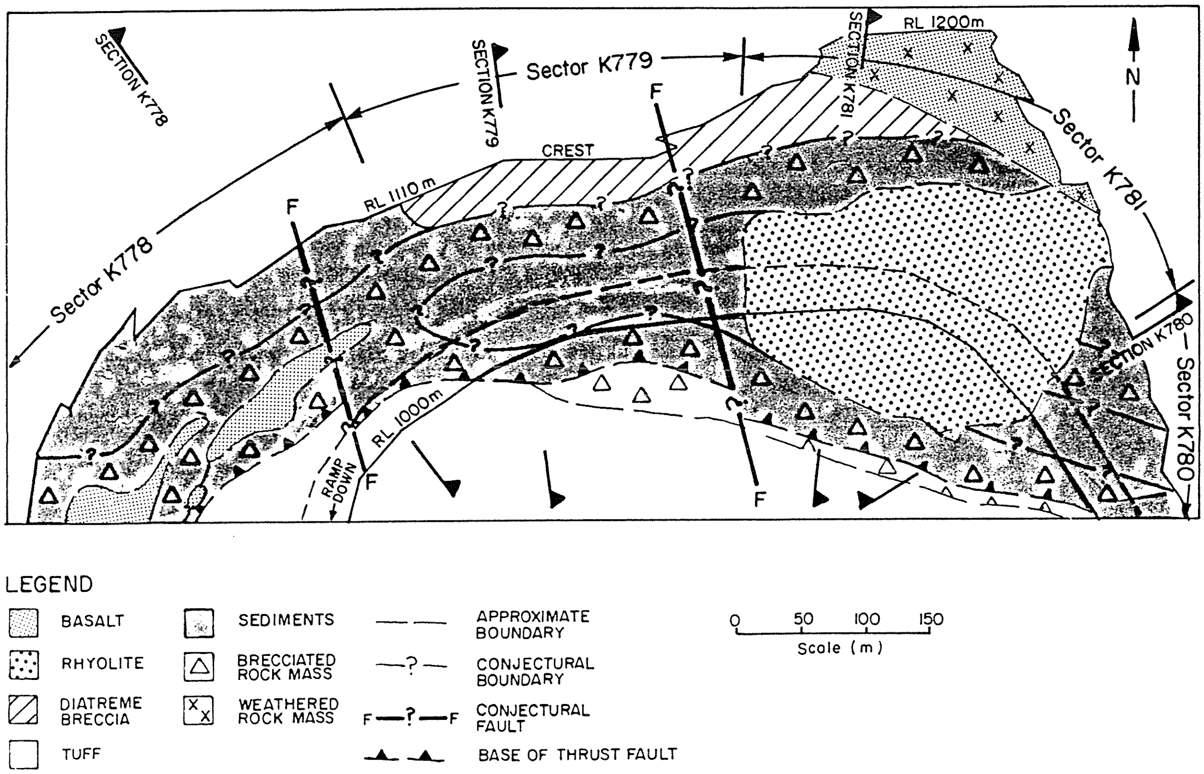


Figure 1. Geological model.

intruded diatreme breccia, rhyolite and basalt which have been "channelised" into the fault zone.

The brecciated rock is characterised by the incipient development of rock fragments with a minor clay matrix. It is very low to low strength with a fair rock mass quality and contains numerous clay seams and sheared zones. Crush breccia occurs in distinct zones less than a few metres thick within the brecciated rock and comprises angular rock fragments in a clay matrix with an intact strength equivalent to a hard soil.

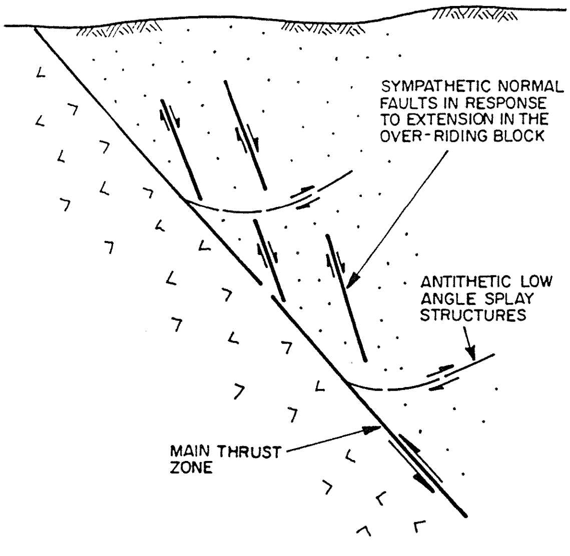


Figure 2. Simplified structural section.

A simplified model of the structural conditions expected within the thrust fault sequence is shown in Figure 2. Antithetic low angle splays dipping opposite to the main thrust direction have been mapped which are listric in nature. Moderate to steep normal faults are also present. These dip in the same direction as the thrust and are the result of sympathetic extension movement in the upper block. Bedding in the sediments dips at shallow to moderate angles generally into the wall towards the northwest to northeast.

Groundwater levels are about 35 to 40m below the surface in the elevated areas around the rhyolite body and are within a couple of metres of the surface in the surrounding relatively low lying areas.

3. ROCK MASS AND DEFECT STRENGTHS

Rock mass strength represents the main source of variability in the probabilistic analysis. Strength properties were predicted from either triaxial shear strength laboratory testing or applying the Hoek-Brown failure criterion using material constants estimated from the Geological Strength Index (GSI) (Hoek et al, 1995). Limited information was also available from back analysis of previous failures and this provided a check on the triaxial and GSI results.

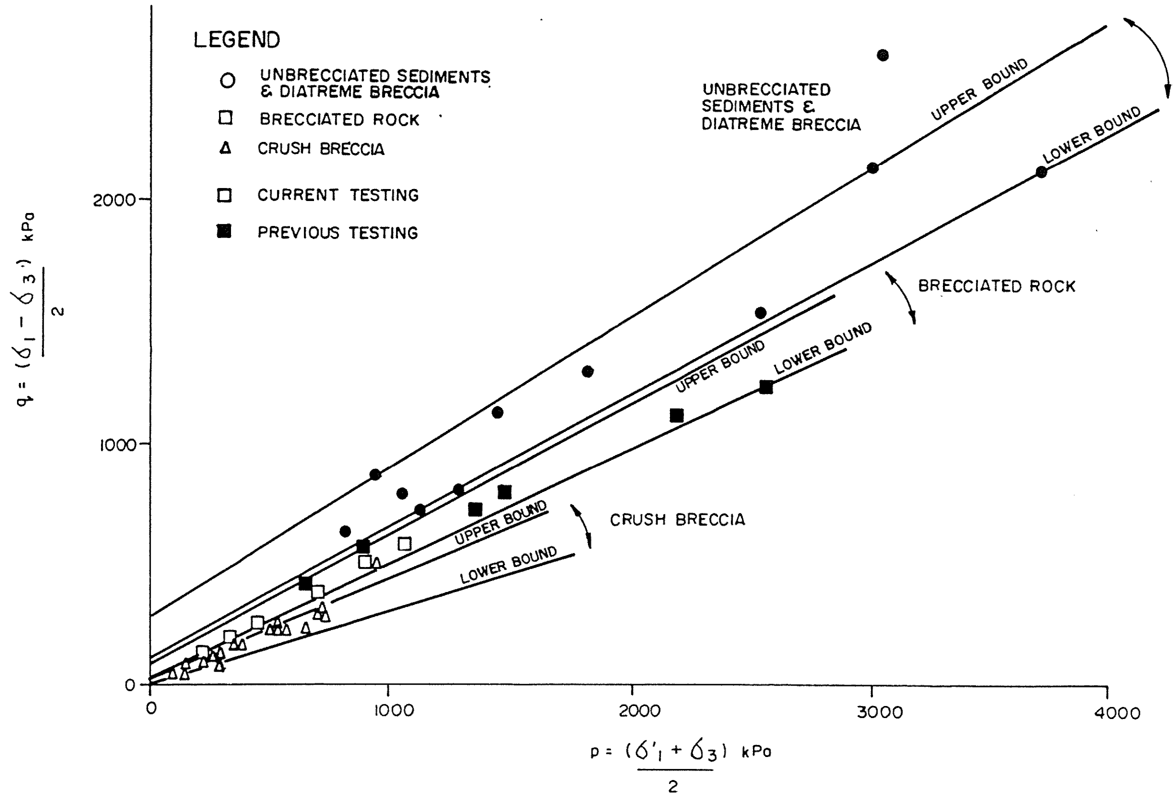


Figure 3. Results of triaxial shear strength testing.

Slope experience in interim pit exposures also indicated the potential for larger defects to play a role in slope failure. As such strength properties of unfavourably oriented defects in unbrecciated units were assessed.

3.1 Triaxial Shear Strength

Triaxial shear strength is relevant due to the potential for circular failure through the rock mass. For a soil/rock mass which is massive or where the size of the brecciated fragments in the test sample are less than about 1/40 of the sample size the triaxial strength can be taken as representative of the mass strength. In other rock masses the interpretation of strength is more problematic and dependent on the rock mass defects and rock substance.

A total of 18 samples were tested as follows:

- 5 sediments,
- 2 diatreme breccia,
- 7 brecciated rock, and
- 4 crush breccia.

The results were presented on a p-q plot (Figure 3) and upper and lower bounds assigned. Results for sediments and diatreme breccia are comparable and were combined for the purposes of assessing triaxial strengths.

Results for brecciated rock are summarised in Table 1. Mean strengths were estimated by linear regression for a stress range up to 1 MPa. An estimate of the standard deviation was made from the upper and lower bounds. An assessment is required on how many (n) standard deviations is represented by the upper and lower bound range based on the number of data points defining the bounds. For this case each range is defined by between 10 and 20 points and was assessed to be about 4 (n=4) standard deviations of the population.

Table 1. Triaxial shear strength results for brecciated rock.

PARAMETER	COHESION c (kPa)	FRICTION ANGLE ϕ (degrees)
Upper Bound	95	33
Lower Bound	36	28
Mean	78	28.7
Std Dev	15	1.7

In this model c' and ϕ' are assumed to be independent, this is generally a conservative assumption (Yu and Mostyn, 1994).

There are more exact procedures for estimating variability of linearly regressed data, these were considered inappropriate in view of the overall method of analysis. The statistical parameters discussed above are intended to represent point estimates. Spatial correlation is discussed later in this paper.

3.2 Hoek Analysis

Recognising the limitations of testing, the GSI method was used to predict rock mass strengths as described by Hoek et al (1995). Equivalent cohesion and friction parameters are estimated from the Hoek-Brown criterion using relationships between GSI and the material constants.

GSI values were assessed from Bieniawski's 1989 Rock Mass Rating (RMR_{89}) classification. Evaluation of RMR_{89} for the purpose of assessing rock mass strength parameters requires a rating of 15 be used for Groundwater and zero to be assigned to the Adjustment for Joint Orientation. Ratings are applied to the following parameters:

- uniaxial compressive strength (UCS),
- rock quality designation (RQD),
- defect spacing, and
- condition of defects.

The final rating, RMR_{89}' is used to estimate GSI:

$$GSI=RMR_{89}'-5$$

Data was available for two separate borehole programs drilled in 1990-1992 and 1996. RMR_{89}' was assessed for each rock mass type by separately calculating RMR_{89}' for each logging interval down the borehole. Mean and standard deviation GSI were calculated from weighted RMR_{89}' parameters based on the relative length of the logging interval.

As an example, for brecciated rock mean GSI values of 51 and 47 were obtained from the 1990-1992 and 1996 data bases with standard deviations of 6 and 5, respectively.

When using the Hoek analysis it is assumed the model is a good predictor of strength. Thus the variability associated with the GSI model is taken to be zero. It is assumed that the variability in c' and ϕ' derived for a Hoek-Brown GSI model includes variability in the model itself and in the estimate of GSI. These assumptions could be varied if desired.

It is assumed that the logging and test results are such that an estimate of the strength at any point is a reasonable estimate of the strength along the failure surface in that logging unit and that the strength in any one logging unit was independent of the strength in adjacent units. This is equivalent to assuming that the scale of fluctuation of the GSI is less than but close to the length of the logging units, see for example Yu and Mostyn (1993).

Mean strengths are taken from the Hoek analysis of mean GSI and mean UCS. Standard deviation of c' and ϕ' were estimated using a point estimate method (Harr, 1987).

Based on trial distributions of c' values the logNormal distribution was used for cohesion . An assessment of the GSI and UCS parameters in the Hoek analysis found they are generally poorly correlated particularly for low intact strength materials. As such the bivariate point estimate method (Harr 1987) was used to estimate the standard deviations for strength parameters, c' and ϕ' obtained from the Hoek analysis.

3.3 Comparison of Strength Parameters

A comparison of mean strength parameters derived from triaxial testing and the Hoek analysis is presented in Table 2 using brecciated rocks as an example.

Table 2. Comparison of mean strength parameters for brecciated rock.

DATA SOURCE	COHESION c (kPa)	FRICTION ANGLE ϕ (degrees)
Triaxial shear strength testing	78	28.7
1990-1992 GSI parameters	59	30.2
1996 GSI parameters	52	29.2
Back analysis of failures	60	33

Good agreement was obtained for brecciated rock and crush breccia where defects have less affect on the mass strength. However, for the higher strength sediments and diatreme breccia there is a larger difference with triaxial testing biased by sampling of very low strength materials.

As a result the Hoek derived strength parameters were adopted for design apart from crush breccia where the triaxial results were used. Selection between the 1990-1992 and 1996 GSI results was based on relative size of the respective databases and using the triaxial test results and back analyses as a guide.

Typical design parameters are listed in Table 3, again using brecciated rock as an example.

Table 3. Design strength parameters for brecciated rock

PARAMETER	COHESION c		FRICTION ANGLE ϕ (degrees)
	NORMAL DISTRIBUTION (kPa)	LOG NORMAL DISTRIBUTION	
Mean	52	1.716	29.2
Standard Deviation	13	0.1294	6

3.4 Defect Strengths

Direct shear strength testing was undertaken on seven samples of crushed or sheared zones between 10 and 55mm thick infilled with stiff to hard clay and rock fragments. Results were plotted on normal versus shear strength graphs and upper and lower bounds applied. Results were:

- upper bound c'=48kPa, ϕ'=22°
- lower bound c'=20kPa, ϕ'=19°
- mean c'=28kPa, ϕ'=21°.

Mean and standard deviation parameters were estimated using the same techniques as for triaxial shear strength testing.

Back analysis of a previous failure involving sliding along a sheared zone in sediments resulted in mean strength parameters of:

- c'=28kPa,
- ϕ'=22°.

This provided good field verification of the direct shear results.

4. SLOPE STABILITY ANALYSIS

4.1 Potential Failure Mechanisms

A fundamental tenet in assessing slope stability is a good understanding of the basic mechanics before applying any analytical techniques. As stated by Hudson & Harrison (1997):

> *"the basic mechanics must be understood first and then any variations in any properties, or any lack in our knowledge, dealt with via appropriate mathematical techniques." (p. 331)*

Knowledge of the geological, geotechnical and structural conditions, rock mass and defect strengths, plus the slope experience from interim slopes, indicates that the primary mechanism controlling maximum overall slope angles is the potential for rock mass failure. Previous failures and structural conditions also indicate the potential for combined defect and mass failure. This will have the greatest impact in the higher strength units.

4.2 Sources of Variability

Potential variability in the stability model will be due to model geometry, strengths and groundwater levels.

Model geometry comprises the geological model including distribution of units, contacts and structure. In this case the model was based on multiple data sources including drilling, mapping and Vulcan computer modelling. On this basis it was assumed the model is well known and differences between the model and actual conditions are unlikely to contribute significant variability to the problem.

High groundwater levels are used in the analyses as it was considered unlikely that significant depressurisation will be possible. This is particularly so in the brecciated rock and crush rock units within the time frame allowed by mining. The key assumption is that high groundwater levels will occur during some stage of exposure of the slope. Groundwater is therefore not a probabilistic variable in this case.

The main source of variability in the analysis is therefore mass and defect strength.

4.3 Deterministic Analyses

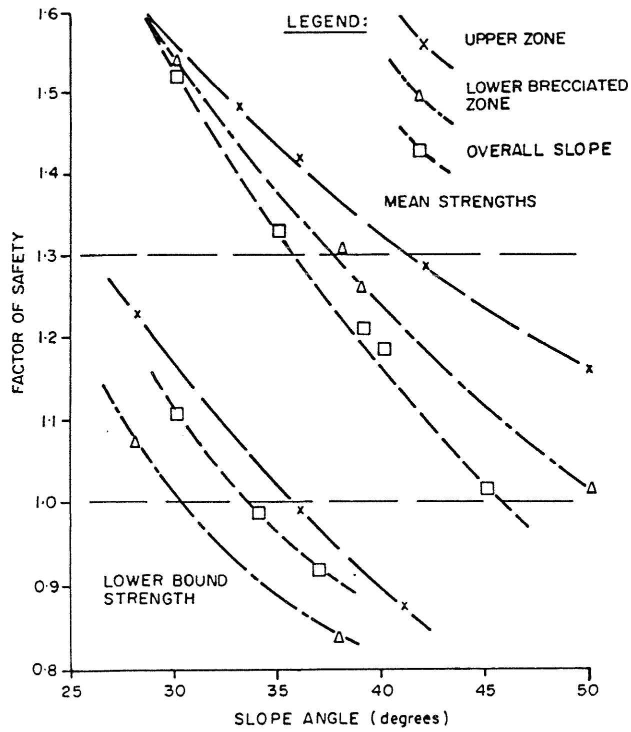
Limit equilibrium analyses were carried out initially using Bishop's method of slices. The objective was to assess the slope angle versus Factor of Safety (FoS) relationship for mean and lower bound strengths for a range of different slope heights. Lower bound strengths for these analyses were taken as the mean value minus two standard deviations for both c' and ϕ'.

An example of the results for one section analysed is shown in Figure 4 together with a simplified version of the model analysed.

The limit equilibrium results identified the critical slope segment for probabilistic analysis. In most cases the slope segment containing the thrust fault zone returned the lowest FoS. In the example shown in Figure 4 which is a section through the rhyolite intrusive body the overall slope returned the lowest FoS.

4.4 Probabilistic Analyses

The probabilistic analysis was carried out using the Monte Carlo technique. In this method c' and ϕ' are selected at random from their distributions. The process is repeated many times to generate a distribution of FOS which is used to estimate the probability of failure.



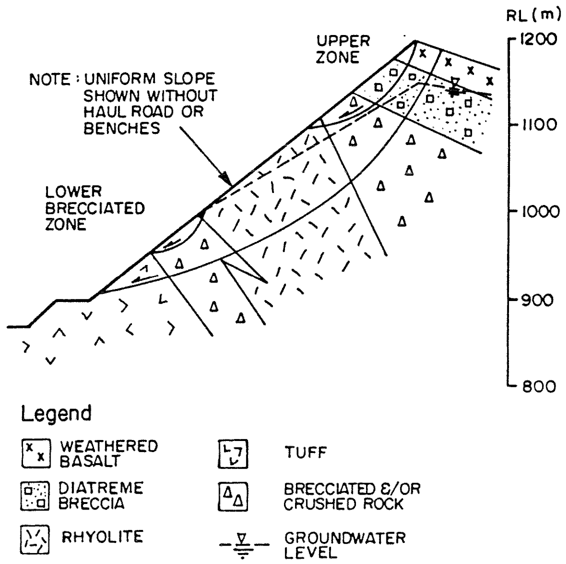


Figure 4. Results of deterministic analyses for section K781

The critical slope segment identified by the deterministic analyses was analysed. A total of 1000 stability iterations were used for each case and the analyses adopted Bishop's method of slices.

Stability of the fault zone sequence will largely be controlled by the strength developed in the stronger "unbrecciated" units. In these units it is considered two potential modes may operate:

- failure through the mass, and
- failure along defined defects.

It was considered that within each unit failure would occur either through rock mass or along a defect. Thus the overall probability of failure is:

$$Pf_{total} = \Sigma[Pf_{defect} \times Pd_{event}] + Pf_{mass} \times [1-\Sigma Pd_{event}] \quad (1)$$

where:

Pf_{total} total probability of failure
Pf_{defect} probability of failure given that adversely oriented and located defects occur in one or more units
Pd_{event} probability of adversely oriented and located defect occurring within a unit or units
Pf_{mass} probability of failure through rock mass in all units without defects.

The probability of an adversely oriented and located defect was assessed from the structural domain mapping database using the following expression:

$$Pd_{event} = Pd_{envelope} \times Pd_{length} \quad (2)$$

where:

$Pd_{envelope}$ probability of defect occurring in an envelope defined by the slope direction and dip range of the failure path
Pd_{length} probability of defect continuity exceeding the length of failure path in slope segment

Review of the structural domain data indicated faults and shears were the most critical defects due to their continuity and orientation (Figure 5). Geological mapping information indicated these structures could be expected throughout the sediments and diatreme breccia units in the vicinity of the thrust fault. Pd_{event} values in the range 0.1% to 4.0% were

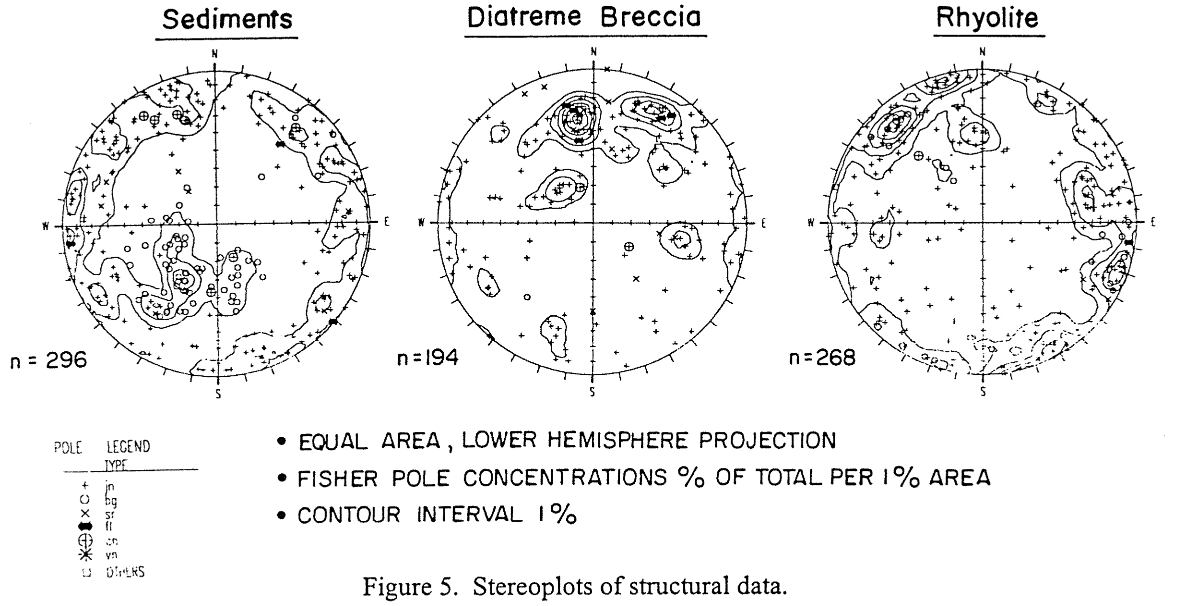


Figure 5. Stereoplots of structural data.

assessed for faults and shears in unbrecciated zones in the sediments and diatreme breccia.

Initially, the Pf_{mass} for rock mass strengths in all layers was determined. Then Pf_{defect} was determined using defect strengths in unbrecciated zones. An example of the results is shown in Figure 6. In this example a third run was required for each case with defect strengths in both the upper and middle unbrecciated zones.

Probabilities of failure (total) in the range 0.2 to 0.7% were adopted for design. These probabilities were based on the shape of the Pf curve, position of the haul road on the wall, the need to maintain consistent overall angles which avoid sudden changes in slope and judgement based on experience and knowledge of the geotechnical conditions.

5. CONCLUSIONS AND RECOMMENDATIONS

Probabilistic analysis is a valuable tool in the slope stability assessment of a rock mass which is characterised by variable geotechnical conditions within a complex geological setting. Using a probabilistic approach allows the variability in important parameters to be modelled and compels the user to consider assumptions on model uncertainty.

Assessment of the variability in rock mass strength can be problematic. This can be accomplished by comparing laboratory strength testing, Hoek analysis of GSI values derived from borehole logs and back analysis of failures. Such an assessment should always be undertaken within the context of a good understanding of the geological and geotechnical conditions. It is also important that the spatial correlation of strength is carefully considered.

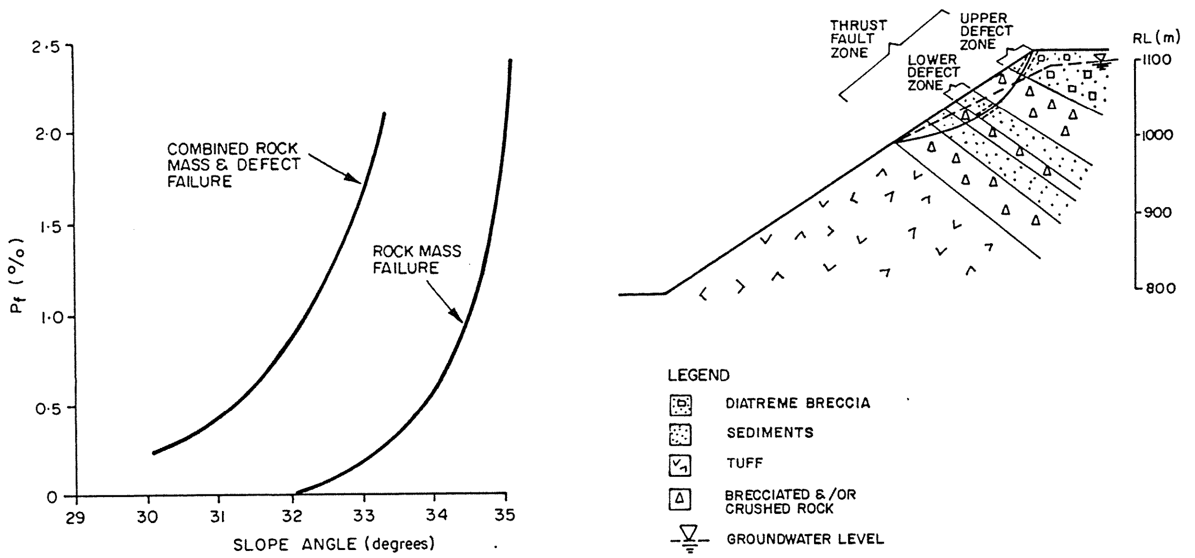


Figure 6. Results of probabilistic analyses for section K779.

Fundamentally any probabilistic analysis will only be valid if the basic mechanics have been properly understood first. In the case presented the probability of failure was estimated for a failure mechanism involving a combination of defect and rock mass failure.

There are a number of areas where refinements in the method can be made for future use.

- Assessment of model error, particularly for the Hoek analysed strength distributions.
- Consideration of log interval lengths used to assess Hoek strengths compared with the failure path intersection length, thus a more detailed view of spatial correlation.
- Examine the assumption that the critical failure path can be correlated with the whole slope.
- Incorporate groundwater as an unknown variable.

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