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An application of a reliability based method to evaluate open pit slope stability

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Abstract

The stability of open pit rock slopes is commonly assessed using deterministic methods. A single value for the Factor of Safety of the slope is calculated, and is assumed to represent the overall stability of the slope. A limitation of the deterministic approach is that it does not account for the natural variability of the input parameters or the uncertainty caused by sampling. The uncertainty and variability of input parameters such as frictional strength, cohesive strength, and discontinuity orientations are often accounted for by selecting values that incorporate conservatism based on limited laboratory testing or other data collection methods. The confidence in the Factor of Safety calculated using deterministic analyses remains a matter of engineering and experiential judgment.

Reliability based methods, now adopted by many practitioners, overcome some of the limitations of deterministic methods by incorporating the natural variability of key input parameters into the calculation methods. If the natural variability is included, then a range of Factors of Safety can be calculated for the various combinations of the input variables. This range represents the probability density function of the continuous distribution of Factors of Safety, from minimum to maximum values. The cumulative distribution of values can then be used to quantitatively describe the likelihood of achieving a certain Factor of Safety. Understanding the distribution of Factors of Safety, and the reliability of the results, provides a method to make reliability based decisions.

This paper presents a feasibility level study undertaken for Diavik Diamond Mines Inc. using point estimation methods in combination with limit equilibrium methods to evaluate the Factor of Safety of a proposed open pit slope. Point estimation involves the use of sample statistics to estimate population parameters. Confidence intervals are constructed that include the value of an unknown variable—in this case Factor of Safety—with high probability. The confidence intervals are developed based on the method of moments from probability theory, which is used to describe random variables of the sample population using the expected value of the random variable and the square of the expected value. A Monte Carlo random sampling method is used to develop a simulated population to approximate a normal distribution, which in this case represents the probability of slope performance defined in terms of the Factor of Safety. This paper describes the process followed to develop the probability density and cumulative distribution functions for the Factor of Safety. The statistical distribution of Factor of Safety is presented and is used to evaluate the pit slope stability in terms of probability of failure and acceptable risk tolerance.

1 Introduction

Diavik Diamond Mines Inc. (Diavik) is approximately 300 km northeast of Yellowknife in the Northwest Territories, Canada. Diavik currently has developed the A154 and A418 open pit mines, and is proposing to mine the A21 kimberlite pipe, located approximately 3.5 km to the southwest of the A154 and A418 project areas. The A21 kimberlite pipe intruded into tonalite country rocks, which are igneous plutonic rock. The tonalite is typically strong rock of good rock mass quality.

The A21 kimberlite pipe is overlain by Lac de Gras. An option being considered for mining of the kimberlite is the construction of a dike around the mining area to allow dewatering of the area enclosed by the dike. Under this option, the close proximity of certain sectors of the pit wall to the proposed dewatering dike has resulted in a need to quantify the limit equilibrium Factors of Safety of those sectors in terms of probability. This task has been accomplished using a reliability based approach and point estimation methods. The focus of the study is the southeast wall of the proposed A21 pit, which has been determined to be the most critical based on the results of deterministic limit equilibrium analyses.

2 Approach

This study has used standard methods for evaluating slope stability and developing design criteria. These methods involved the following studies: oriented geotechnical data were collected and interpreted, kinematic and cumulative frequency analyses were completed to develop bench scale design criteria, hydraulic conductivity testing was carried out in geotechnical boreholes and the data used to develop a hydrogeological model, and deterministic limit equilibrium models that incorporated groundwater pressures from the hydrogeological model were analysed. Point estimation methods were then used to develop a Factor of Safety probability density function to quantify the Factor of Safety for a specific pit wall.

2.1 Deterministic versus reliability approaches

Typically, geotechnical studies describe the stability of a slope in terms of a Factor of Safety (FS). The FS approach applies deterministic solutions to the slope stability problem, resulting in a single value for the Factor of Safety that represents the overall stability of the slope. Input parameters for slope stability analyses commonly include cohesive and frictional strength parameters, and discontinuity orientations. The deterministic solution does not account for the natural variability of the input parameters, and the true Factor of Safety (or the probability attached to a range of Factors of Safety) for a slope is never known.

Deterministic methods account for natural variability of the input parameters by selecting parameters that incorporate levels of conservatism. For example, a conservative value of cohesion equal to 0 kPa is often assigned to joint surfaces; in reality, cohesive strength is likely to have a probability distribution where 0 kPa may represent an insignificant proportion of the overall data set. In the same way, conservative inputs for joint frictional strength and failure plane orientations may also be selected. The resulting deterministic solution may be overly conservative, as the conservatism in the assumptions for the input parameters is accumulated. Furthermore, the deterministic approach does not take into account that the probability of the combined occurrence of ‘worst case’ values is expected to be low.

To address the conservatism of the deterministic modelling approach, sensitivity analyses are carried out by varying each significant input parameter over its maximum reasonable range, while holding others constant, to evaluate the influence of that parameter on the Factor of Safety. The sensitivity of the outcome to variability in each parameter can therefore be determined. A range of Factors of Safety is developed, but the probability density function describing the Factors of Safety is not explicitly defined. Thus, the confidence in the deterministic Factor of Safety remains a matter of engineering judgment and experience.

An alternative method for assessing the stability of a slope involves the use of point estimation as a form of statistical inference (Harr, 1989; Rosenblueth, 1975). The point estimation method uses the statistical distributions for each key input variable to define confidence intervals that bound the true value of the variable. A probability density function of pit slope Factor of Safety is then developed. Understanding the distribution of Factors of Safety, and the reliability of the results, provides a method for assessing slope performance and for making reliability based decisions.

3 Performance and acceptability criteria

3.1 Acceptance criteria for deterministic solutions

A commonly accepted Factor of Safety criterion for overall pit slope stability is 1.3. This criterion forms the acceptance criterion for the stability assessment using reliability methods. Slope performance is evaluated in terms of achieving or exceeding a minimum Factor of Safety of 1.3 for overall pit slope stability, rather than evaluating the probability of slope failure defined by a Factor of Safety less than 1.0.

3.2 Acceptance criteria for reliability based solutions

There are currently no clearly defined acceptance criteria for the probability of unsatisfactory slope performance. Selected literature on this topic is briefly summarised below, and describes acceptable risk in terms of probability of failure, or probability of a Factor of Safety of less than 1.0.

Khalokakaie et al. (2000), with reference to McCracken (1983), proposed the acceptable risks defined in Table 1 when probabilistic methods are used in slope design.

Table 1 Maximum acceptable risk of failure for use in mining and civil works when probabilistic methods are used in slope design

Conditions	Stand-up Time	Probability of Failure	Percentile
Permanent civil works	Very long	<0.005	>99.5
Civil works	Long	0.005–0.015	98.5–99.5
Small open pit/quarry final slopes	Medium–long	0.015–0.05	95–98.5
Open pit final slopes	Medium	0.05–0.15	85–95
Large open pit final slopes	Short	0.15–0.30	70–85
Temporary benches	Very short	0.30–0.50	50–70

Ref: Modified from Khalokakaie et al. (2000)

Terbrugge et al. (2006) provide an informative discussion and approach to the evaluation of open pit slope design using risk consequence methods. The study cites Steffen's (1997) recommendation that an acceptance level of approximately 5% for the risk of loss of equipment be used. Terbrugge et al. (2006) present data indicating that the expected range in annual probability of failure for mine pit slopes is between 1% and 10%. These ranges are comparable to acceptable risk criteria presented by Khalokakaie et al. (2000) for small quarry, small open pit, and large open pit final slopes with medium to medium-long stand-up time requirements (Table 1).

Based on the current literature review, it appears that probabilities of failure of between 1% and 10% constitute generally acceptable risk criteria when combined with an understanding of important site−specific contributing variables and a suitable monitoring program to manage risks to personnel.

4 Input parameters to deterministic and reliability methods

The mechanism for slope instability at the A21 pit area consists of bi-linear wedge failure. This failure mechanism is shown schematically in Figure 1.

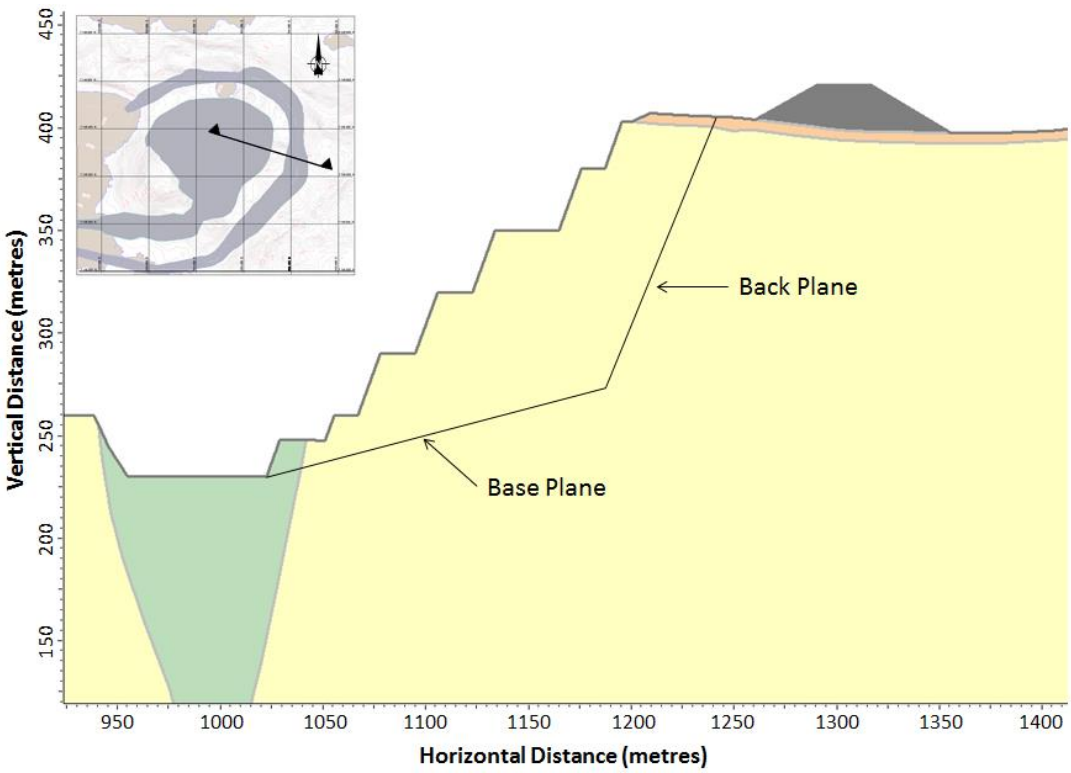

Figure 1 Schematic of bi-linear wedge failure mechanism

4.1 Identification of the key variables

Reliability based design requires the identification of key variables that will influence the outcome of the stability analyses. For the purposes of the current study, the following four variables were identified as the main variables to be considered in the assumed bi-linear wedge failure illustrated in Figure 1:

- The frictional strength of the base and back planes (ϕ).
- The cohesive strength of the base and back planes (c).
- The angle of inclination of the base plane (θL).
- The angle of inclination of the back plane (θH).

For the purposes of the two-dimensional analyses upon which the reliability assessment is based, resistance to sliding from the side surfaces of the wedge (i.e., the release planes), is assumed to be negligible.

A numerical hydrogeological model developed in MODFLOW-2005™ (United States Geological Survey, 2005) for the site was used to predict the hydraulic heads and water pressures within the walls of the A21 Pit. The groundwater pressures were incorporated into the limit equilibrium models as a pore pressure grid.

Rock mass strength was not considered to be a key variable for the probabilistic analyses, and was considered deterministically. This is because experience at the site has shown that the rock mass strength of the tonalite host rock does not vary considerably within the project area. Furthermore, the rock mass quality of the tonalite is good, with an average Geological Strength Index (GSI) rating of 73. Therefore, overall slope failure mechanisms are expected to be structurally controlled, and not related to rock mass strength.

The input parameters to the deterministic and reliability methods are based on the geotechnical data collected from the borehole drilling programs and from the geotechnical mapping of an exploration decline driven to collect a bulk sample. The simulated probability distributions discussed in the subsequent sections

are based on the borehole and mapping data, and have been generated using the program Crystal Ball™ (Oracle, 2013) using a Monte Carlo random sampling method on 50,000 trials. Crystal Ball is a Microsoft Excel™ add-in for carrying out complex 'what-if' simulations.

4.2 Probability distribution for inclination of the base and back planes

The lowest deterministic Factor of Safety in a limit equilibrium model typically corresponds to sliding along the maximum inclination of the base plane, if a range of possible inclinations for a dominant joint set has been specified. However, this inclination represents typically only the outliers of the data set. Joint set orientations representing a potential base sliding plane were collected during the A21 geotechnical drilling investigations. The simulated probability distribution for the base plane inclination is shown in Figure 2. In the figure, positive values indicate planes that dip into the wall, and negative values indicate planes that dip out of the wall.

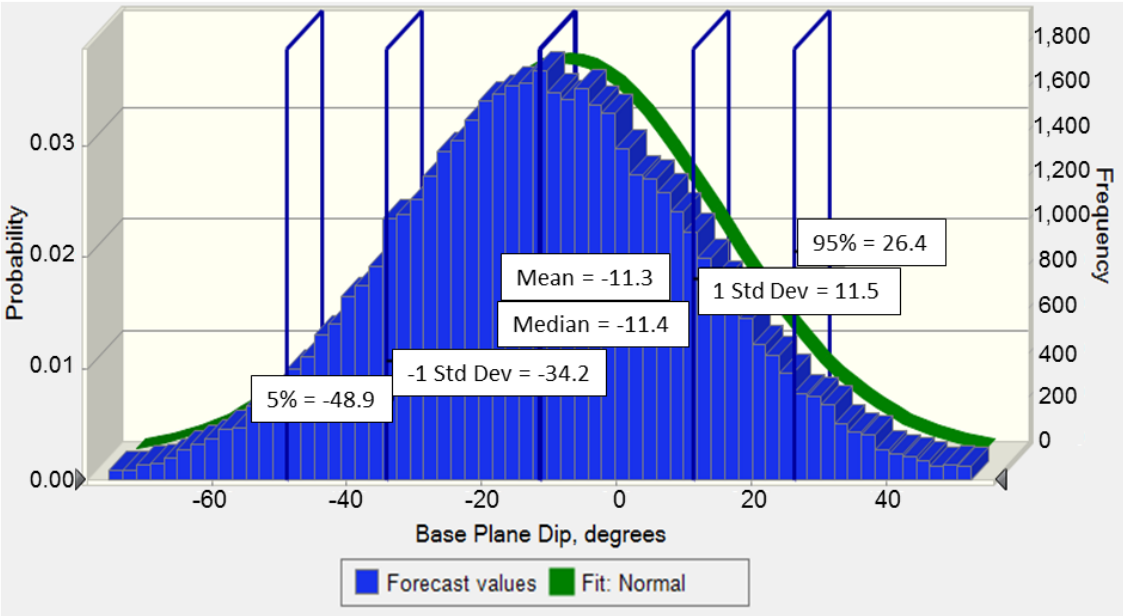

Figure 2 Base sliding plane inclination distribution based on simulated population

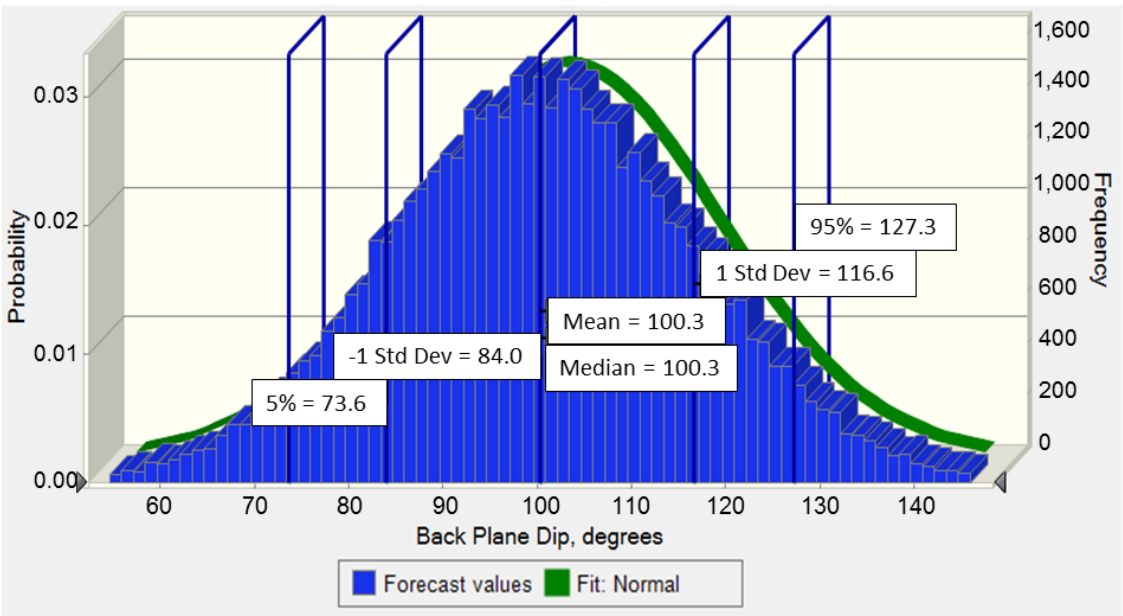

Figure 3 Back plane inclination distribution based on simulated population

The deterministic limit equilibrium analyses also indicate that the stability of the slope is less dependent on the back plane dip, assuming that sliding along a base plane requires some sort of back release plane. Slope stability is less dependent on the back plane dip because the steep inclination of the back plane contributes very little to the shear strength of the overall failure surface and the tensile strength of the rock mass is assumed to be near zero. Nonetheless, the distribution of the back plane inclination is important in defining the shape of the overall failure surface. The simulated probability distribution for the back plane inclination is shown in Figure 3. In the figure, inclinations ranging from 46 to 90° indicate planes that dip into the wall, and inclinations ranging from 90 to 134° indicate planes that dip out of the wall.

4.3 Probability distribution for joint continuity

Step path failures form through a combination of plane shear along existing joint surfaces and shearing through the intact rock in the absence of joint surfaces. Joint continuity and rock bridge length have been evaluated from mapping of continuity data from the A21 decline and reviewing photographs of structures observed in the decline.

The simulated distribution for joint continuity is shown in Figure 4, and fitted with a lognormal distribution. A limitation of the continuity data collected from underground mapping results from the truncation of joints by the back and floor. This limitation could lead to an underestimation of joint continuity. However, the lognormal distribution is consistent with studies by Palmstrom (1995) and others on the statistical distribution of joints, specifically spacing and trace length.

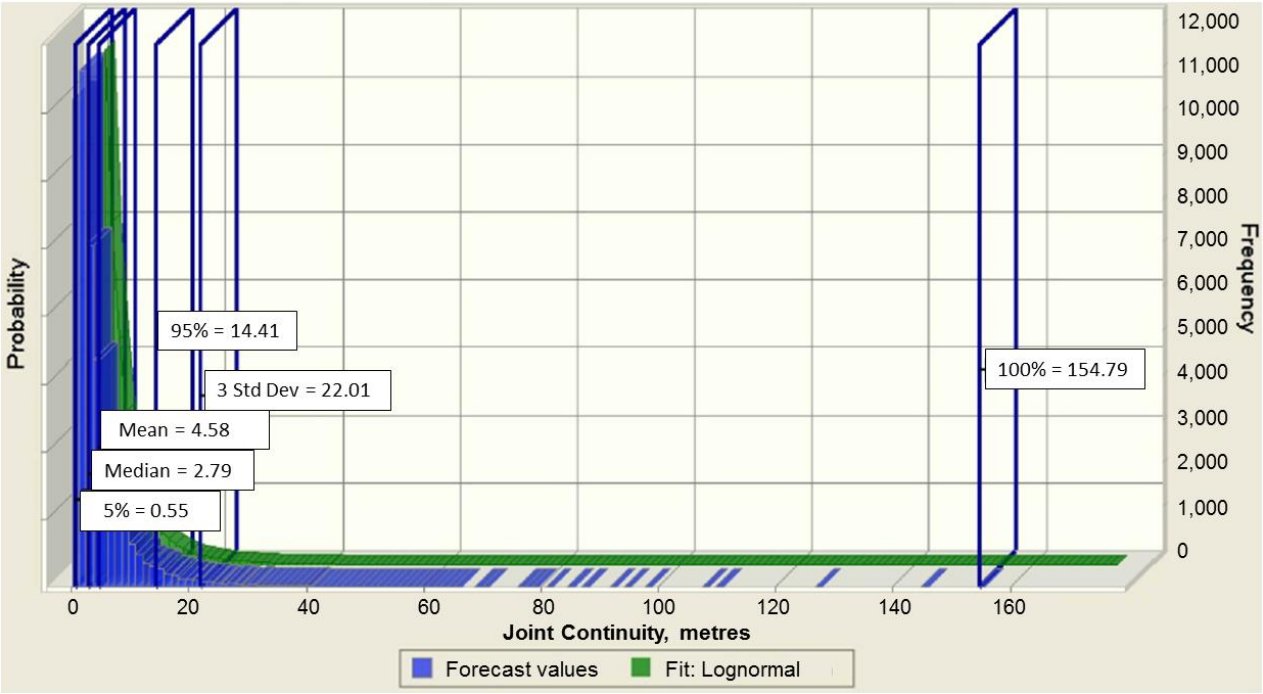

Figure 4 Joint continuity distribution based on simulated population

4.4 Probability distribution for rock bridge

The presence of intact rock between major co-planar joints (i.e. rock bridges) in a rock mass imparts a cohesive strength to step path failure modes. Estimates of rock bridge length were developed from mapping data and from photographs of the sub-horizontal structures observed in the A21 decline. The simulated distribution for rock bridge length is shown in Figure 5.

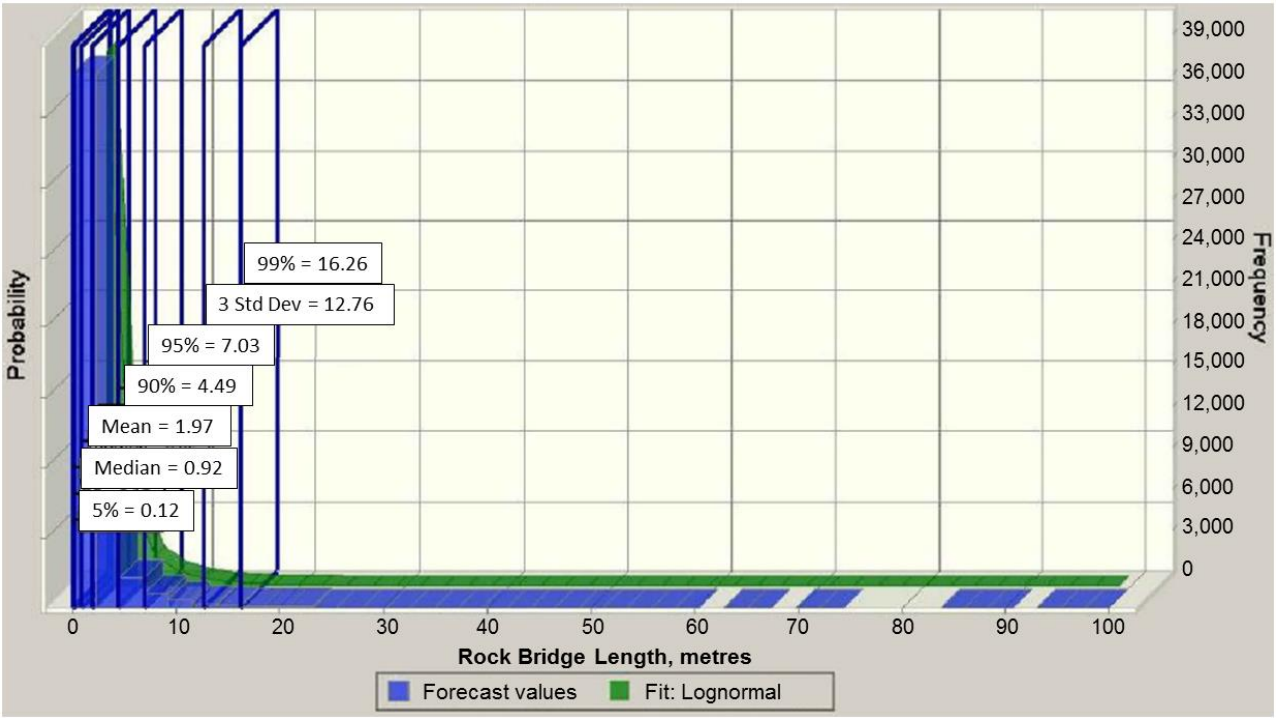

Figure 5 Rock bridge length distribution based on simulated population

The joint continuity and rock bridge length data were used to develop estimates of rock bridge percentage, based on the following equation (Jennings, 1972):

$$\%\,Rock\ Bridge = \left(\frac{Rock\ Bridge\ Length}{Rock\ Bridge\ Length + Joint\ Continuity}\right) \times 100\% \tag{1}$$

The simulated distribution for rock bridge percentage is shown in Figure 6. Descriptive statistics were developed for the simulated rock bridge data sets, to be used in the subsequent reliability assessments.

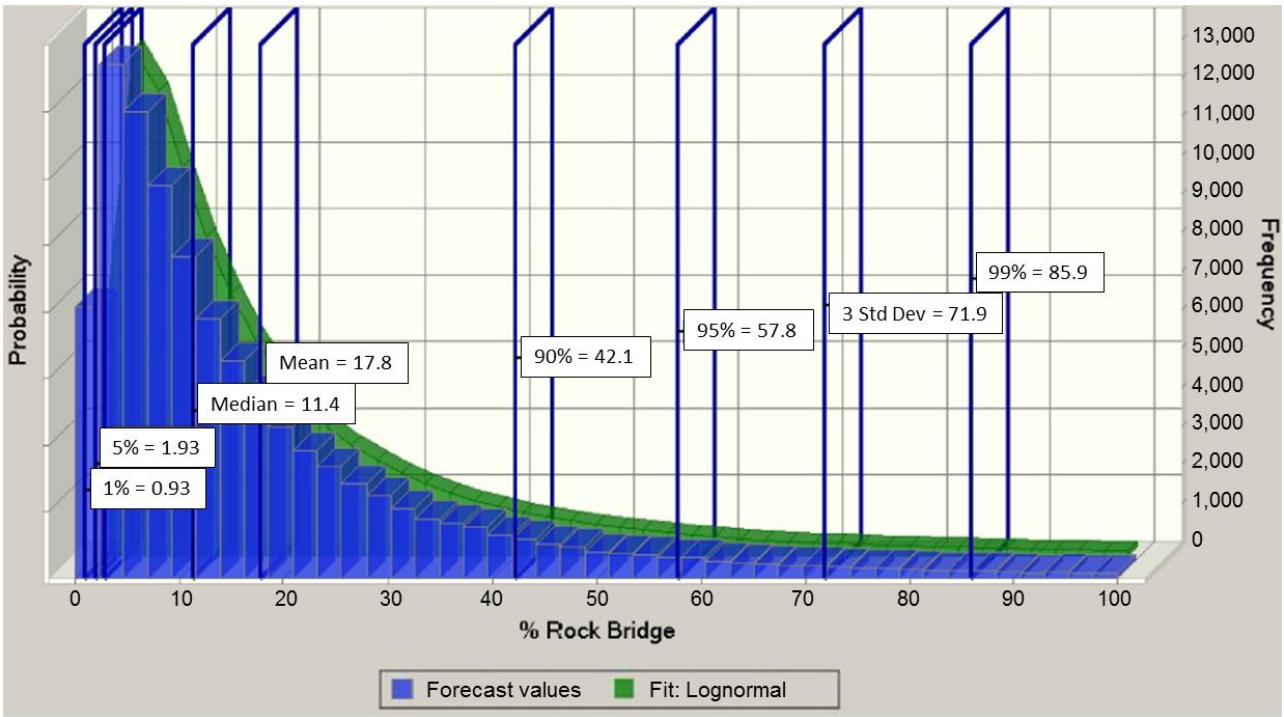

Figure 6 Rock bridge percentage distribution based on simulated population

4.5 Probability distribution for cohesion

Rock bridges contribute additional cohesive strength to potential shear surfaces. The magnitude of the additional strength depends on the percentage of intact rock that must be sheared through by a potential failure surface. The joint cohesion due to the presence of rock bridges is estimated by the following equation (Jennings, 1972):

$$Cohesion = (\%\ Rock\ Bridge) \times Rock\ Mass\ Cohesion + (1 - \%\ Rock\ Bridge) \times Joint\ Cohesion \quad (2)$$

A joint cohesion of 70 kPa was used in the analyses, based on peak shear strength data obtained from direct shear testing. The rock mass cohesion was estimated using the RocData program (Rocscience Inc., 2013) to develop Mohr–Coulomb parameters based on curve fitting of non-linear Hoek–Brown criteria for rock mass strength (Hoek et al., 2002). The analysis was undertaken for a range in normal stresses associated with a 100 m high overburden stress, which is the estimated maximum depth of a failure surface within the A21 pit wall. Total stresses were used for fitting the curved Hoek–Brown envelope in recognition that the critical slopes will be depressurised. The rock mass strength parameters for the southeast area of the A21 project are summarised in Table 2.

Table 2 Summary of rock mass strength parameters for the A21 east domain

GSI	Maximum Depth of Failure Surface (m)	UCS (MPa)	D	m_i	Cohesion (kPa)	Friction Angle (deg)
73	100	146	1	33	2,182	62

The simulated distribution of rock bridge cohesion is shown in Figure 7.

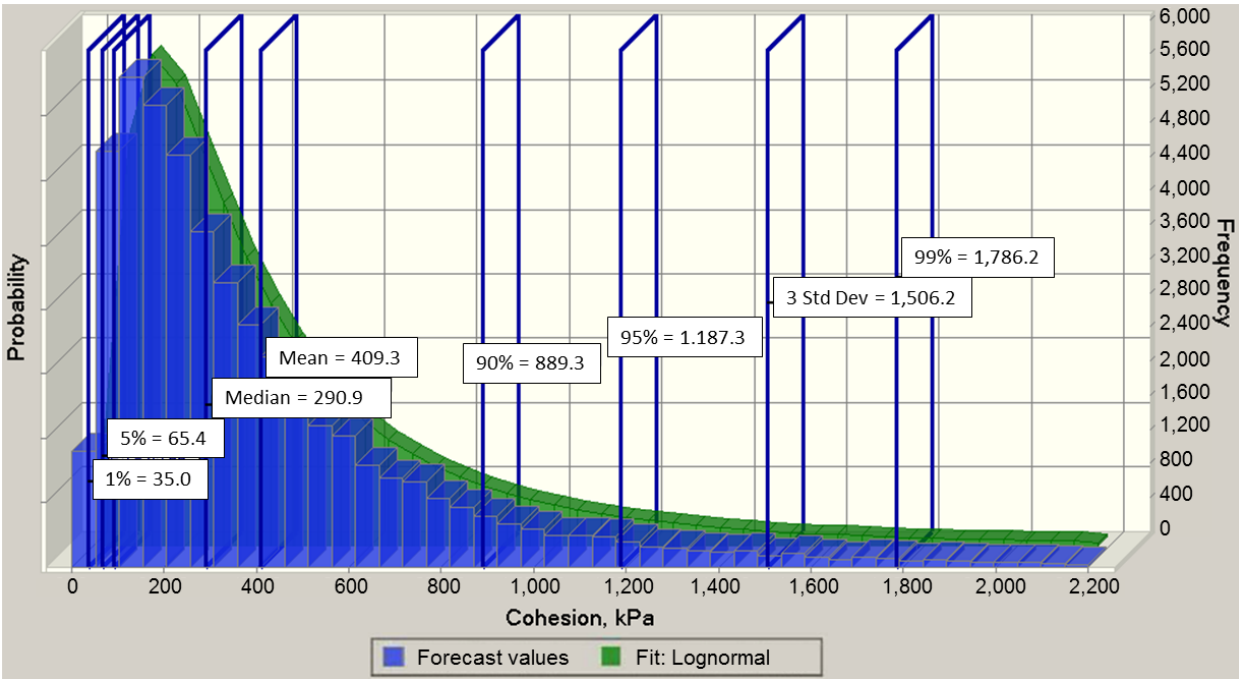

Figure 7 Cohesion distribution based on simulated population

Descriptive statistics were developed for the simulated data set, to be used in the subsequent reliability assessments.

4.6 Probability distribution for frictional strength

Joint roughness coefficient (JRC) data were collected from joint surfaces during geotechnical mapping of the A21 decline. The simulated distribution of JRC values is shown in Figure 8.

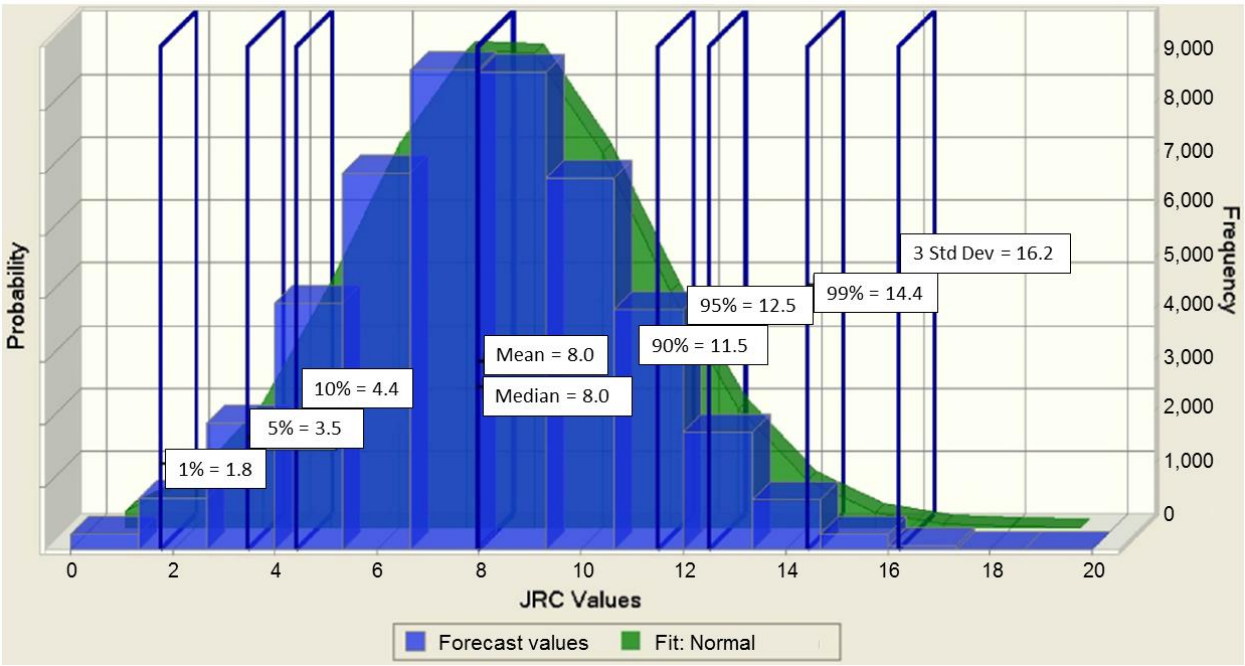

Figure 8 JRC distribution based on simulated population

The JRC values are related to peak friction angle based on the following relationship developed by Barton and Choubey (1977):

$$\phi_{peak} = \phi_{basic} + JRC \times log_{10}\left(\frac{JCS}{\sigma_n}\right) \tag{3}$$

Where:

ϕ_{peak} = peak friction angle.

ϕ_{basic} = basic friction angle.

JRC = joint roughness coefficient.

JCS = joint compressive strength.

σ_n = normal stress.

The JRC data collected during the A21 decline mapping were used to develop a simulated data set representing a range of expected values of peak frictional strength. The lowest residual friction angle determined from laboratory shear strength testing of joint surfaces, 29°, was adopted as the basic friction angle, ϕ_{basic}, to develop the probability distribution. The calculation of peak friction angle was evaluated for an average normal stress of 1.2 MPa across potential failure surfaces, as determined from limit equilibrium analyses for the southeast wall. An average joint compressive strength of 146 MPa was assumed, based on laboratory strength testing data on the country rock. The simulated distribution of friction angle is shown in Figure 9.

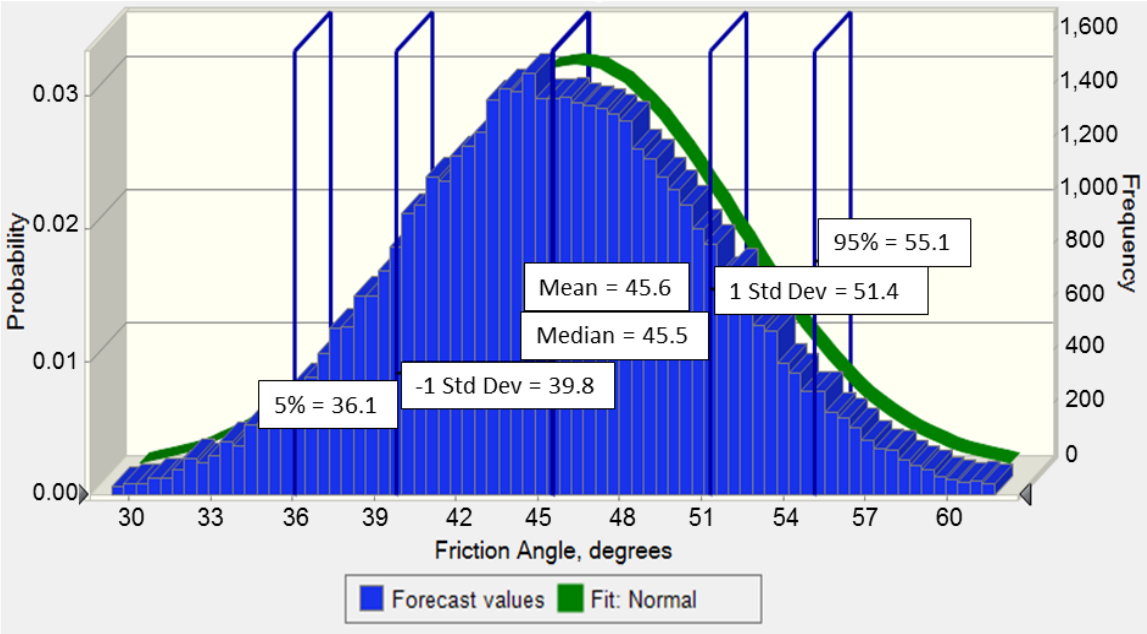

Figure 9 Friction angle distribution based on simulated population

5 Evaluating the performance of the southeast wall

The simulated probability distributions developed in the preceding sections were used to develop input parameters for the point estimation method. The estimations of the probability of unsatisfactory slope performance that follow are based on methods presented by Thornton (1995) and on point estimation theory (Harr, 1989; Rosenblueth, 1975).

5.1 Point estimation theory

Point estimation involves the use of sample statistics to estimate statistical characteristics of the larger population, such as means or proportions. The characteristics of the sample distribution become estimators of the characteristics of the larger population distribution. Confidence intervals are constructed that include the value of an unknown parameter, in this case Factor of Safety, with a known probability. Central limit theory recognises that for a sampling distribution, the unknown parameter can be approximated even if the actual distribution of the overall population that is being sampled is unknown. The expected value of the parameter, in this case Factor of Safety, can then be developed once the key variables that influence the expected value are identified. This process is described below.

5.2 Distribution of joint strength and plane inclination values

Variation within the independent variables is incorporated into the analyses through addition or subtraction of the standard deviation to the mean value. The following equations summarise the distributions:

Cohesion:

$$C_+ = \mu_C + \sigma_C \tag{4}$$

$$C_- = \mu_C - \sigma_C \tag{5}$$

Where:

μ_c = mean value of cohesion.

σ_c = standard deviation of cohesion.

Friction angle:

$$\phi_+ = \mu_\phi + \sigma_\phi \tag{6}$$

$$\phi_- = \mu_\phi - \sigma_\phi \tag{7}$$

Where:

μ_ϕ = mean value of friction angle.

σ_ϕ = standard deviation of friction angle.

Base plane inclination:

$$\theta L_+ = \mu_{\theta L} + \sigma_{\theta L} \tag{8}$$

$$\theta L_- = \mu_{\theta L} - \sigma_{\theta L} \tag{9}$$

Where:

$\mu_{\theta L}$ = mean value of base plane inclination.

$\sigma_{\theta L}$ = standard deviation of base plane inclination.

Back plane inclination:

$$\theta H_+ = \mu_{\theta H} + \sigma_{\theta H} \tag{10}$$

$$\theta H_- = \mu_{\theta H} - \sigma_{\theta H} \tag{11}$$

Where:

$\mu_{\theta H}$ = mean value of back plane inclination.

$\sigma_{\theta H}$ = standard deviation of back plane inclination.

5.3 Combinations of variables

The number of combinations of the different variables is 2^n, where n is the number of variables as described above. For the current evaluation, there are four variables resulting in 16 combinations. Therefore, the Factors of Safety were calculated for 16 different combinations of variables.

5.3.1 Weighting functions

Weighting functions are applied to each of the Factors of Safety and are point estimates, p, of the Factor of Safety distributions. For the four independent variables, the number of point estimates is given by the following equation:

$$p = \frac{1}{2^n} \tag{12}$$

Where:

p = number of point estimates.

n = number of variables.

In this case the number of variables is four; therefore, a 1/16 weight has to be given to each point estimate. The point estimates for the current assessment are as follows:

p++++ = p+++- = p++-+ = p+-++ = p-+++ = p++-- = p+-+- = p--++ = p+--- = p-+-- = p--+- = p---+ = p-++- = p+-+- = p---- = p-+-+ = 1/16

The symbology of the point estimators is defined by the order of the variables: friction angle, cohesion, base plane inclination, and back plane inclination. Each of the variables is represented by a symbol, either + or -, representing the mean plus or mean minus the standard deviation of the variable. For example, p++++ refers to the point estimate associated with the Factor of Safety based on the mean plus standard deviation for all variables. A symbol of p--++ indicates the point estimator for the Factor of Safety based on the mean minus standard deviation for the friction angle and cohesion, and the mean plus standard deviation for the base plane and back plane, as used in the stability analyses.

The values of the point estimator variables used in the analyses are summarised in Table 3, and are based on the simulated populations developed in Crystal Ball and on the equations described in Section 5.3.

Table 3 Values of point estimator variables

Input Variable	Mean, μ	Standard Deviation, σ	μ + σ	μ - σ
Friction angle (deg)	45.6	5.8	51.4	39.8
Cohesion (kPa)	409.3	356.6	774.9	43.7
Base plane inclination (deg)	-11.3	22.9	11.5	-34.2
Back plane inclination (deg)	100.3	16.3	116.6	84.0

5.3.2 Solution

The 16 possible combinations of point estimates were used as input parameters into the SLIDE™ limit equilibrium software developed by Rocscience. The base plane and back plane orientations were represented in the limit equilibrium analyses as anisotropic strength functions with an angular variability of one degree on either side of the point estimator values presented in Table 3. This approach was taken to limit the allowable failure surfaces to a very narrow range associated with the anisotropic strength functions, to reduce the potential of representing the single point estimators as a variable range. Values for friction angle and cohesion were input as single point estimates based on the mean and standard deviations determined from the simulated populations.

A limitation of the limit equilibrium method and modelling using the SLIDE™ software is that structures are assumed to be infinitely continuous. In reality, joint structures have a finite continuity, and the bi-linear failure mechanism modelled as part of this study considers that the development of a failure surface involves a combination of failure along the joint surfaces, and failure through the rock mass. To accommodate this limitation, the variability of joint continuity and rock bridging are accounted for implicitly within the model by using the range of cohesive strength values derived from the physical mapping data, by the rock mass strength, and by the equations of Jennings (1972).

Factors of Safety were determined for the 16 combinations of point estimates. The evaluated Factors of Safety were imported into the Crystal Ball software, and simulated populations were developed using a Monte Carlo random sampling method on 50,000 trials. Figure 10 shows the cumulative frequency distribution for Factor of Safety for the overall slope, and Table 4 summarises statistics of the simulated Factor of Safety distribution.

Table 4 Summary statistics for FS for overall slope

	Overall Slope FS
Distribution	Normal
Trials	50,000
Minimum	0.0
Maximum	14.1
Mean	4.3
Standard deviation	2.1

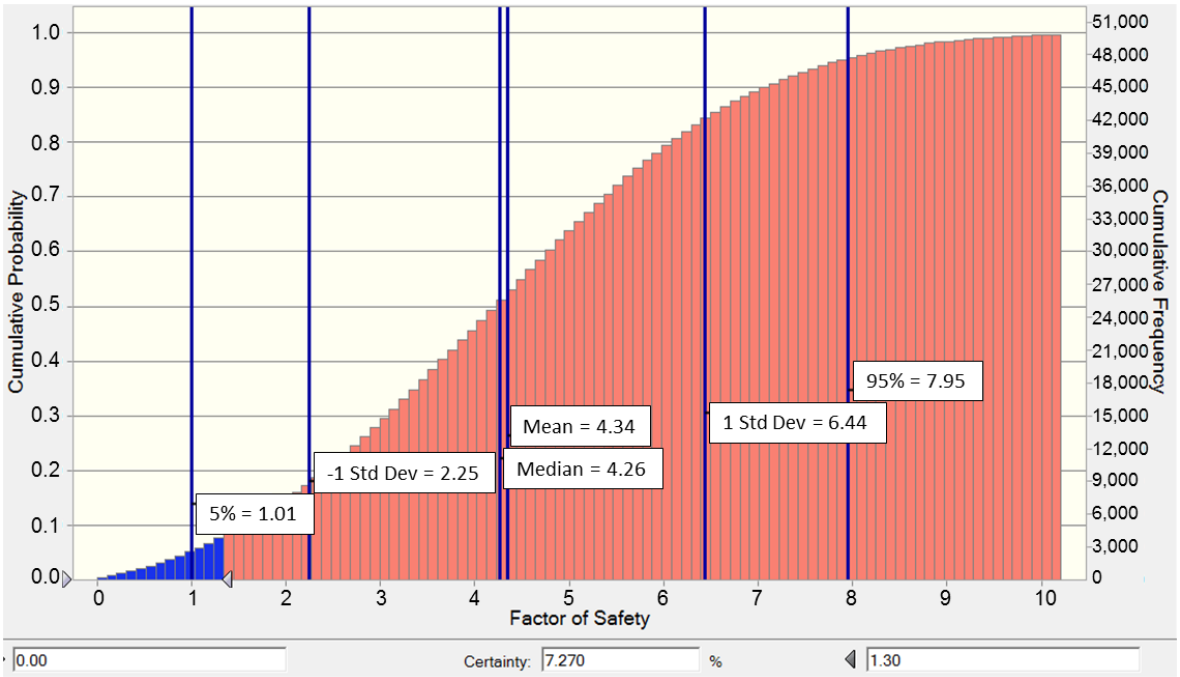

Figure 10 Overall slope FS cumulative frequency distribution based on simulated population

The probability of the Factor of Safety not meeting the design criteria (P(FS <1.3)), and the probability of failure (P(FS <1.0)) were evaluated using the simulated populations developed in Crystal Ball. The results are summarised in Table 5.

Table 5 Results of probability analyses

		P(FS < Criteria)	Reliability Index, B
Overall slope criteria	P(FS <1.3)	7.3%	1.4
Failure criteria	P(FS <1.0)	4.9%	1.6

While the average FS for the slope is indicated to be 4.3, there is still a probability associated with not meeting the design criteria (P(FS <1.3)), or of failure (P(FS <1). These results illustrate an important outcome of this approach to evaluating slope stability. While the possibility exists of not meeting the design criteria, or of slope failure, the probability of these events occurring is low and within the range of generally acceptable risk criteria as described in Section 3.2.

Given the unusual situation of the A21 project, specifically the presence of the dike and the lake at the crest of the pit slopes, the results of the probabilistic analyses are considered to represent appropriate conservatism in the design based on the recommended criteria discussed in Section 3.2.

5.3.3 Reliability index

Uncertainties in probabilistic analyses can be described in terms of the reliability index, which is defined by the following equation:

$$B = \frac{E[FS] - \ Design\ Criteria}{\sigma[FS]} \tag{13}$$

Where:

B = reliability index.

E[FS] = computed expected Factor of Safety.

σ[FS] = standard deviation of the computed Factor of Safety.

The reliability index is used to describe the Factor of Safety by the number of standard deviations that separate the expected value of the Factor of Safety from a defined design criterion. For example, if the criterion is defined by a Factor of Safety of 1.5 and the standard deviation is equal to 1, a reliability index of 2 indicates that the mean Factor of Safety is 2 standard deviations greater than the criterion, i.e. equal to 3.5.

6 Conclusions

The evaluation of the stability of open pit rock slopes is commonly assessed using deterministic methods. The deterministic approach is limited in that it does not explicitly account for the natural variability associated with the input parameters used for the modelling process. In deterministic studies, natural variability is typically accounted for by carrying out sensitivity analyses using conservative end-members of the input parameters. The results of these analyses are often conservative.

Reliability methods provide an alternative approach to the deterministic method and use probabilistic analyses to develop the probability density function that defines the range of Factors of Safety for a particular slope profile. An advantage of probabilistic methods is that the statistical distribution of Factor of Safety can then be used to evaluate the pit slope stability in terms of probability of failure and acceptable risk tolerance. Decisions relating to the development of an open pit mine project can therefore be expressed in terms of reliability, which can assist in performance-based decision making.

A combination of deterministic and probabilistic analyses has been used to provide a level of confidence in evaluating the overall slope stability for one option being considered for mining of the A21 kimberlite pipe. The level of confidence is expressed in terms of a simulated probabilistic distribution of a range of Factors of Safety, rather than in terms of a single deterministic Factor of Safety value. The cumulative frequency distribution for the factor of safety has been compared with commonly stated acceptance criteria for the probability of failure of a pit slope. The minimum Factor of Safety for the evaluation of long-term overall slope stability that is commonly accepted by the mining industry is 1.3. As discussed in Section 3.2, the equivalent acceptance criterion for a probabilistic analysis considers risk in terms of probability of failure, or the probability of a Factor of Safety less than 1.0. The review of current literature suggests that industry accepted probabilities of failure of between 1% and 10% constitute generally acceptable risk criteria when combined with an understanding of important site-specific contributing variables and a suitable monitoring program to manage risks to personnel. The results of the study indicate that, for one option being considered for mining of the A21 kimberlite, the stability of the southeast wall of the A21 pit assessed using probabilistic methods falls within these industry accepted probabilities.

Acknowledgement

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