The significance of identifying potential failure mechanisms from conceptual to design level for open pit rock slopes

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Slope instability in open pit mining environments present significant safety hazards. The quality and quantity of geotechnical data often expands over time and may give rise to an increase in reliability and a corresponding reduction in uncertainty of input parameters. In this paper, the geotechnical model is built on data obtained from four different consultants over 15 years, spanning from conceptual study to design. Stability conditions are investigated through Limit Equilibrium Method and compared to the numerical analysis using Finite Difference Method. Three critical profiles, based on areas of known concern, are analysed. Kinematically admissible joint orientations are incorporated as Ubiquitous Joint models and materials are modelled based on the Mohr-Coulomb failure criterion. Limit Equilibrium Method results revealed that profile A is the most critical slope, with a significant probability of planar and wedge failure at stack angle level. Safety factors for large scale planar failure of profile B, although stable, remains below the acceptance criteria for the overall slope angle, which opted for numerical analysis. Profile C was deemed stable and no further analyses were required. Good agreement between methods of analysis, in terms of safety factors and failure surfaces. Finite Difference Method computed lower safety factors to the point of critical stability for profile B. A reduction in overall slope angle by 12° for this profile increases the safety factor to an acceptable value and reduces the probability of failure to 2% from a previous 14%. The lowered range in probability suggests a reduction in result variability and thus an increased level of confidence in data and analysis.

INTRODUCTION

Mining operations in open pit mines produce progressively deeper pits. These structures account for a large portion of the world's mineral production (Wyllie and Mah, 2004). With the widening scope of mineral applications, their demand is driven by an ever-growing population (Lusty and Gunn, 2015). The United Nations (2015) projected a global population increase of more than 1 billion within the next 15 years; an increase from approximately 7.3 billion in the year 2015 to 8.5 billion. The imminent future thus carries the mineral industry into even more precarious environments in order to accommodate this demand (Lusty and Gunn, 2015). With increasing depths comes increasing risks of slope failure and it is thus essential to rigorously manage the hazard associated with rock slope stability. Fortunately, the advancement and increased utilization of computational tools now allows mine personnel to make improved informed decisions (Hochbaum and Chen, 2015). With increasing depths and associated stresses, Stacey et al. (2003) emphasized the importance of numerical stress analysis methods and Hoek et al. (2000) promoted the undertaking of numerical modelling, particularly for more complex slopes or lithologies where limit equilibrium analyses are often too simplistic.

The fundamental aim of slope stability analyses is to contribute to the safe and economic design of slopes (Abramson et al., 2002). Slope stability analyses, particularly in open pit mines, was a popular research theme in the 1960's and 1970's, most of which focussed on limit equilibrium methods. This period however also marked to onset of the early development of numerical methods of analyses (Stacey et al., 2003). Probabilistic slope stability analysis is another essential component as it can be employed in the quantification of uncertainty (El-Ramly et al., 2002).

Mines are required to develop acceptability criteria to be used as a standard value in quantifying the performance of slopes (Hoek et al., 2000). The performance can be described in terms of the factor of safety (FoS) or probability of failure (PoF). The importance of the slope in question gives an indication of the level of acceptance, where critical slopes with vital facilities such as ramps, are designed based on a higher FoS and lower PoF (Read and Stacey, 2009). Typical acceptance criteria for bench, inter-ramp, and overall slope are shown in Table 1.

Table 1. Typical FoS and PoF acceptance criteria values (Read and Stacey, 2009).

		Acceptance criteria		
Slope scale	Consequences of failure	FoS (min) (static)	FoS (min) (dynamic)	PoF (max) P[FoS ≤ 1]
Bench	Low - high	1.1	NA	25 – 50 %
Inter-ramp	Low	1.15 - 1.2	1.0	25 %
	Moderate	1.2	1.0	20 %
	High	1.2 - 1.3	1.1	10 %
Overall	Low	1.2 - 1.3	1.0	15 – 20 %
	Moderate	1.3	1.05	10 %
	High	1.3 - 1.5	1.1	5 %

This study the Limit equilibrium method (LEM) and numerical modelling through finite difference method to three critical profiles (AA', BB', CC') selected based on areas of known concern. Structurally controlled controlled failure mechanisms were assessed LEMs and compared with numerical models represented as pseudo-discontinuum media, using the Ubiquitous Joint network model (UJ). Pseudo-discontinuum media were based on a range of joint network models incorporating kinematically feasible joints which vary in terms of orientation, length, spacing and persistence.

SLOPE STABILITY ANALYSIS

Limit Equilibrium Method

Limit equilibrium methods (LEM) of slices can be employed to calculate the factor of safety (FoS) for circular, composite or fully specified failure surfaces (Stead et al., 2006). There are numerous methods of slices which vary in terms of the statics used in deriving the FoS as well as the assumptions employed to render the problem statistically determinate (Fredlund, 1975). According to Duncan (1996), the assumptions made for methods that satisfy all conditions of equilibrium do not have a significant effect on the safety factor, but methods that satisfy force equilibrium alone calculate safety factors which are strongly influenced by the assumed inclinations of the side forces existing between the slices.

Based on methods best suited for a specific failure surface shape, two LEM's will be discussed, namely the Bishop Simplified and Janbu Corrected methods. The Bishop Simplified method, proposed by Bishop (1955), satisfies moment equilibrium and vertical force equilibrium, but neglects horizontal force equilibrium. The safety factor calculation is identical to that of the Ordinary Method, but a variation in normal force definition exists (Fredlund and Krahn, 1977). Janbu's Corrected Method, proposed by Janbu et al. (1956), satisfies both horizontal and vertical force equilibrium and neglects moment equilibrium. It makes use of an empirical correction factor (f_0) to account for the effect of interslice shear forces, which are assumed to be zero, and then multiplying this factor by the computed FoS (Fredlund et al., 1981). The correction factor is a function of cohesion and the internal friction angle, as well as the failure surface (Fredlund and Krahn, 1977). This method may be employed for non-circular failure surfaces (Cheng and Lau, 2008).

Aside from the aforementioned assumptions, a major limitation of the LEM is the notion that the stress-strain behaviour of the material is ductile since no information regarding strain levels and strain variation along the failure surface is provided. This thus suggests uncertainty of peak strength being mobilized all together and across the entire failure surface, and the potential of progressive failure (Duncan, 1996).

Finite Difference Method

Ore bodies are often linked to intrusions and faults, thus resulting in typically non-uniform material properties (Hoek et al., 1991). For this reason, the LEM can be inappropriate in analysing complex conditions surrounding pit excavations, thereby presenting the need for numerical methods. Numerical methods of slope stability analysis compute approximate solutions to problems, incorporating strain during failure (Stead et al., 2006). Numerical models refer to computer codes that portray the behaviour of a rock mass that has been exposed to several initial conditions, which could include in situ stresses and water pressure, boundary conditions, and induced modifications to the surroundings (Lorig and Varona, 2001). The availability of powerful computers has resulted in such stability studies being done more efficiently and accurately than in the past (Duncan, 1996).

The FoS in numerical methods can be calculated by the Shear Strength Reduction (SSR) technique, by Zienkiewicz et al. (1975). This technique works by reducing shear strength properties, cohesive strength (c) and angle of friction (ϕ), until failure takes place (Matsui and San, 1992). It enables the computation of the sliding surface, since the failure mechanism is directly related to the development of shear strain, and a reduction in shear strength will thus lead to an increase in shear strain and the development of a potential failure zone (Matsui and San, 1992). Simulations are run for a series of increasing trial safety factors (f) each corresponding to a reduction of strength properties (ϕ_{trial} and c_{trial}) as seen below (Hoek and Bray, 1981):

$$c_{trial} = (1/f)c \qquad \text{Equation 1}$$
$$\phi_{trial} = \arctan(1/f)\tan\phi \qquad \text{Equation 2}$$

With a gradual increase of f, or the Strength Reduction Factor (SRF), and a corresponding reduction in shear strength, a critical SRF or FoS is calculated from local safety factors along the failure surface (Matsui and San, 1992).

Lorig and Varona (2001) discussed the general procedure of numerical modelling, which operates by dividing a rock mass into a series of elements or zones, which may be connected (continuum models) or separated by discontinuities (discontinuum models), and then allocating a material constitutive model and properties to each. The outcome of the simulations could be that of equilibrium or collapse of the rock mass, where the latter displays the mode of failure. For the purpose of this study, continuum methods of slope stability analysis will be discussed.

Continuum methods, such as the Finite Difference Method (FDM), are good alternatives to the conventional LEM since they are accurate, versatile and require minimal assumptions to be made (Griffiths and Lane, 1999). According to Stead et al. (2006), continuum methods are best applied to slopes comprised of massive, intact rock, weak rock or heavily fractured rock masses. However, Hoek et al. (1991) suggest their suitability for conditions relating to heterogeneous and non-linear properties due to the fact that each element explicitly models the behaviour of its allocated material.

The FDM solves algebraic equations explicitly via a time-marching procedure called dynamic relaxation (Dawson et al., 1999). It can be applied more effectively where more complex constitutive relations occur. These methods have the advantages of allowing for material deformation and failure, being able to model complex failure mechanisms and assessing the effects of groundwater and parameter variation with reasonable computing run times (Coggan et al., 1998). According to the same author there are limitations such as a need for an excellent understanding of the code in use, and the need of a great amount of data.

Ubiquitous joint model

The ubiquitous joint model introduce joints into originally isotropic intact rock to form a jointed rock mass where anisotropy of joint strength and deformability is presented in order for the rock mass to remain isotropic (Wang and Huang, 2009). Directional variation of rock properties, or anisotropy, is typically associated with distinct fabric elements in the form of bedding, foliation in metamorphic rocks or intense jointing (Amadei et al., 1987). According to Jakubec et al. (2001), the ubiquitous joint model can be used for failure simulations and is defined as an anisotropic plasticity model that exhibits strength anisotropy in a series of weak planes with predefined joint parameters. The associated parameters to be specified include the joint direction and shear strength properties, whereas other physical properties not defined, include joint spacing and location (Valdivia and Lorig, 2001). The geomechanical properties of the material and joints determine whether failure takes place through the solid material itself, along a weak plane, or both (Singh et al., 1994; Soren et al., 2014). Amadei et al. (1987) suggest that during mining operations, principal stress aligns with the direction of dip of ubiquitous joints. This stress adjustment encourages movement along these joints and may lengthen them (Cicchini et al., 2001) thereby increasing potential slip surfaces and ultimately, the magnitude of slope failures. The ubiquitous joint model has been used by several authors to account for the presence of weak planes, some of which include Sjöberg (2001); Valdivia and Lorig (2001).

THE GEOTECHNICAL MODEL

Slope designs are based on the geotechnical model, which is constructed from geological, structural, rock mass and hydrogeological models (Read and Stacey, 2009). All material types were considered to be isotropic, having identical mechanical properties in all directions and thus acting uniformly when subjected to stress. All materials were specified to be plastic in nature in order for their strength properties to be employed in the analysis of stresses and displacements in the event of failure.

The Rock Mass

In order to simulate the behaviour of rock masses, it is important to determine reliable estimates of rock mass properties and discontinuity properties (Ulusay, 2013). The elastic properties were defined by Young's Modulus (E_m) and Poisson's Ratio (v). Due to the limited availability of data, estimates of these parameters were based upon typical values from practical problems addressed by Hoek and Brown (1997). Values of 42 GPa and 0.2 for E_m and v respectively were employed in this study. The selection of these values relate to the Geological Strength Indices (GSI) of 67, 70 and 62 for tonalite, metagabbro and dolerite respectively which were calculated by Consultant B (2006).

The strength of jointed rock masses were initially estimated using the Generalized Hoek-Brown strength criterion (Hoek and Brown, 1980a). This made use of the uniaxial compressive strength (σ_{ci}) values obtained from consultant B (2006), m_i constants from consultant C (2009) as well as the Geological Strength Index (GSI) and disturbance factor (D) from Consultant C (2009). Triaxial testing was conducted on metagabbro and tonalite samples during the year 1996 and 2005 by consultants A (1996) and B (2006) respectively. The results of triaxial test were imported into RocData 5.0 (Rocscience Inc., 2015) in order to obtain representative and reliable estimates of rock mass strength parameters pertaining to the Generalized Hoek-Brown strength criterion (Hoek et al., 2002), and equivalent Mohr-Coulomb strength criteria parameters (Table 2). The minor (σ_3) and major principal stresses (σ_1) resulted in an intact uniaxial compressive strength (σ_{ci}) of 299.15 MPa and 214.05 MPa and an m_i constant of 11.07 and 33.6 for metagabbro and tonalite respectively. A Blast damage factor (*D*) value of 1, as suggested by consultant B (2006), was used and corresponds to a 'disturbed' rock mass, which may be attributable to blast damage and stress relief from overburden rock removal (Hoek and Brown, 1988). These parameters were derived by performing a curve fit of the laboratory data using the Modified Cuckoo fitting algorithm (Walton et al., 2011).

Table 2. Rock mass strength properties.

	Parameters	**Metagabbro**	**Tonalite**	**Dolerite**
Hoek-Brown classification	σ_{ci} (MPa)	299.15	214.05	145
	GSI	70	67	62
	m_i	11.07	33.6	16
	D	1	1	1
Hoek-Brown Criterion	mb	1.30	3.18	1.06
	s	0.0067	0.0041	0.0018
	a	0.50	0.50	0.50
Failure envelope range	Application	Slopes	Slopes	Slopes
	σ_{3max} (MPa)	4.82	4.84	4.45
	γ (MN/m^3)	0.028	0.028	0.028
	Slope height (m)	200	200	200
Mohr-Coulomb fit	c (MPa)	4.24	2.84	1.73
	φ (°)	49.19	55.23	44.00
Rock mass parameters	σ_t	-1.55	-0.27	-0.24
	σ_c	24.39	13.56	6.02
	σ_{cm} (MPa)	48.47	51.54	20.16

Shear Strength of Discontinuities

Input data pertaining to the shear strength of failure surfaces include the selection of the preferred joint shear strength model and the associated properties. Throughout this study, the Mohr-Coulomb shear strength model or slip criterion was employed with the friction angle (ϕ_{joint}) and cohesion (c_{joint}) being the input parameters. Consultant C (2009) determined basic friction angles (ϕ_b) of metagabbro and dolerite by performing shear tests on saw cut samples. Instantaneous friction (ϕ_i) and c_{joint} values were determined using the Barton-Bandis (Barton, 1973; Barton, 1976) equation which rely on normal statistical distributions for Joint Roughness Coefficients (JRC) and ϕ_b as well as average values of Joint Compressive Strength (JCS), normal stress (σ_n) and unit weight (γ). The average of the combined data was subsequently used in the Monte-Carlo simulation risk analysis program called @Risk (Palisade, 1997) to obtain a statistical range of friction and cohesion values. The resulting mean ϕ_{joint} of 48.82° and c_{joint} of 0.0145 MPa were used for all discontinuities throughout this study. The only specified driving force acting on the slope include gravity. The effect of water pressure on stability was not considered in this study since Consultant D (2011) noted that the majority of discontinuities in the pit were dry. Data relating to groundwater conditions were also unavailable hence all models were assumed to be dry.

Data Uncertainty/Reliability

The reliability of data associated with the geotechnical model is essential and should be defined. Hoek (1999) stated that data collected pertaining to this model, regardless of the amount, is unreliable to some degree due to the uncertainty related to the methods of allocating numbers to geology and that these numbers are merely usable estimates. Read and Stacey (2009) also mentioned a reduction in reliability due to the limited availability of data during initial stages of a project. Hacking (1975) differentiated between two types of uncertainties, these being aleatory and epistemic uncertainty. Aleatory uncertainty refers to the natural variability in the environment due to random processes occurring across space and time. A parameter linked to the rock mass is an example of aleatory uncertainty. In contrast, epistemic uncertainty pertains to the lack of data, or having a limited understanding of events and processes. Additional data associated with the latter could mean a decrease in this this form of uncertainty and thus an improved understanding of aleatory uncertainties of data (Fillion and Hadjigeorgiou, 2015). Read and Stacey (2009) separate uncertainties into geological, parameter and model uncertainty. Geological uncertainty includes aspects regarding the unpredictability inherent in the identification of, geometry of, and relationships between, the different lithologies and structures making up the geological model. Examples of such uncertainties include erroneously defining lithological boundaries and faults. Parameter uncertainty however refers to the unpredictability of the parameters employed in the geotechnical model, including values associated with the description of the rock mass. One component of this description includes the use of rock mass classification systems in rock slope engineering which remains a current practice, but could increase uncertainty if used incorrectly. The development of most such systems was based on confined underground conditions thus implying potentially questionable outcomes if not adjusted to suit surface conditions (Hoek et al., 2001). Lastly, model uncertainty relates to the unpredictability associated with the choice of the analysis in preparing the slope design and estimating the reliability of pit walls, and includes methods such as limit equilibrium and numerical analyses (Read and Stacey, 2009).

RESULTS

To evaluate potential non-structurally controlled instability modes, i.e. circular failure, the LEM code Slide (Rocsience Inc., 2015) was used. Analyses were based on the Bishop Simplified and Janbu Corrected methods and the resultant safety factors placed in the range of 4.35 and 5.67, with profile B having the greatest safety factors. When statistical values corresponding to

Discontinuities orientation for individual cut walls were analysed using Dips (Rocscience, 2015) and clusters of pole vectors and contour plots were defined to obtain the mean discontinuity sets shown in Table 4. The kinematic analyses were based on measurements of individual cut walls instead of a combination of data for the whole pit, to avoid potential masking of critical sets.

Table 4. Mean discontinuity sets by profile

	Set	Mean Dip (°)	Mean Dip Direction (°)	Window Dip range (°)	Window dip Direction range (°)	Slope face Dip Direction (°)
Profile AA´	J1	81	031	66-90 & 90-82	044-018 & 198-224	133
	J2	83	305	64-90 & 90-80	321-284 & 104-140	133
	J3	58	128	41-72	116-140	133
Profile BB’	J1	56	010	41-70	031-347	081
	J2	44	070	32-61	114-037	081
	J3	85	284	71-90 & 90-78	294-264 & 084-114	081
	J4	85	052	74-90 & 90-83	084-020 & 200-264	081
	J5	69	235	58-82	210-260	081
Profile CC’	J1	53	298	42-62	270-329	230
	J2	87	009	74-90 & 90-76	031-346 & 166-211	230
	J3	06	296	0-15	15-206	230

Kinematic analyses indicated the potential for planar failure in profile A and B at both overall and bench face angles Figure 1, (Table 5). Wedge failure was also likely at bench level in profile A, B and C and at the overall slope angle in section B (Table 5). Toppling failure was identified as a possible failure mode but was not analysed for the purpose of this study. The kinematically unstable joint sets for planar and wedge failure were further analysed using limit equilibrium analysis programs RocPlane and Swedge (Rocscience, 2015).

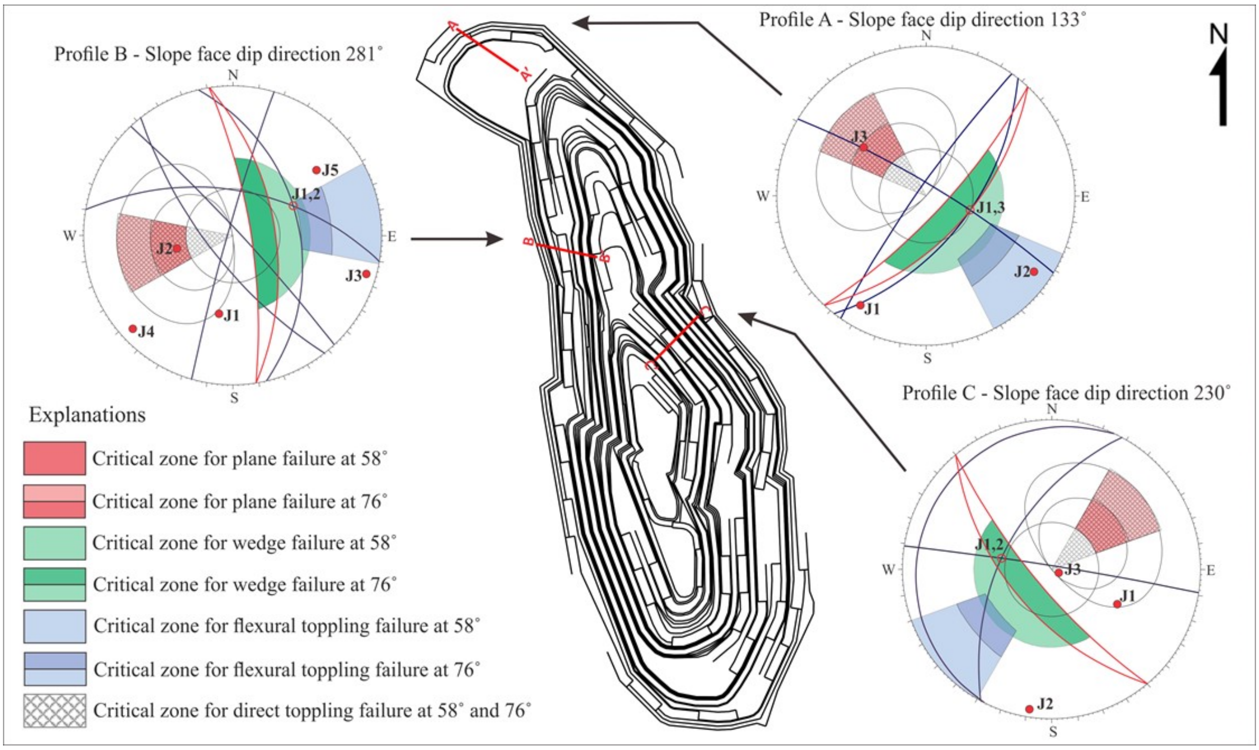

Figure 1 . Pit configuration and measurement zones from consultants with critical profiles' location, Kinematic analysis for thee critical profiles along overall slope angle and stack angle levels

Table 5. Kinematic analysis results for profiles A, B and C

	Overall slope angle			Stack angle		
	A-A′	B-B′	C-C′	A-A′	B-B′	C-C′
Planar	J3	J2	-	J3	J2	-
Wedge	-	J1; J2	-	J1; J3	J1; J2	J1; J2
Flexural toppling	J2	-	-	J2	-	-
Direct Toppling	J1; J2 J3 (base)	J3; J4 J2 (base) J3; J5 J2 (base)	-	J1; J2 J3 (base)	J3; J4 J2 (base) J3; J5 J2 (base)	-

Deterministic and probabilistic limit equilibrium analysis results revealed that the most problematic profile was Profile A, particularly at bench face level where safety factors are less than 1 and probabilities of failure are very high (Table 6). It is therefore clear that the bench face angle of 76° is not suitable in terms of stability. According to the general acceptance criteria proposed by Read & Stacey (Table 1), the FoS and PoF at bench slope scale should be 1.1 and 25% to 50% respectively whereas for overall slope scale the FoS can range from 1.2 to 1.3 and the probability of failure from 15% to 20%. Since Profile A has already failed through the deterministic analyses, planar failure at overall slope angle of profile B is the main focus as the safety factor is lower than the acceptance criteria (Table 6).

In order to account for the presence of weak planes in a FDM Mohr-Coulomb model, joint networks were simulated via the Ubiquitous Joint network model. Two joint set inclinations were analysed separately, the inclination selection being based on critical mean joint sets that predicted plane and wedge failure.

Table 6. Results of the deterministic and probabilistic limit equilibrium analysis

	Overall slope angle (58°)			Stack angle (76°)		
Profile A	Type	FoS	PoF (%)	Type	FoS	PoF (%)
	Planar	0.93 (unstable)	n/a	Planar	0.72	n/a
	Wedge	-	-	Wedge	0.77	n/a
Profile B	Planar	1.14	13.86	Planar	1.14	13.86
	Wedge	1.23	4.90	Wedge	1.23	4.90
Profile C	Wedge	-	-	Wedge	1.36	0

FLAC (Itasca, 2011) was used together with the Ubiquitous Joint constitutive model to represent multiple joint sets in order to see the effect that joints have the overall strength of the rock mass. When incorporating joint set 3, responsible for planar failure in profile A, it is confirmed that the problem is confined to bench level (Figure 2).

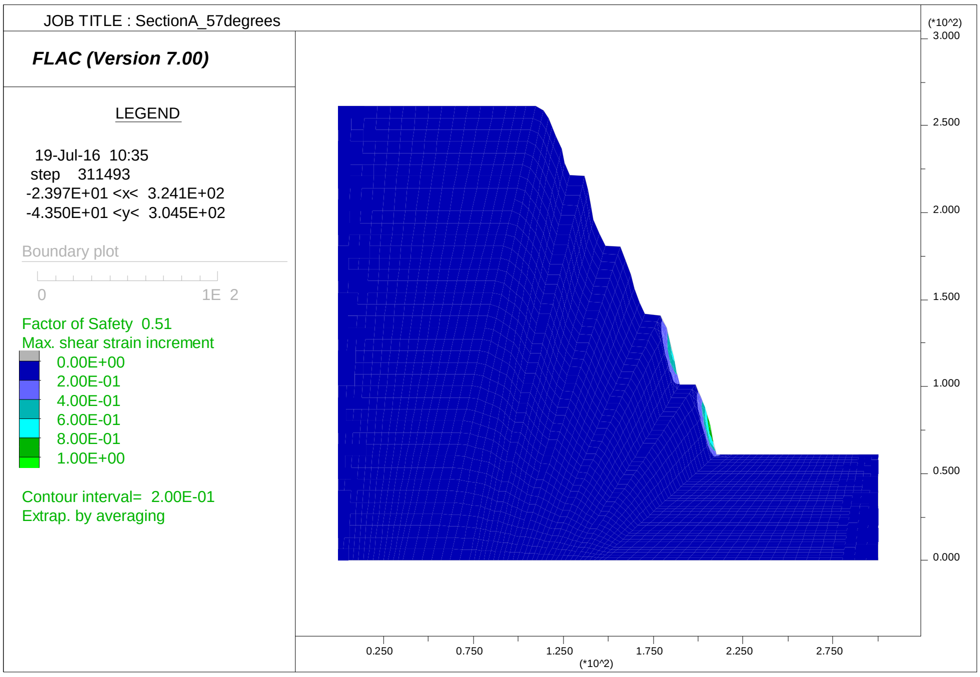

Figure 2. Profile A ubiquitous joint rock mass model using joint set 3 (57° inclination)

When applying joint set 2 as ubiquitous joints, which is responsible for planar failure, there is evidence of the initiation of tension cracks behind the crest of the slope as well as a deep seated approximately planar failure surface extending down to the toe of the slope (Figure 3). A safety factor of 1.01 is obtained and it is clear that the problem is now at overall slope level and not just multi-bench level as seen in profile A.

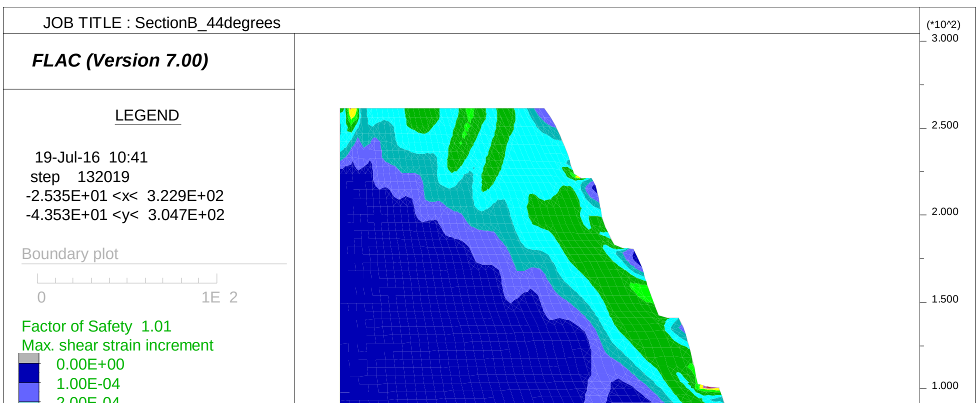

Figure 3. Profile B ubiquitous joint rock mass model using joint set 2 (44˚ inclination).

Figure 4 shows the 2-D profile views for both the entire pit (Figure 4a) and optimized conditions (Figure 4b). A reduction in overall slope angle of 12.5° resulted in an acceptable FoS of 1.20 where the probability of failure is also reduced.

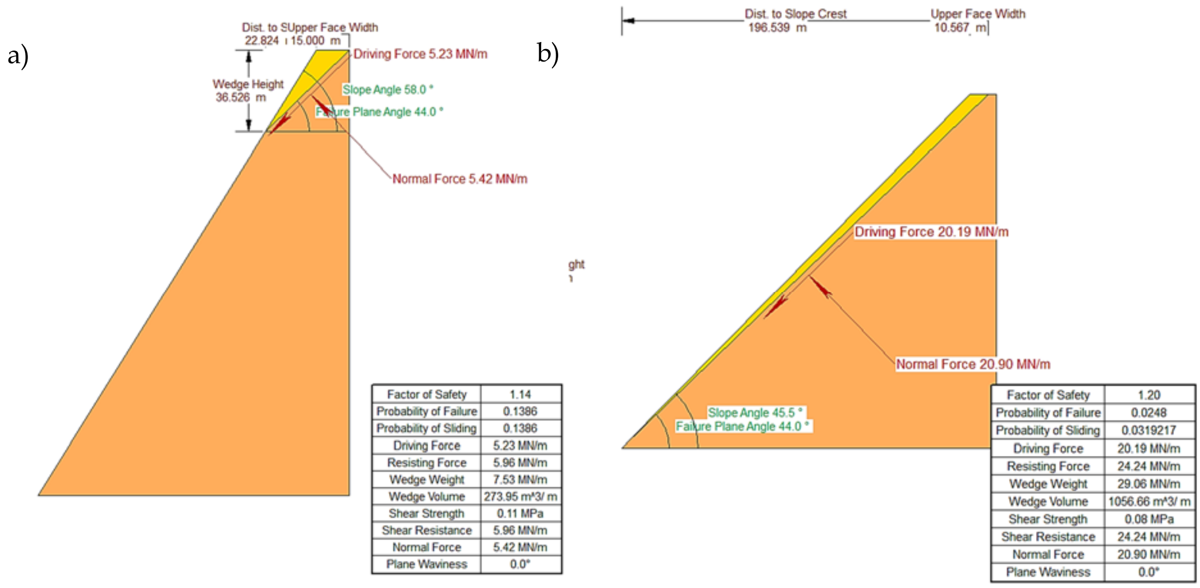

Figure 4. Profile B overall slope angle profile view at a) 58˚ and b) 45.5.

During the pre-feasibility stage, Consultant C (2009) obtained safety factors in high risk areas ranging from 1 to 1.3 and probabilities of failure from 20% - 40% whereas very high risk areas were associated with safety factors less than one and probabilities of failure greater than 40%. The results of the current study showed lower probabilities of failure associated with similar safety factors, thus suggesting narrower distributions and therefore less variability of results and potentially greater economic benefits.

DISCUSSION

There is good agreement between limit equilibrium and finite element analysis results, where joint sets associated with planar failure are represented in FLAC as Ubiquitous Joints and failure is confirmed. Failure can thus be said to be controlled by the presence of discontinuities. There is a need of optimization where costs allow for it. This should be in the form of the redesign of slope profiles, a change in slope face orientation or reinforcement. With information collected in 2009, consultant B assumed that the most likely mode of failure for the entire pit was circular failure. This is unlikely for hard rock slopes such as those present. With the addition of extensive discontinuity data two years later, failure is more likely to be linked to unfavourable orientated discontinuities in relation to the slope face. The epistemic uncertainty or the uncertainty related to the lack of information is therefore reduced to give a better understanding of the aleatory uncertainty of the data or the natural variability of the area.

CONCLUSION

This study provided an assessment of slope stability analyses based on data collected at several mining stages. Results showed that Profile A failed at multiple benches and that profile B is expected to fail at overall pit slope level. A proposed reduction in overall slope angle from 58° to 45,5° degrees is suggested, where the probability of failure is reduced to only 2%. With the reduction in range of obtained probabilities of failure, results became less variable and the level of confidence increased. Closely spaced joints with continuous joint lengths in continuum numerical models revealed that the stability of slopes are structurally controlled, where most safety factors were found to be below 1.5, and comparable to those of the predicted failure modes in the structurally controlled LEMs. Planar failure and/or wedge sliding is common at stack angle level, whereas planar and toppling modes are typically associated with overall pit slope angles.

This study shows the importance of performing more than one type of analysis in assessing the stability of slopes. LEM method can give relevant information, but it is recommended as an initial study where results should be confirmed and/or enhanced through numerical modelling. Numerical modelling methods allows for a better understanding of material behaviour through the calculation of stresses and displacements, and are better suited to more complex conditions.

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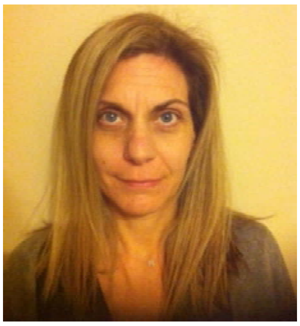

Dr. Maria Ferentinou, University of Johannesburg, Senior Lecturer, Civil Engineering Science, PhD, BSc, mferentinou@uj.ac.za Dr. Maria Ferentinou graduated with a degree in Geology from the University of Patras and a PhD in Engineering Geology from the National Technical University of Athens, where she has spent three years as Post-Doc researcher. She moved to South Africa in June 2013, to work at the University of KwaZulu – Natal as a lecturer in Rock Mechanics and Rock Engineering. Recently she joined University of Johannesburg as Senior Lecturer in the Department of Civil Engineering Science. She is a Professional Natural Scientist, and also a member of the South African Institute for Engineering & Environmental Geologists, and a member of the Geological Society of South Africa. She is also a member of the International Society of Rock Mechanics. Dr. Ferentinou's research focuses on landslides and slope stability analysis in natural and engineering slopes. To meet this this aim she is applying among others computational intelligence tools, and geoinformatics to assess landslide hazard.