Probabilistic Slope Design and its Use in Pit Slope Geometry Optimisation

S. Narendranathan, R. Dalugoda and P. Edmondson
Infra Tech Pty Ltd, Perth, Western Australia

ABSTRACT: The current economic climate and demand for iron ore has resulted in mine operators being prepared to accept aggressive slope designs and associated failures provided that they can be quantified (i.e. volume of failure, and clean up costs etc) and incorporated into the mining costs, at a feasibility study level. This has numerous implications for pit slope design / geomechanics, which are outlined in this paper. The authors intend to provide a detailed overview of the processes involved in deriving a geotechnical design for pit slope design.

1 INTRODUCTION

It is the intention of the authors that this paper would provide an adequate method for the quantification of this uncertainty hence enabling the slope design engineer to provide a slope design where the reliability is known. There are a number of benefits that arise as a result of this design process particularly for Iron ore mines in recent times where costs have to be kept to a minimum, due to falling metal prices. These economic benefits are detailed within this paper.

2 FACTOR OF SAFETY AND ITS IMPLICATION TO SLOPE DESIGN

One of the key items to that should be appreciated, is that probabilistic slope design potentially enables the slope engineer to design slopes with Factors of Safety (FOS) less than the usual 1.2 or even 1.0, providing the 'consequences' or risk can be catered for. To put into context, i.e. how an engineer might recommend a slope with a FOS of less than 1.0 is acceptable, the process by which FOS is calculated needs to be understood. What does a FOS of less than 1.0 indicate? Essentially it indicates that the sum of the driving forces is greater than the sum of the resisting forces. This implies that the slope is continually displacing, i.e. moving. What the reader should appreciate is that it may be quite tolerable for large displacements to take place and the slope still remains serviceable.

3 GEOTECHNICAL DATA COLLECTION FOR A PROBABILISTIC STABILITY MODEL

3.1 *Core Logging*

The authors recommend that when data is being collected as part of any geotechnical drilling and logging programme, it is done so to facilitate the calculation of empirical rock mass rating systems, for example, Q system (Barton et al. 1974), the Geological Strength Index, (GSI) system (Hoek and Brown 1997), The Rock Mass Rating (RMR) system (Bieniawski 1976). For the purposes of this paper the RMR system will be considered, this is discussed in further detail in the following sections of the paper.

3.2 *Surface (Wall) Mapping*

The information recorded during surface mapping shares some similarities with core logging; however there are some key differences particularly in relation to geological structure and slope performance assessment. Specifically the following 'extra bits' of information are recorded. Top and bottom termination observations of an observed geological structure, as well as its lateral (along strike of the slope) persistence so as to assess its influence within a slope exposure, as persistence is a 3D issue not 1D; Actual dip and dip direction as mapped from the observed structural plane within a slope as opposed to recording alpha and beta values from drill core that need to be converted to dip and dip direction based on the survey of the drill hole, this usually introduces some level of uncertainty (i.e. survey errors etc); Blast damage and degree of exfoliation of structures, i.e. to what level partially or weakly healed features may 'open up' due to rock mass exfoliation from overburden removal.

3.3 *Laboratory Testing*

Laboratory testing is primarily undertaken to correlate the empirical strength assessments made from core logging data to actual strength values. A laboratory testing program would consist of Unconfined Compressive Strength (UCS) testing, to assess the compressive strength of the respective lithological units, 3 stage direct shear testing, to assess the shear strength of individual defect planes under various levels of confinement, and 3 stage unconsolidated undrained triaxial testing (UU), to assess the shear strength of a rock mass under various levels of confinement.

3.4 *Formulation of a Probabilistic Geotechnical Model*

3.4.1 *Rock Mass Compressive Strength*

Rock mass compressive strength can be assessed empirically from field strength and point load indices recorded during core logging or surface mapping, typically (assuming the RMR system). Fig. 1 and Table 1 show a histogram of the distribution of UCS values for this material, as obtained from laboratory testing. It can be seen from the frequencies of each 'bin' that the total number of samples tested to assess UCS is significantly less than the number of available field strength indices. This implies that the distribution of field indices would provide a more accurate (i.e. more representative of insitu conditions) for the assessment of this material's variability. Fig. 1 presents the UCS distribution of Material A.

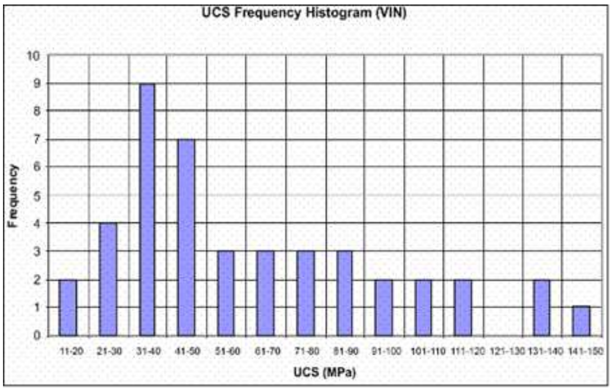

Fig 1. UCS Distribution for Material A

Table 1. Statistical Parameters of Material A's UCS Test Results

Mean (M)	Mode (M_0)	Median (M_d)	Standard Deviation (S)	Coefficient of Variability (S/M)
38.8MPa	20.0 MPa	20.0 MPa	34.8 MPa	89.7%

How does one choose a design value and assign variability i.e. choose a Coefficient of Variability (COV) for input into a stability model for the purposes of a statistical analysis?

3.4.2 *Rock Mass Shear Strength*

Rock mass shear strength can be determined empirically using correlations between RMR values and shear strength components after Bieniawski (1976), and analytically using 3 stage triaxial testing of material samples. The author would like to stress that determining rock mass shear strength based solely on the RMR values may provide inaccurate or dubious results as they are 'experience' based correlations as opposed to being site specific. However the distribution of RMR values for a particular lithological unit is an ideal method to calculate the relative variability of the material. What the slope design engineer should do is rely on the tri-axial results to provide the actual (analytical) values of cohesion and friction angles as interpreted from the stress path fitted to the Mohr's envelopes produced from 3 stage tri-axial testing, and assign variabilities to the individual parameters based on the COV of the RMR distribution. An example is discussed below. Fig. 2 shows the Mohr's circles (envelopes) and a fitted stress path as derived from a 3 stage tri-axial test, from which cohesion and friction angle values were interpreted. Shown in Table 2 is a statistical summary of the for the RMR values for this lithological unit.

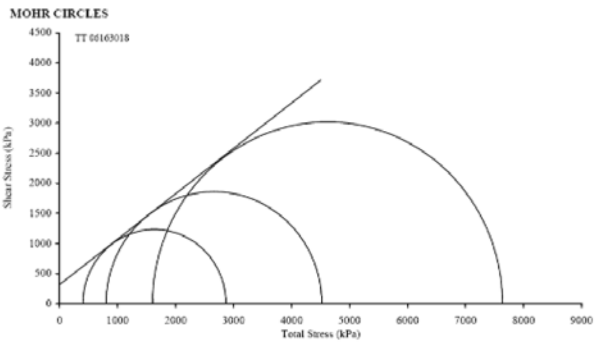

Fig 2. Mohr Circles

Table 2. Statistical Parameters of Material A's RMR

Mean (M)	Mode (Mo)	Median (Md)	Standard Deviation (S)	Coefficient of Variability (S/M)
46.7	42.0	47.0	8.8	18.8%

As 3 stage tri-axial testing is relatively expensive and time consuming to conduct and not many are usually done as part of a typical rock mechanics investigation. Therefore it is usually difficult to obtain meaningful statistical distributions of rock mass shear strengths. The author proposes obtaining this distribution from RMR values, which can be combined with the laboratory data (i.e. from tri-axial testing) in a similar manner to which the UCS values were combined with the COV calculated from the field strength indices. Hence in the above case the slope design engineer would choose average values of cohesion and friction angle of 320kPa and 37°, and assign a COV of 18.8 percent to both of these values.

3.5 *Geological Structures*

In many open pit mines the batter (bench) geometry is strongly influenced by geological structures, except in the presence of weak or altered rock masses. The orientation of geological structure is typically assessed from surveyed drill core. One of the draw backs of this is that it is difficult to determine the relative persistence (trace lengths) of the individual features and the level of mechanical inter-locking they exhibit within the slope. It is common practice for slope design engineers to perform kinematic (stereographic) analyses, based on contoured structural sets obtained solely from geotechnical drilling and core logging to assess the likelihood of structurally controlled slope instability mechanisms. A downside of performing this analysis on a stereonet is that the persistence of a particular structure is not taken into account. However, as stereonets do not consider the persistence of the structures the likelihood of these failures occurring (i.e. probability of the trace lengths of these structures are long enough to intersect each other) cannot be accurately assessed. As an example consider a batter wall height of 20m, a structure that has a 1m trace length will not result in overall batter slope instability. It may however result in the formation of small scale blocks that can be operationally managed via batter wall clean ups and scaling. Hence if the persistence of structures was recorded, the data plotted within the stereonet could be filtered to depict structures of ‘importance’ i.e. those that have a continuity of half a batter wall height or greater. From the stereonet in Fig. 3, of the structures that fall within the day light envelope, only 27 have a trace length greater than 10m.

Hence when calculating the likelihood (probability) of planar or tetrahedral wedge sliding the engineer needs to consider the following; what is the probability that a given structure will fall within the daylight envelope of a particular slope (PDE); what is the probability that a structure will have a persistence of half the batter height or greater (P0.5BH); Sliding or failure will occur when there are structures that satisfy both criteria; i.e. that fall within the daylight envelope of the slope and have a trace length of greater than half the batter height,

i.e. P Failure = PDE x P0.5BH (1)

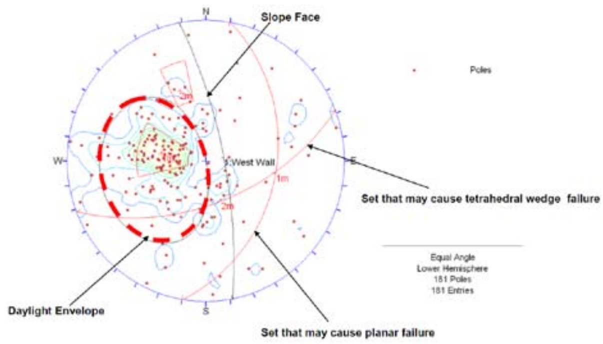

Fig 3. Stereonet of a slope face

4 CONDUCTING A PROBABILISTIC SLOPE STABILITY ANALYSIS

For typical open pit Iron ore mines, stability analyses are conducted in typically two broad parts namely:

Batter scale stability analysis – This analysis is primarily based on kinematics to calculate batter face angles and catchment berm widths which are dependent upon the persistence of structures and their respective orientations to the pit walls. The author would like to point out that when designing pits in very weak rock masses, slope geometry would be governed by the strength of the rock mass and effects of pore water pressures (if present).

Inter-ramp scale stability analysis – This analysis takes into account the batter slope geometry that has been formulated on the basis of kinematics and assesses the interaction of large scale structure and the insitu rock mass, to derive limitations on inter-ramp ‘stack heights’ and the ideal location and widths of ‘geotechnical’ berms or haul ramps to effectively decouple the overall slope height.

4.1 *Global Scale Slope Stability Analysis*

The global scale stability analyses incorporates the results from the batter and inter-ramp scale slope stability analysis can be quite complex as it involves a number of interactions, i.e. materials of different shear strengths (i.e. different lithological units) and their combined performance with large scale geological structure. Section 3.2 of this paper discussed on how the components of material shear strengths i.e. cohesion and friction can be formulated probabilistically, this information can be inputted within a within a software package like Slide to calculate the likely failure geometry and the distribution of Factors of Safety (FOS) and hence probability of failure. This probability of failure (PFailure) can be used to determine the reliability of a mine slope whereby:

Reliability (R) = 1 – Probability of Failure (2)

R can be interpreted as the likelihood that a particular slope would perform adequately under insitu conditions i.e. accounting for variabilities within material (lithology) and geological structure (Harr 1987). The Fig. 4 shows the output (FOS distribution) and cumulative distribution of FOS from a probabilistic analysis conducted on Slide.

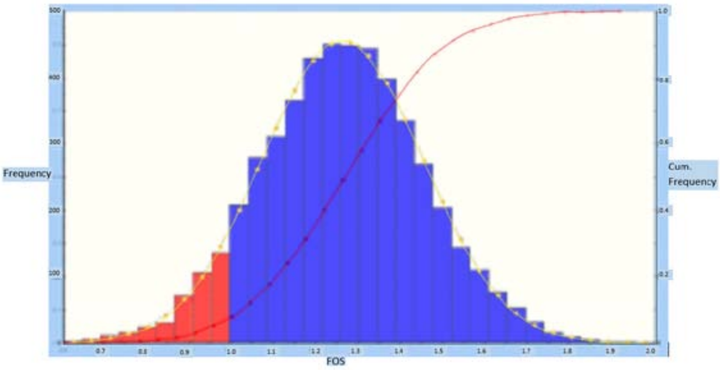

Fig 4. Distribution of FOSs

It can be seen that given the variability associated with the input parameters for this analysis, the mean FOS is approximately 1.25 and the PFailure i.e. the probability that the calculated FOS is less than 1.0 is 7 percent. This implies that the slope has a reliability of 93 percent.

4.2 *Optimising a Pit Slope*

The fundamental item to bear in mind when considering optimising a mine slope within a particular geotechnical domain is the consequence of the failure, i.e. the sum of the costs of the items identified above, in relation to the failure volume. It should be noted that for 'long life' pits this process may have to be carried out over a range of pit depths as the time value of money which strongly influences the projects net present value (NPV) may change over time. Within a particular slope domain the anticipated failure volume (Vf) can be estimated based on the calculated failure geometry from modelling and the associated slope face angles. However the actual failure will depend on the uncertainty of the model i.e. the probability of failure, hence:

V Expected Failure = P Failure x V Failure. (3)

The reliability of the slope design and anticipated consequence (cost) of failure can be plotted on the one chart to assist the mining engineer to select an optimum slope angle to form the slope. If the mining operation is prepared to 'accept' a certain degree of failure which can be included within the mining costs, then a lower reliability of slope design can be accepted, i.e. a steeper angle.

5 CONCLUSIONS AND DISCUSSIONS

Probabilistic slope design has a number of benefits in comparison to deterministic slope design as it addresses uncertainty of a particular design and gives the engineer an indication of the reliability of the calculated FOS. It can also be used to optimise slope angles based on a quantified level of risk (probability of failure / expected volume of failure). Fig. 5 below demonstrates how a relationship can be developed between slope angle reliability and financial risks.

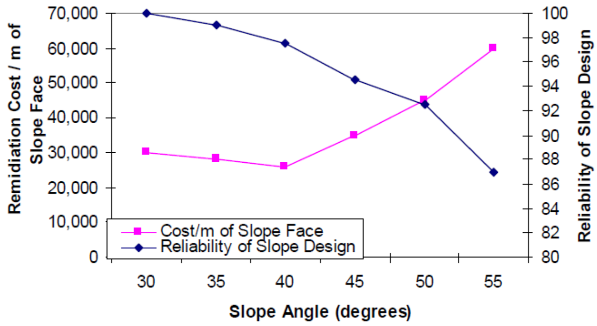

Fig 4. Slope organisation chart

This would facilitate better scheduling requirements, i.e. as the mine scheduling engineer now has an indication of the expected volumes of failure so he can make an estimate on clean up times and equipment requirements. This uncertainty of the geotechnical model can be inputted into spreadsheets that calculate Net Present Value (NPV) and Internal Rate of Return (IRR) or any other method for tracking performance.

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