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A New Approach to Assess Open Pit Slope Stability

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Abstract

A limit equilibrium or numerical model which is suitably representative of the shear strength character of a pit wall is a prerequisite for optimised pit slope design. Conventionally, evaluation of slope stability uses a failure criterion for either rock mass shear strength or the shear failure of structure. For example, the Hoek-Brown criterion for rock mass and the Barton-Choubey criteria for structure. The strength data determined from either or both of these criteria are then input into limit equilibrium analyses or numerical models and used independently to assess slope stability. However, their respective strengths are usually adjusted, either up or down, to allow for rock bridge or structure.

A new approach has been developed that estimates the ratio of rock bridge to geological structure and then simultaneously represents the strength characteristics of both rock mass and structure for input into limit equilibrium and numerical models. The method uses data from a three dimensional rock mass model that is derived from geotechnical logging. It uses the fracture frequency, Hoek-Brown and Barton strength parameters for the different rock mass domains identified for the rock mass variability established in the model, with strength parameters being calibrated against laboratory test data. Statistical analyses of shear strength and rock bridge data is then carried out for the rock units within each geotechnical domain. This provides input for slope stability analyses that represent both rock bridge and discontinuity shear strength parameters according to the calculated proportion of rock bridge. This method is more precise and representative than conventional methods that use single or factored failure criteria values. The paper describes this new approach to the assessment of open pit slope stability together with relevant case studies.

1 Introduction

Overall pit slope failure is a combination of failure through intact rock (termed rock bridge) and structure. Pit slope design engineers generally use the Hoek-Brown failure criteria to determine the likelihood of failure occurring through intact rock and the Barton shear strength envelope to assess the likelihood of structurally controlled failure. The strength data determined from either or both of these criteria are then independently applied to limit equilibrium or numerical models to assess slope stability. However, their respective strength parameters are usually adjusted to allow for rock bridge or structure.

In order for both methods to be applied simultaneously to slope stability design, a process has been developed that uses the fracture frequency within a rock mass to determine the ratio of rock bridge to structure expressed as a percentage of rock bridge. This ratio per rock type within each geotechnical domain is input into limit equilibrium and numerical models. Strength parameters for each are then assigned from Hoek-Brown (Hoek et al. 2002) and Barton (Barton & Choubey 1977) which have been statistically analysed.

2 Data collection

Data is collected from drillcore through application of the Domain Logging methodology (Dempers 1991). Selecting the geotechnical intervals or domains is the first step in the logging process. A domain can be many metres in length or less than a metre. Domains are determined from significant lithological boundaries which are further sub-divided according to geological structure, weathering, hydrogeology, veining and alteration within those major lithological boundaries. In addition, drill core domains are selected according to geotechnical characteristics by grouping together rock which displays similar geotechnical properties.

During the logging process, a full suite of geotechnical parameters is recorded (rock strength, joint characteristics and fracture frequency) enabling determination of ratings for the major rock mass classification systems.

3 Pit slope design process

The raw logging data is used to calculate various geotechnical parameters and rock mass rating values which enable three dimensional block models to be created using the resource estimation routines currently available in geological software packages (Seymour et al. 2007). The Mining Rock Mass Model (MRMM) allows all the logged values and calculated geotechnical parameters for the major rock classification systems to be represented in three dimensional block models.

Variability in rock mass conditions can be as a result of major geological structures, large fault zones, areas of closely spaced jointing, geological structures carrying water, weak rock, intense alteration and extensive rock bridge. This variability can be identified and visualised in the MRMM and each distinct zone can be individually domained to reflect the variability. The MRMM is interrogated for various geotechnical parameters to determine design zones for areas of the open pit. Figure 1 shows a plan of an open pit with geotechnical design zones selected according to geotechnical characteristics.

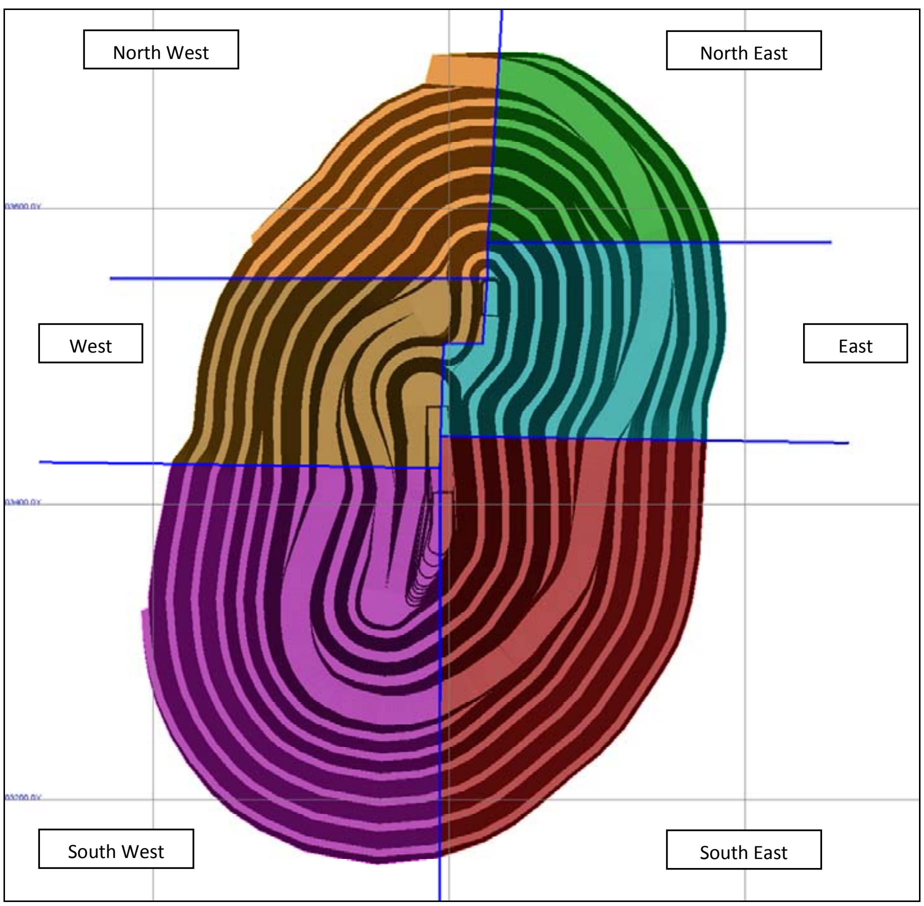

Figure 1. Plan showing an open pit with geotechnical design zones.

In addition to the classification ratings (RMR, MRMR, GSI, Q, etc.), individual critical geotechnical parameters are modelled. The critical parameters include:

- Joint intensity: calculated from RQD/Jn, represents the structure of the rock mass and gives a measure of block size (McCracken & Stacey 1989)
- Discontinuity shear strength: determined from micro roughness and joint infill, represents the roughness and frictional characteristics of the joint wall and infill material (McCracken & Stacey 1989, Barton et al. 1974)
- Fracture frequency (Laubscher 1990)
- Rock strength (Laubscher 1990)

4 Rock bridge

After the geotechnical design zones have been identified, the average RMR fracture frequency rating (FFR) per rock type within each design zone is calculated in the MRMM. The FFR ranges from 0 to 40 (Table 1).

Table 1. Fracture frequency rating ranges.

Average joints per metre	**Fracture Frequency Rating**		
	1 joint set	**2 joint sets**	**3 joint sets**
0.10	40	40	40
0.15	40	40	40
0.20	40	40	38
0.25	40	38	36
0.30	38	36	34
0.50	36	34	31
0.80	34	31	28
1.00	31	28	26
1.50	29	26	24
2.00	26	24	21
3.00	24	21	18
5.00	21	18	15
7.00	18	15	12
10.00	15	12	10
15.00	12	10	7
20.00	10	7	5
30.00	7	5	2
40.00	5	2	0

The percentage of rock bridge is determined from the average FFR. From experience in Western Australia, the maximum possible rock bridge is 70%. Therefore a FFR of 40 is equivalent to 70% rock bridge. For example, a rock type within a particular geotechnical design zone may have two fractures per metre with three joints sets which equates to a rating of 21 (Table 1). The rock bridge percentage is determined as follows:

$$\text{Rock Bridge \%} \quad = \quad \left(\frac{\text{FFR}}{\left(\frac{40}{0.7}\right)}\right) \times 100$$

$$= \left(\frac{21}{57}\right) \times 100$$

$$= 37\%$$

The calculated rock bridge percentage values for each design zone shown in Figure 1 are given in Table 2.

Table 2. Rock bridge percentage.

Design Zone	**Rock type**	**Fracture Frequency Rating**	**Percentage Rock Bridge**
NE	Felsic	20	35
	Metased	17	30
	Silsed	18	32
	UMF	18	32
E	Felsic	22	38
	Metased	20	35
	Silsed	19	33
	UMF	20	35
SE	Felsic	22	38
	Metased	23	41
	Silsed	21	37
	UMF	21	36
SW	Felsic	23	41
	Metased	22	38
	Silsed	22	38
	UMF	19	34
W	Felsic	24	42
	Metased	22	39
NW	Felsic	22	38
	Metased	19	33
	Silsed	14	24
	UMF	18	31

Having calculated the rock bridge percentage from the average FFR, the next stage is to determine the fracture frequency cut off rating, above which the rock mass is assigned rock bridge. The cut off is determined from the cumulative fracture frequency for each rock type per design zone obtained from the MRMM as shown in Figure 2. For example, the Metased rock type for the East design zone (Table 2) has a FFR of 20 and a rock bridge percentage of 35%. The fracture frequency cut off rating based on the distribution of the fracture frequency data is 22 as shown in Figure 2. The advantage of this process is that the variability of the rock mass characteristics within each design zone is considered during the design process.

After the fracture frequency cut off rating is determined, the distribution of rock bridge to structure can be spatially interpreted from the fracture frequency rating MRMM. A cross section through the East design region showing the rock type boundaries and the average fracture frequency rating block values for the design zone is shown in Figure 3.

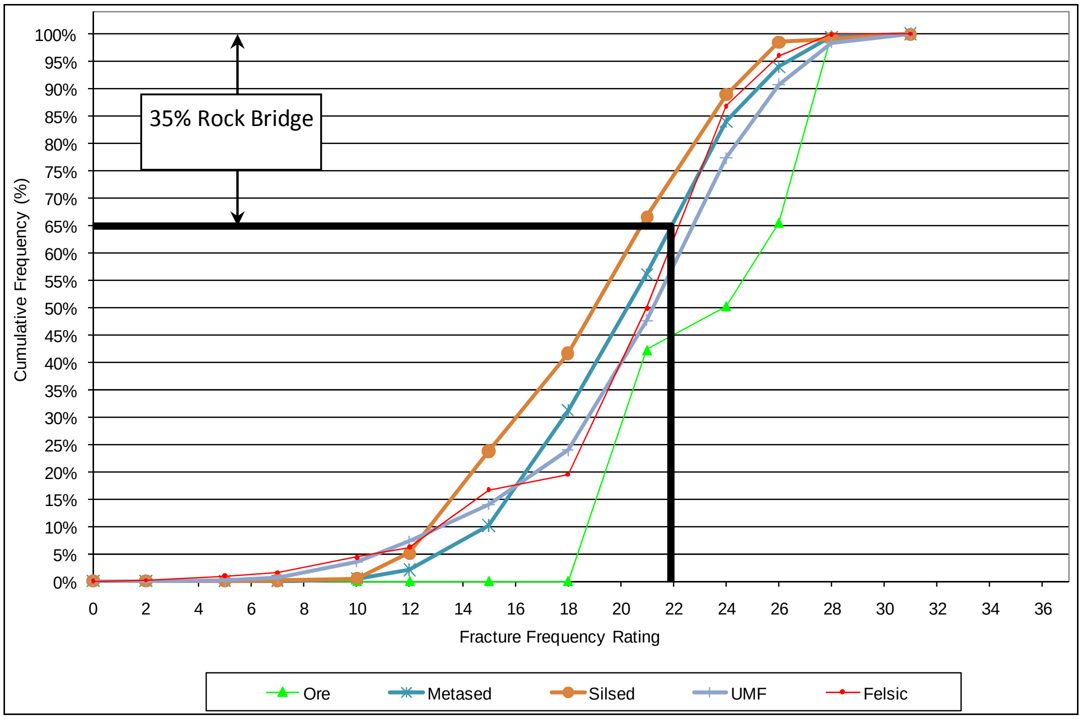

Figure 2. East domain fracture frequency cumulative curve.

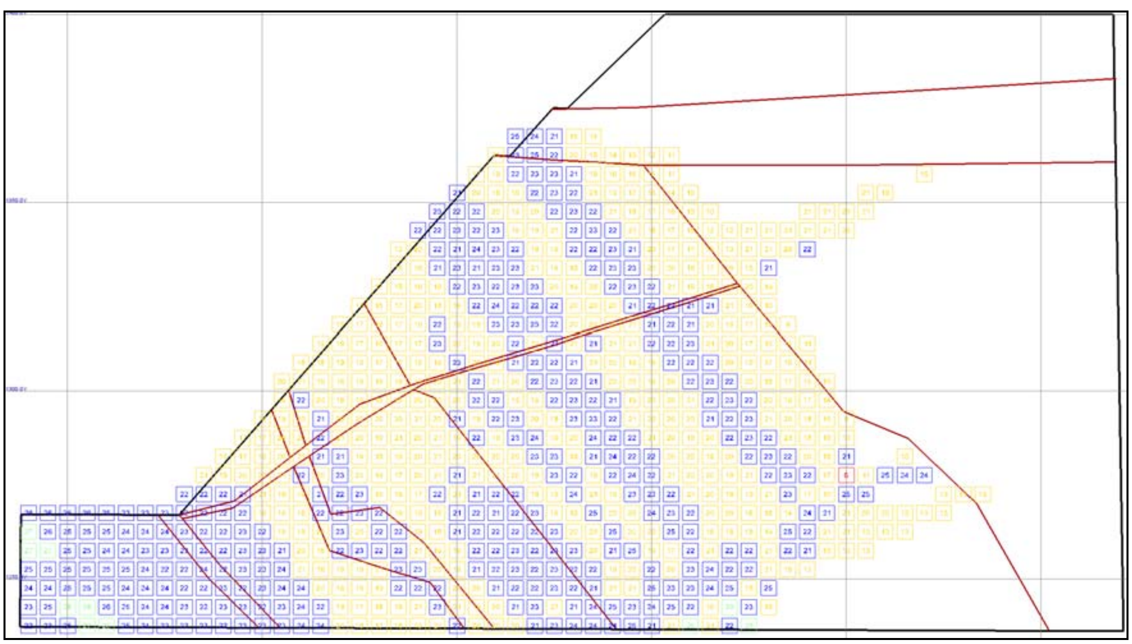

Figure 3. Cross section showing rock boundaries and average fracture frequency block values. Note fracture frequency rating values have been averaged across the thickness of the design zone.

Using the fracture frequency cut off rating for each rock type, the fracture frequency block model can be contoured to represent rock bridge boundaries as shown in Figure 4. The rock type and rock bridge boundaries are output as a DXF file and imported to limit equilibrium or numerical models as shown in Figure 5.

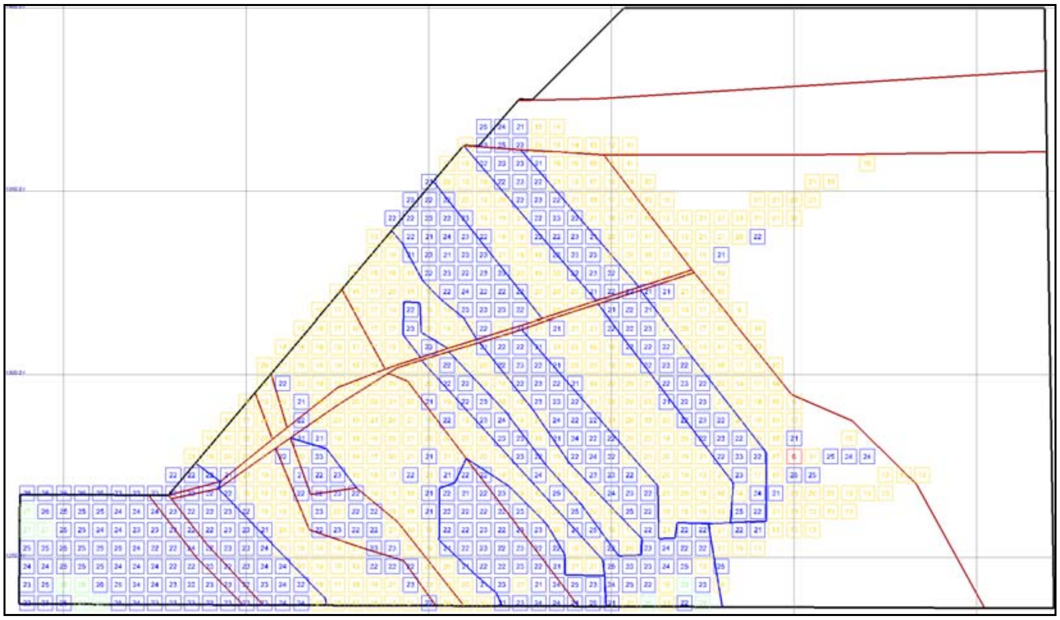

Figure 4. Cross section showing rock boundaries, average fracture frequency block values and rock bridge boundaries.

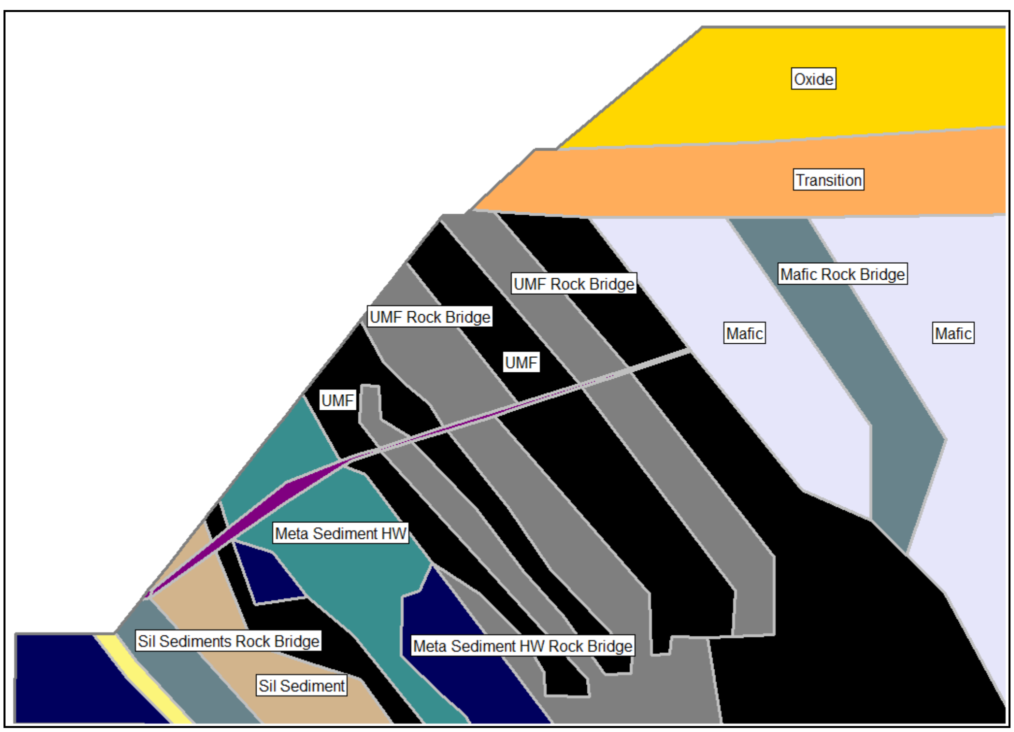

Figure 5. Rock and rock bridge boundaries imported into numerical models.

5 Strength parameters

Laboratory testing is carried out to determine UCS strengths, residual friction angles and base friction angles on selected samples throughout the rock mass. Test results are also used to calibrate the geotechnical logging. A sample set of data showing calibration of UCS laboratory results against logged values is shown in Figure 6. The calibrated logged values are used to create the MRMM block models. The advantage of using calibrated MRMM data is that statistical distributions between the minimum and maximum ranges of data for each geotechnical domain can be determined for probabilistic analyses. There is generally insufficient data for statistical analyses using only laboratory test results.

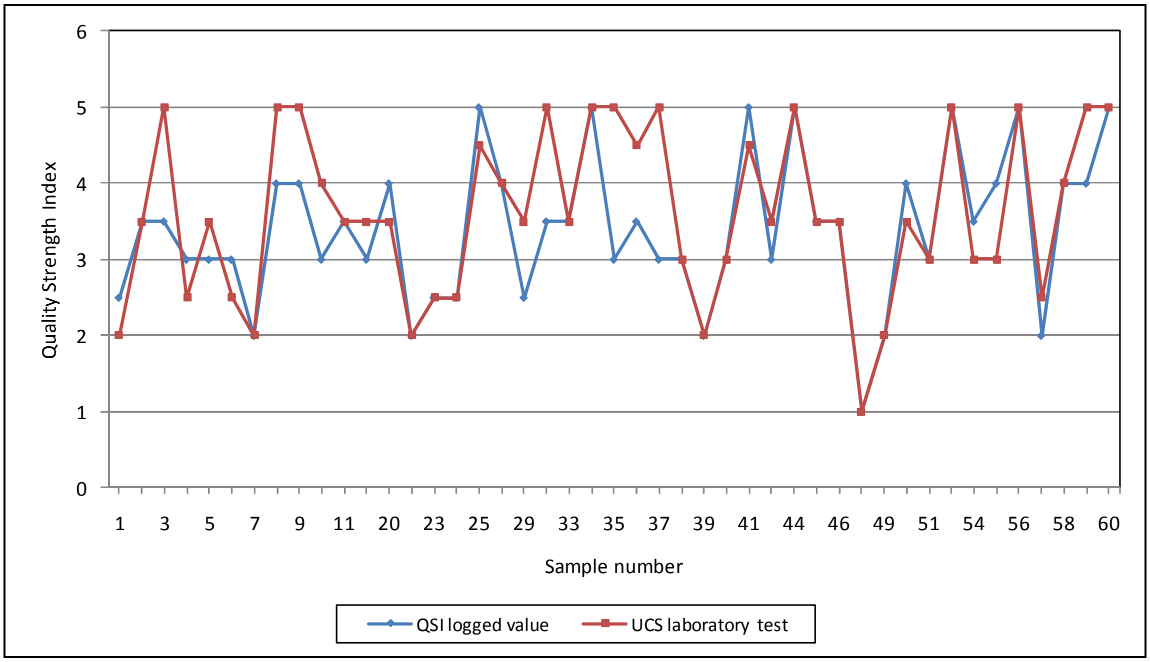

Figure 6. Calibration of logged rock strength (QSI) data.

Statistical analyses of all the strength data modelled in the MRMM (i.e. Hoek-Brown and Barton) are carried out for each rock type per geotechnical domain and then applied to the zones of rock bridge and structure in the models. These parameters are input to probabilistic limit equilibrium analyses for each rock type and geotechnical design zone to represent rock bridge and discontinuity shear strength. A sample data set is shown in Table 3.

Rock mass and joint property data from the statistical analyses are applied to design zones created from the MRMM and input to deterministic and probabilistic analyses using two dimensional limit equilibrium software packages.

An example of this approach using the software programme SLIDE (version 6.0) can be seen in the following case study. Analyses were carried out applying strength parameters based on Hoek-Brown (Hoek et al. 2002) and Barton (Barton & Choubey 1977) with no adjustments, together with the proposed rock bridge ratio approach.

Strength parameters for these analyses were determined from laboratory testing, calibrated core logging and rock mass classification.

Table 3. Statistical evaluation of strength data.

Rock type / Design Region	Statistics	Parameter			
		Uniaxial Compressive Strength (MPa)	**Geological Strength Index**	**Cohesion (kPa)**	**Phi (deg)**
Felsic / E	Mean	120	57	167	29
	Standard Deviation	27	3	27	0
	Minimum	64	45	123	28
	Maximum	154	68	284	30
	Relative Minimum	56	12	44	1
	Relative Maximum	34	10	117	1
	Count	1286	1286	1286	1286
Metased / E	Mean	121	53	127	29
	Standard Deviation	25	6	41	1
	Minimum	64	36	53	28
	Maximum	174	73	263	31
	Relative Minimum	57	17	74	1
	Relative Maximum	53	19	136	2
	Count	1939	1939	1939	1939
Ore / E	Mean	123	61	208	29
	Standard Deviation	17	6	17	1
	Minimum	100	49	167	28
	Maximum	154	70	238	31
	Relative Minimum	23	11	41	1
	Relative Maximum	31	9	30	2
	Count	246	246	246	246
Silsed / E	Mean	110	53	158	28
	Standard Deviation	37	7	28	1
	Minimum	64	35	67	25
	Maximum	174	72	221	31
	Relative Minimum	46	18	92	4
	Relative Maximum	64	20	63	2
	Count	794	794	794	794
Ultramafic / E	Mean	111	51	95	30
	Standard Deviation	34	8	37	1
	Minimum	4	32	40	25
	Maximum	174	71	256	32
	Relative Minimum	107	19	55	5
	Relative Maximum	63	20	161	2
	Count	4104	4104	4104	4104

These analyses for the same slope configuration show that using Hoek-Brown (Hoek et al. 2002) may result in an aggressive, steeper pit slope angle for the final design; and whereas the Barton (Barton & Choubey 1977) approach would result in a more conservative pit slope angle. Whilst both methods can be used, the suitability of the end result is very dependent on the design engineers experience and ability to adjust the parameters up or down. The Factor of Safety results from these analyses range from 2.2 to 0.9 (Figs. 7-8).

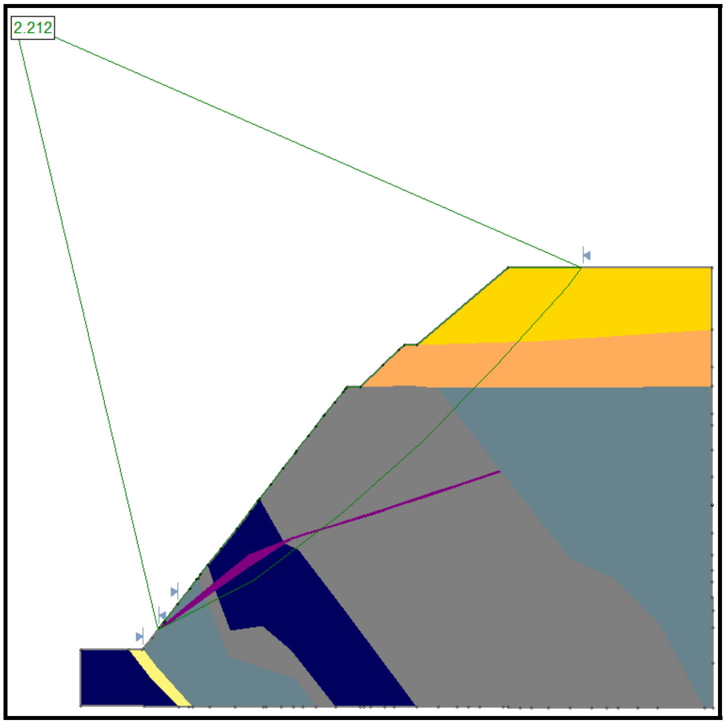

Figure 7. Analyses applying Hoek-Brown strength data (Hoek et al. 2002).

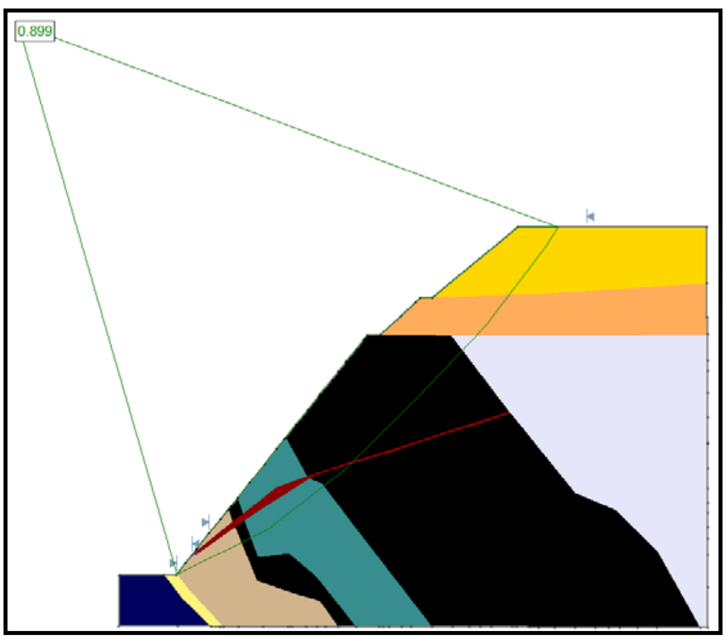

Figure 8. Analyses applying Barton strength data (Barton & Choubey 1977).

Application of the proposed rock bridge ratio approach results in a more realistic Factor of Safety of 1.3 with a pit slope angle based on the spatial properties of the rock mass in the slope (Fig. 9).

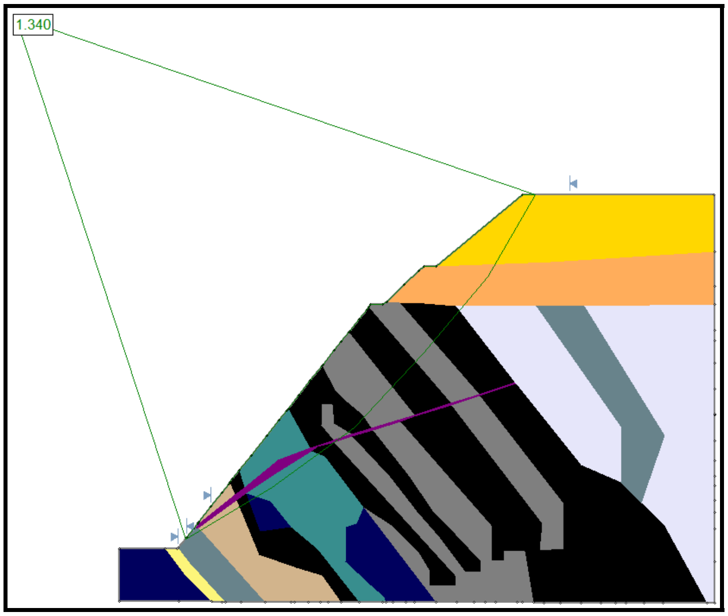

Figure 9. Analyses applying the rock bridge ratio with Hoek-Brown and Barton strength data (Hoek et al. 2002, Barton & Choubey 1977).

6 Conclusions

Routine pit slope design using either Hoek-Brown (Hoek et al. 2002) or Barton (Barton & Choubey 1977) relies on adjusting values to account for rock bridge and structure. The proposed method presented in this paper allows for both failure criterion to be applied simultaneously enabling a more realistic representation of the pit slope and more appropriate factors of safety and probabilities of failure.

The use of a 3 dimensional MRMM developed using calibrated data collected from drillcore will give a much greater range of strength parameters for probabilistic analyses compared to a limited number of results obtained from laboratory testing.

The inability to transfer geometrical and geotechnical information directly from a three dimensional model into a numerical model for slope stability analyses has been identified (Hoek et al. 2000). With the development of the MRMM, it is now possible to transfer an accurate and representative rock mass model directly to limit equilibrium and numerical models.

7 References

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