



Calibratable rock mass shear strength for open pit slopes

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Abstract

Open pit slope stability analyses for large scale instabilities are often carried out using limit equilibrium, finite element or finite difference computer programs. All of these require empirical estimates of rock mass shear strength when calibrated values are not available, which is a common occurrence. A common method of estimating rock mass shear strength is the Hoek-Brown failure criterion, which often over- or underestimates rock mass shear strength when used in limit equilibrium or elastic linearly plastic slope stability analyses, resulting in published alternative estimates for the coefficients of the Hoek-Brown failure criterion.

This paper investigates the differences between limit equilibrium and elastic linearly plastic slope stability analysis based on the Hoek-Brown failure criterion. The investigation consisted of a comparison of 3840 RS2 finite element analysis models completed with strain softening, and 216 Slide2 limit equilibrium models. The models were used to evaluate a methodology to determine equivalent limit equilibrium and elastic linearly plastic shear strength parameters that provide the same stable slope angles and instability back break distances as the more representative strain softening stress analysis methods. This paper presents the methodology and compiles the results into design charts and equations for equivalent limit equilibrium Mohr-Coulomb shear strength parameters, that simplify calibration against actual case studies. The results compare well with older empirical shear strength estimates, however, a detailed calibration of the results against case studies will need to be carried out. The analysis also did not consider pore pressure effects; this is left for future publication.

Keywords

rock mass strength, open pit slopes, Hoek-Brown, strain softening, Mohr-Coulomb

Introduction

Estimation of rock mass strength forms part of the open pit slope design process, as this parameter is a required input for the calculation of inter ramp and overall slope factors of safety (FS) and strength reduction factors (SRF). FS and SRF are typically assessed through limit equilibrium or elastic linearly plastic stress analysis methods. Rock mass shear strength estimates are particularly important for slope failure mechanisms where rock mass shear strength is a significant contributor to stability. Examples of rock mass shear strength controlled failure mechanisms are rotational failure in soft to medium strength rock masses and complex structural and rock mass fabric failure mechanisms where rock bridges often contribute most to the stability.

In an ideal world, rock mass strength would be measured directly, but as demonstrated by Hoek and Brown (1980), there is a scale component to rock mass strength, hence testing would have to occur on rock blocks of similar scale as the slopes being designed, for truly representative results. Testing of specimens of two orders of magnitude greater than current laboratory standards is physically impractical, thus rock mass strength estimation requires empirical rock mass strength models based on experience and back analysis. The most common, and perhaps most debated, model used today is the Hoek-Brown empirical peak rock mass shear strength model. The latest update of this model is published by Hoek and Brown (2019).

Limit equilibrium analysis models exclude strain compatibility and cannot accurately model rock mass behaviour, while stress models (i.e., finite elements and finite difference) can, if used correctly. This argument was used against the use of 3-dimensional limit equilibrium models by Read (2021). Read (2021) did however concede that 2-dimensional limit equilibrium methods can be used as they

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represent a calibrated system due to being in use for so long. The problem with adopting a stress model only approach, is that this higher-level analysis also requires significantly more model settings that can impact the results, such as element (or grid) types, shapes and sizes, maximum number of iterations, and also the elastic (and plastic) properties of the materials. The practical significance is that slope designs during the investment phase of projects (i.e., concept, prefeasibility, and feasibility) often will not have sufficient information for model correlation or calibration to build reliable stress models. Stress models also tend to take significant time to build and run, which becomes capital (time) intensive. In practice, the time and data limitations to perform the sensitivity analysis that is required to understand failure mechanisms early on in a project, are therefore still reliant on traditional limit equilibrium approaches.

While Hoek (2007) proposes using the Hoek-Brown failure criteria in limit equilibrium software, Hoek (2007) also states that rock mass should be analysed using strain-softening failure criteria, as that represents the known behaviour of rock masses. Using strain-softening failure criteria is a significant hurdle to the current implementation of the Hoek-Brown failure criterion within limit equilibrium software, as Hoek-Brown only describes the peak shear strength criterion, which should be used as a yield criterion, as per Hoek (2007) and Hoek and Brown (2019). Many slope designs are carried out using limit equilibrium methods while basing the rock mass strength estimates on the Hoek-Brown failure criterion, and as such are over/under estimating stability.

A methodology by Hoek (2007) to estimate an equivalent linear Mohr-Coulomb failure criterion from the non-linear Hoek-Brown failure criterion, simplified the use of the Hoek-Brown criterion for computational purposes, but did not capture the strain softening behaviour of rock mass material. This problem could have been resolved by demonstrating the calibration of the Hoek-Brown failure criterion as used in limit equilibrium software. Sjöberg (1999), Douglas (2002), and Garcia and Alcantara (2024) compared the Hoek-Brown failure criterion to slope stability cases using elastic linearly plastic models in finite difference software, and all these authors concluded that the Hoek-Brown model could significantly over- or underestimate shear strength, leading these authors to provide alternative coefficients for the model. While these authors used 2-dimensional elastic linearly plastic models, the SRF results will likely be similar to limit equilibrium FS values, as elastic linearly plastic and limit equilibrium analysis in two dimensions provide similar results as demonstrated by Hammah et al. (2004). It can therefore be concluded that a calibration of the Hoek-Brown peak shear strength model against slope case studies will show that stability can be over- or under predicted by a significant margin.

Instead of arguing against the use of the Hoek-Brown model, this paper takes the approach that all of these authors have valid points, and that reconciliation between their ideas can be found by tweaking the approach, as explained with reference to Figure 1 and Figure 2.

Figure 1 shows an idealised stress-strain graph for rock mass, with the rock mass behaviour close to slope faces at low confinement represented as Line 1 (green dash dots). Stability analysis using finite element and finite difference software is capable of modelling such behaviour, however, as stated earlier, significant additional data and computational effort is needed and so this approach is not practical for many slope stability problems. A simplified approach, the elastic instantaneously plastic model, is available in many software codes, and approximates rock mass

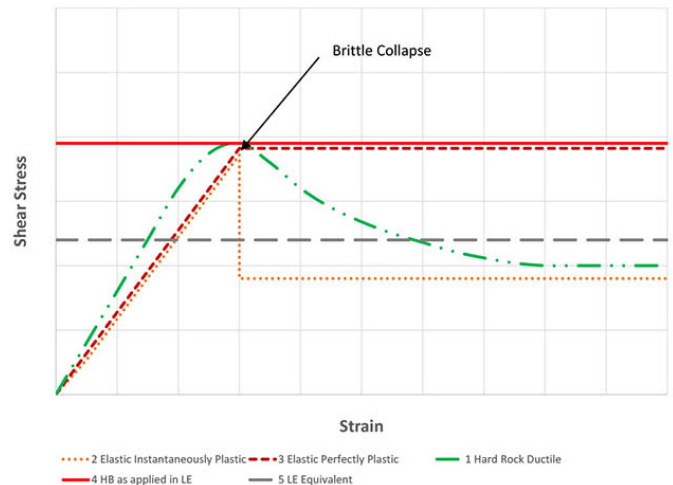


Figure 1—Stress-strain comparison of plastic vs. strain softening behaviour of rock masses

behaviour using an idealised linear stress-strain profile with instant strain softening as demonstrated by Line 2 (orange dots).

It is common for design engineers to use an elastic linearly plastic stress-strain profile (i.e., peak strength criteria) as shown by Line 3 (dark red dashes) when using finite element or finite difference software to estimate the SRF. The elastic linearly plastic approach tends to deliver SRF values close to the limit equilibrium FS (Hammah et al., 2004).

The reason the elastic linearly plastic SRF and the limit equilibrium FS tend to be similar, is that they both reflect a similar stress-strain profile. This is demonstrated by Line 4 (solid red) in Figure 1, which shows the stress-strain profile assumed when using limit equilibrium software. This is caused by two reasons:

- The limit equilibrium method can only account for force and moment equilibrium (depending on which method of slices is used; see Duncan and Wright (2005) for a good comparison) and is unable to account for the stress/strain relationship of rock masses, hence strength values must lie on a horizontal line in a stress-strain graph.
- One of the fundamental assumptions behind all limit equilibrium methods is that the FS values are the same for each slice generated using the method of slices as the overall FS value for a given slip surface.

As a result, SRF and FS analysis provide similar results as their stress-strain curves are similar for elastic linearly plastic material parameters.

Using the Hoek-Brown peak failure criterion in SRF or LE analysis over- or underestimates shear strength as real rock masses strain soften or strain harden and are not linearly plastic. This is illustrated in Figure 1 by the difference between Line 1 (representing real behaviour) and Lines 3 and 4 (representing idealised linearly plastic behaviour) to the right of the peak shear strength point. This difference explains why many authors such as Sjöberg (1999), Douglas (2002), and Garcia and Alcantara (2024) required a re-calibration of the Hoek-Brown coefficients to match their back analysis based on elastic linearly plastic analysis.

Another study by Rose et al. (2018) concluded that using the Hoek-Brown failure criterion presents more realistic results when the D factor is scaled with confinement to match their case studies. Rose et al. (2018) propose that this must be caused by the Hoek-Brown failure criterion not accounting for the lack of blast damage

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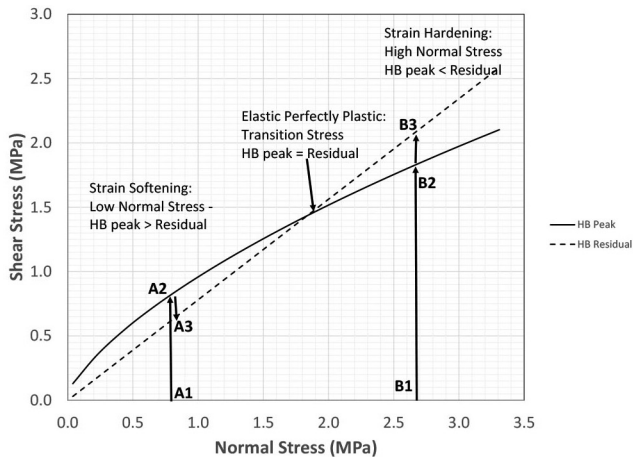


Figure 2—Major-minor principal stress comparison of shear strength approaches

further from the slope face. Their solution was to define a stronger rock mass farther from the slope face to prevent deep seated failure mechanisms. The authors of this paper do not doubt that blast damage reduces with distance from the slope face, however, there is another mechanism that explains the shallow failure surfaces observed in the real world, and also the potential reduction in blast damage with depth. This can be explained by considering Figure 2.

Figure 2 shows the normal stress shear stress graph for a hypothetical rock mass. The solid line represents the Hoek-Brown rock mass peak shear strength. The dashed line represents the residual shear strength, assumed to be a straight line here with a friction angle of 38°. Three zones are indicated: The strain softening zone where the peak shear strength is higher than the residual shear strength, the strain hardening zone where the Hoek-Brown peak shear strength is below the residual shear strength, and the inflection point where the two lines cross, which represents the only point on the graph where elastic linearly plastic conditions occur. It is often assumed that this inflection point occurs at high confinement outside of slope conditions, but as shown in this example, this inflection point can occur at typical slope stresses for weak to medium strength rock. The example here represents a σ_{ci} of 40MPa, GSI of 40 and m_i value of 8 and D of 0.7.

Two stress paths are shown, Stress Path A and Stress Path B, both with constant normal stress. In Stress Path A, the shear stress increases from A1 to the peak shear strength at A2, followed by strain softening from A2 to the residual shear strength at A3. This represents the zone where the Hoek-Brown failure criterion tends to overestimate shear strength and which was the focus of the work done by Sjöberg (1999), Douglas (2002), and Garcia and Alcantara (2024). Stress Path B starts at B1 and behaves elastically to B2 as shear stress is increased. At B2 the rock mass starts strain hardening until the yield point at B3 is reached where the rock starts yielding without strength gain. In this zone, using the Hoek-Brown peak failure criterion will result in underestimated FS or SRF values as the curvature in the Hoek-Brown curve results in shear strength predictions below that of a friction only material, such as sand or gravel.

The investigation presented in this paper is based on the position that all the authors who investigated this problem have valid points, but that their results differ because they were not talking about the same regions on the shear stress – normal stress graph. To investigate this position, this paper determines the equivalent limit equilibrium cohesions and friction angles that will

provide an FS of 1 for a series of dry slope scenarios analysed using elastic instantly plastic finite element analyses. This is analogous to Line 5 (dashed grey) in Figure 1, which lies above the residual shear strength, but below the peak shear strength. This analysis is presented for slope heights of 200 m, 300 m, and 500 m, and a range of σ_{ci} , GSI, m_i and D values that span across most of the valid ranges for these parameters. Pore pressure effects were not investigated in this paper.

Method statement

The equivalent limit equilibrium rock mass shear strength model was developed by comparing 3 840 analysed slope scenarios using the finite element software RS2, and 216 scenarios in the limit equilibrium software Slide2. The analysis consisted of generating 1 280 shear strength scenarios covering almost the full spectrum of possible combinations of Hoek-Brown parameters in a multi-dimensional grid. For each of the shear strength combinations, three sets of RS2 model slopes were generated with heights of 200 m, 300 m, and 500 m. For each slope height set, a model representing a different slope angle series was generated. The slope angles in the series ranged in one-degree increments from 25° to 89°.

The aim of the RS2 scenario analysis was to determine the slope angle that is ‘just stable’ (e.g., near an FS of 1) for each shear strength parameter set. For the purpose of this paper, this angle is called the critical angle. This was achieved by analysing each slope scenario in a number of slope angle stages, with the first stage starting at 25° and increasing the slope angle by a degree for each subsequent stage. As the first stage was defined to always be stable, increasing the slope angle in subsequent stages resulted in decreased stability until the critical angle was reached, with further slope angle scenarios failing to converge. To save time, the analysis was programmed to stop once the critical angle was reached. The last stage where the model converged within a set number of iterations was taken as the critical angle slope. In this way a database was created representing slope heights of 200 m, 300 m, and 500 m, and all the different combinations of σ_{ci} , m_i , GSI, and D, containing backbreak distances corresponding to each critical angle.

The limit equilibrium analysis was then used to determine Mohr-Coulomb parameters that match the critical angles and backbreak distances for the slope angle scenarios obtained in RS2.

Integration of the results was carried out by matching the critical angles and backbreak distances from the limit equilibrium models and the finite element models (see Figure 3 for a demonstration).

The relationship between the equivalent limit equilibrium cohesion and the critical angle for each of the three slope heights were analysed. This correlation was used to determine the

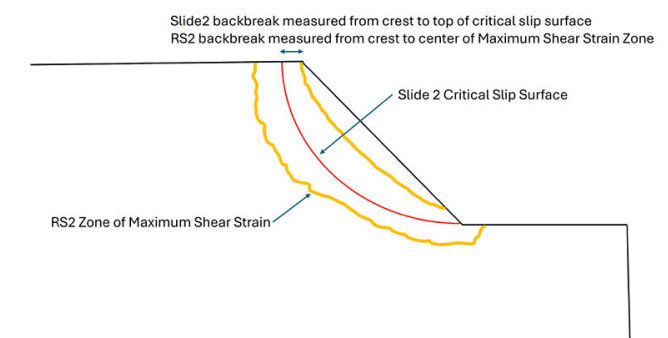


Figure 3—Measurement of back break distance

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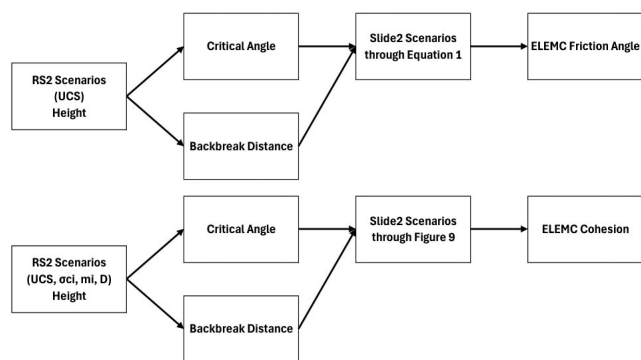


Figure 4—Matching RS2 and Slide2 analysis results

equivalent limit equilibrium cohesion. The process is illustrated graphically in Figure 4, which refers to equations and figures in Section 3.

In this way equivalent limit equilibrium cohesion and friction angle values, that provide the same Critical Angle and back break distance from the crest between limit equilibrium and finite Element methods, can be calculated.

Finite element analysis

The finite element analysis consisted of a single model with slope angles from 25° to 89° in one degree increments, as shown in Figure 5.

The model was run by simulating each slope angle as a different stage starting with the first stage having a slope angle of 25° and then progressively increasing the slope angle one stage at a time until the model no longer equilibrates. The model was then terminated to save time as further stages would simply increase the model run time without adding to the result. The last stable angle was used for the analysis and reported as the critical angle.

To test the effect of mesh element size on the model result, a sensitivity analysis was carried out on the following mesh element number scenarios: 10 000, 20 000, 30 000, 40 000 and 80 000. All the scenarios were evaluated for the 500 m slope height model, which has the largest mesh element size, and for 3 sets of material parameters: an intact rock σ_{ci} of 10MPa, 100MPa and 140MPa, and GSI values of 10 and 50, respectively, with m_i values of 40. The sensitivity analysis showed that the error made if using 20 000 mesh elements per model (as opposed to 80 000 mesh elements) would be between 0° and 3° with a standard deviation of 1.4° in terms of critical angle. As the combined model run time for all analyses was approximately 1 year for a 20 000-mesh element case and increasing the number of mesh elements to 30 000 would take approximately 1.5 years to run, the decision was made to accept the error.

The main model settings were:

- Uniform mesh type;
- 3 noded triangles;
- 20 000 mesh elements regardless of slope height;
- Generalised Hoek Brown material strength model with strain softening.

1 280 scenarios for each slope height were simulated using the following parameters:

- Slope Height = 200 m, 300 m, and 500 m;
- σ_{ci} = 10 MPa to 160 MPa in 10 MPa increments;
- Hoek Brown GSI value = 10 to 80 in increments of 10;
- Hoek Brown m_i value = 5, 10, 20, 30, 40;
- Hoek Brown D value = 0.7 and 1.
- Residual shear strength of 0 kPa cohesion and 38° friction angle, following the suggestion by Hoek (2007). The residual strength was considered linearly frictional to high confinement motivated by experiments done on high confinement friction angles of rock by Byerlee (1978), and is therefore considered approximately realistic for rock masses.

For each of the models, the elastic linearly plastic option for the generalised Hoek-Brown shear strength parameters were used. The Poisson's ratio was set to 0.25 representing a mid-range for most rock types and the rock mass elastic moduli were calculated using the Hoek and Diederics (2006), as shown in Equation 1.

$$E_{rm} = 10^5 \frac{1 - D/2}{1 + \exp \left[\frac{75 + 25D - GSI}{11} \right]} \quad [1]$$

As the RS2 software requires Hoek-Brown inputs for both the peak and post peak shear strengths, the equivalent Hoek-Brown inputs for 0kPa and 38° friction angle were determined using the chart derived for this purpose, as depicted in Figure 6. The chart was derived by fitting the Hoek-Brown equation through a series of data points representing various friction angles with zero cohesion. Using this chart, selecting a 38° friction angle with zero cohesion requires using the same σ_{ci} value as the peak shear strength, a $s_{residual}$ value of 0.0001, an $a_{residual}$ value of 1 and an $m_{residual}$ value of 3.3. The fixed $s_{residual}$ and $a_{residual}$ values will always result in a Mohr-Coulomb line with zero cohesion. Varying the $m_{residual}$ changes the residual friction angle, while varying the $\sigma_{residual}$ makes no difference. This is, however, hard coded into software codes, and so needed to be factored in.

Limit equilibrium analysis

The limit equilibrium analysis utilising Slide2 from Rocscience, consisted of carrying out a slope stability sensitivity analysis to determine the scenario slope angle with a factor of safety of 1 (critical angle) and the corresponding backbreak distance from the crest. A total of 216 scenarios were analysed, representing all combinations of the following attributes:

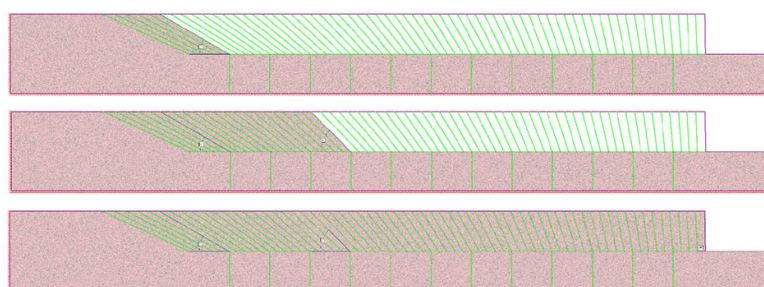


Figure 5—Example finite element analysis model showing steps (flatter slope Stage 5 Top, moderate slope Stage 20 Middle, and steeper slope Stage 64 representing 89°)

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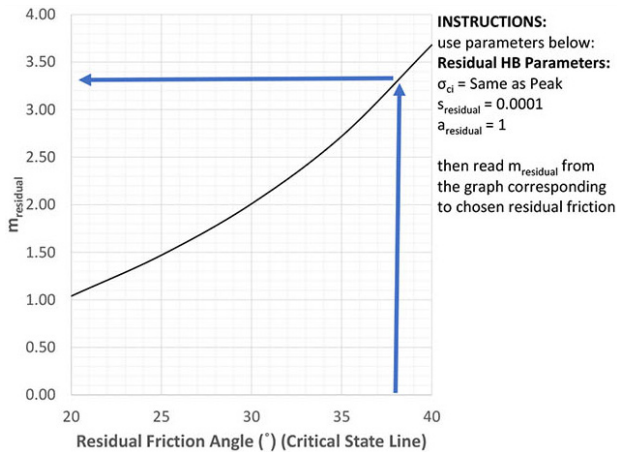


Figure 6—Application of residual friction angle through Hoek-Brown parameters

- Slope Height = 200 m, 300m, and 500 m;
- σ_{ci} = 10 MPa, 50 MPa, 100 MPa, 150 MPa;
- Hoek Brown GSI value = 20, 40, 60;
- Hoek Brown m_i value = 5, 20, 40;
- Hoek Brown D value = 0.7 and 1.

These values were selected to represent a realistic full spread of the likely parameters for the majority of rock slopes.

An example of a model used is presented as Figure 7 representing a 55° slope.

The model settings were:

- GLE/Morgenstern-Price;
- Number of slices = 50;
- Tolerance 0.005;
- Maximum Iterations 75;
- Non-circular;
- Path search;
- Optimisation settings = defaults.

Equivalent limit equilibrium Mohr-Coulomb parameters

During this process, it was realised that the relationship between these two analysis types can be simplified to reduce the number of Slide2 scenarios needed, because equivalent friction angles could be found to match both the critical angle and the backbreak distance from the crest by considering only variations in the σ_{ci} . Therefore, the relationship between σ_{ci} and the equivalent limit equilibrium friction angle was utilised to simplify the integration.

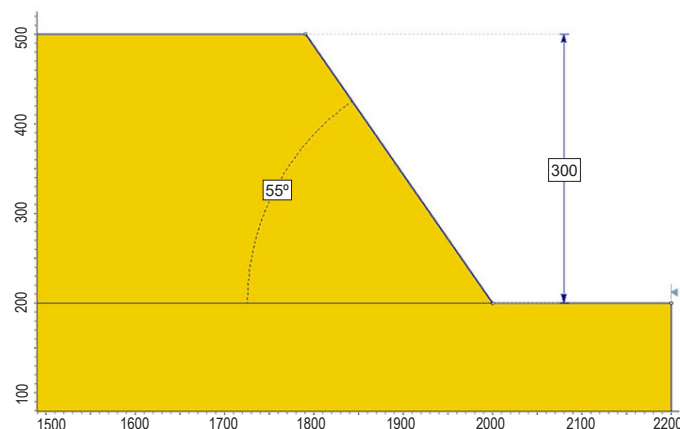


Figure 7—Example limit equilibrium analysis model

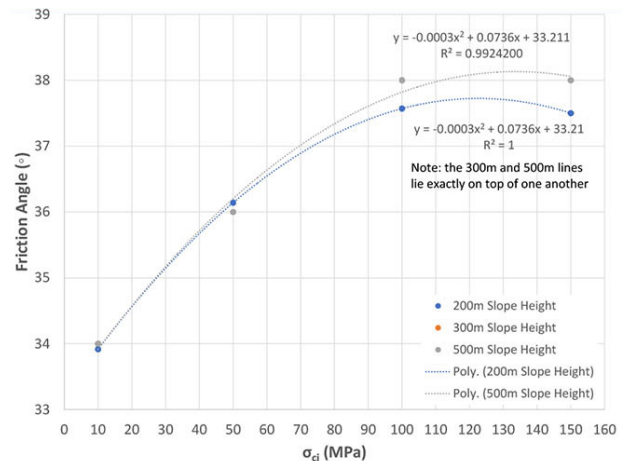


Figure 8—The σ_{ci} friction angle results for 200 m, 300 m, and 500 m slope heights

Results of the limit equilibrium and finite element analysis are compiled in the following to show how the limit equilibrium results were matched to the finite element analyses, the equivalent limit equilibrium Mohr-Coulomb (ELEM) parameters were matched to the Hoek-Brown parameters, and how rock mass strength for limit equilibrium and elastic linearly plastic finite element analyses can be estimated. To achieve these, the ELEM friction angle results are presented first, followed by the ELEM cohesion.

Equivalent limit equilibrium Mohr-Coulomb friction angle

Matching of LE to FE occurred by matching the critical angle and back break distance for each FE scenario to an LE scenario. As the LE back break for each slope is precisely specified by the critical failure surface, but the FE back break is provided by a broad zone of maximum shear strain, there was a little leeway in matching multiple FE analyses to each LE analysis. Using this leeway, it was discovered that the ELEM friction angle can be directly correlated to the Hoek-Brown σ_{ci} parameter, as demonstrated in Figure 8, with the choice of GSI, m_i and D making no difference. Each point visible in the figure represents 6 points as combinations of GSI, m_i and D. Figure 8 also shows that the 300 m and 500 m regression curves are exactly the same, but that the 200 m line provides slightly lower ELEM friction angles by about half a degree, which is considered immaterial. This difference is likely smaller than the margin of error in estimating σ_{ci} and so, for practical purposes, one might choose the 300 m and 500 m equation for the relationship as presented in Equation 2:

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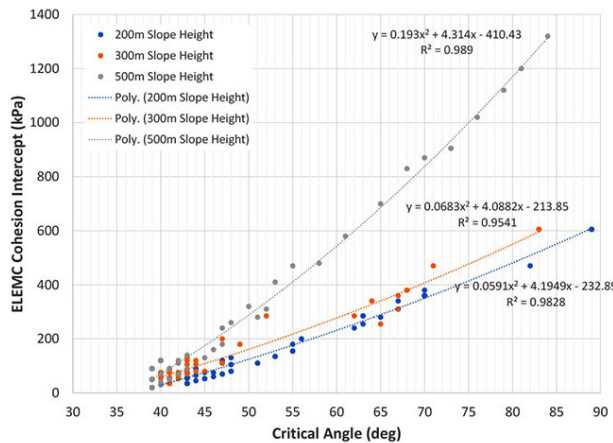


Figure 9—Cohesion vs. critical angle for 200 m, 300 m, and 500 m slope heights

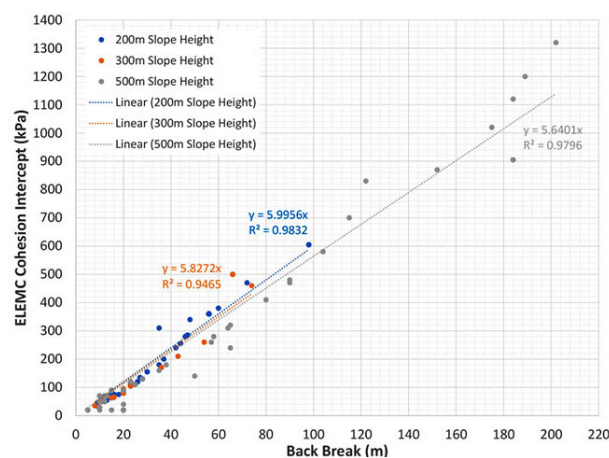


Figure 10—ELEM C cohesion intercept vs. backbreak

$$\text{friction angle} = -0.0003 \times \sigma_{ci}^2 + 0.0736 \times \sigma_{ci} + 33.211 \quad [2]$$

Equivalent limit equilibrium Mohr-Coulomb cohesion

The first step in deriving the ELEM C cohesion values was to find the correlation between the critical angle and the ELEM C cohesion for the LE results, these relationships are presented as Figure 9, with the matching equations for each slope height scenario. Some scatter is evident at lower critical angles due to the wide possible range of backbreak obtained in low slope angle finite element models.

The results also showed that a correlation is available between the back break distance and the ELEM C cohesion, as presented in Figure 10. This demonstrates that back break distance, cohesion, and the critical angle are closely related.

As the critical angle and the ELEM C cohesion intercept is correlated, and the critical angles were determined to the nearest 1°, the equation corresponding to the appropriate slope height scenario in Figure 9 was used to determine the ELEM C cohesion for the RS2 scenarios.

In summary, the ELEM C friction angles were determined from the FE scenario σ_{ci} using Equation 2, and the ELEM C cohesion intercept was determined from the appropriate equation in Figure 9. This allowed the compilation of a data table with 3 840 scenarios that matches the Hoek-Brown parameters to the ELEM C parameters through the back break and critical angle.

Analysis of the 3 840 finite element scenarios

The previous section showed how the LE and FE analyses could

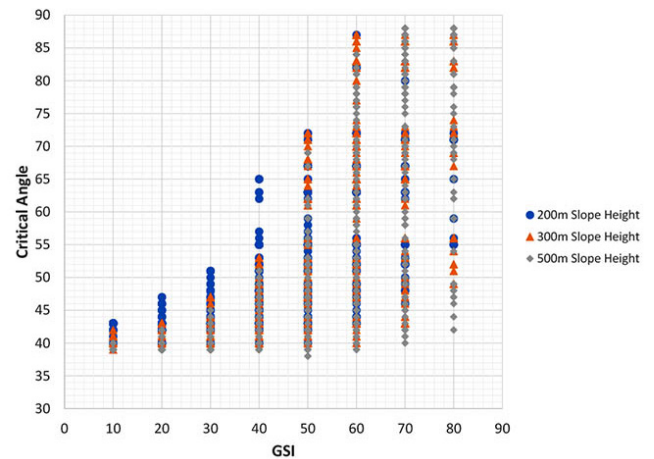


Figure 11—Example scatter plot of finite element results

be matched to obtain an ELEM C friction angle and cohesion intercept for each Hoek-Brown FE scenario. This section presents selected figures from the resulting database of matching σ_{ci} , GSI, m_i , D, cohesion and friction angle groups, and derives simplified regression equations to estimate the ELEM C parameters from the data table. During the batch runs, 445 of the model result files became corrupted and were discarded. Most of these were during the 200 m slope height runs that would have resulted in critical angles of greater than 70 degrees. Furthermore, an additional 318 of the models did not produce a critical angle as the slope height and slope angle combination was not sufficiently high/steep to cause slope instability.

The first relationship to be displayed is that of the critical angle vs. GSI in Figure 11. Figure 11 shows that the maximum slope angle for all three slope height scenarios (200 m, 300 m, and 500 m) reached 89°, which means that for the stronger rock mass scenarios, the slope heights were not high enough to trigger instability, and the critical angle reported in Figure 11 is merely where the analysis stopped, not where the slope failed. The 200 m case has a higher critical angle for GSI values below 50, as the lower slope height requires less shear strength to be steeper than higher slope heights. A similar difference can be seen between the 300 m and 500 m cases.

The resulting plateau in critical angles can clearly be seen in Figure 12 that shows the average critical angle for all D and σ_{ci} scenarios corresponding to each GSI value for the 3 slope heights.

As a result, one can expect to see a limit to the ELEM C cohesion values corresponding to each slope height. This is presented as the red arrows in Figure 13, which shows the ELEM C cohesion intercepts vs. GSI. These limits are 1 480 kPa for 500 m slope scenarios, 920 kPa for 300 m slope scenarios, and 620 kPa for 200 m slope scenarios.

To understand the effect of the Hoek-Brown D value on the ELEM C cohesion intercept, a GSI vs. ELEM C cohesion intercept sensitivity is presented for the two D value scenarios, as shown in Figure 14.

The two lines represent the average of all m_i values for each GSI and D combination based on a σ_{ci} value of 100 MPa. From Figure 14 reducing the D value from 1 (open pit disturbed) to 0.7 (open pit undisturbed) increases the ELEM C cohesion intercept by as much as 300 kPa for GSI values between 50 and 60, but the difference is smaller for GSI values below 50 and above 60. The result is that D has a greater effect on ELEM C cohesion and could potentially be a lever that can be used to calibrate this model against case studies.

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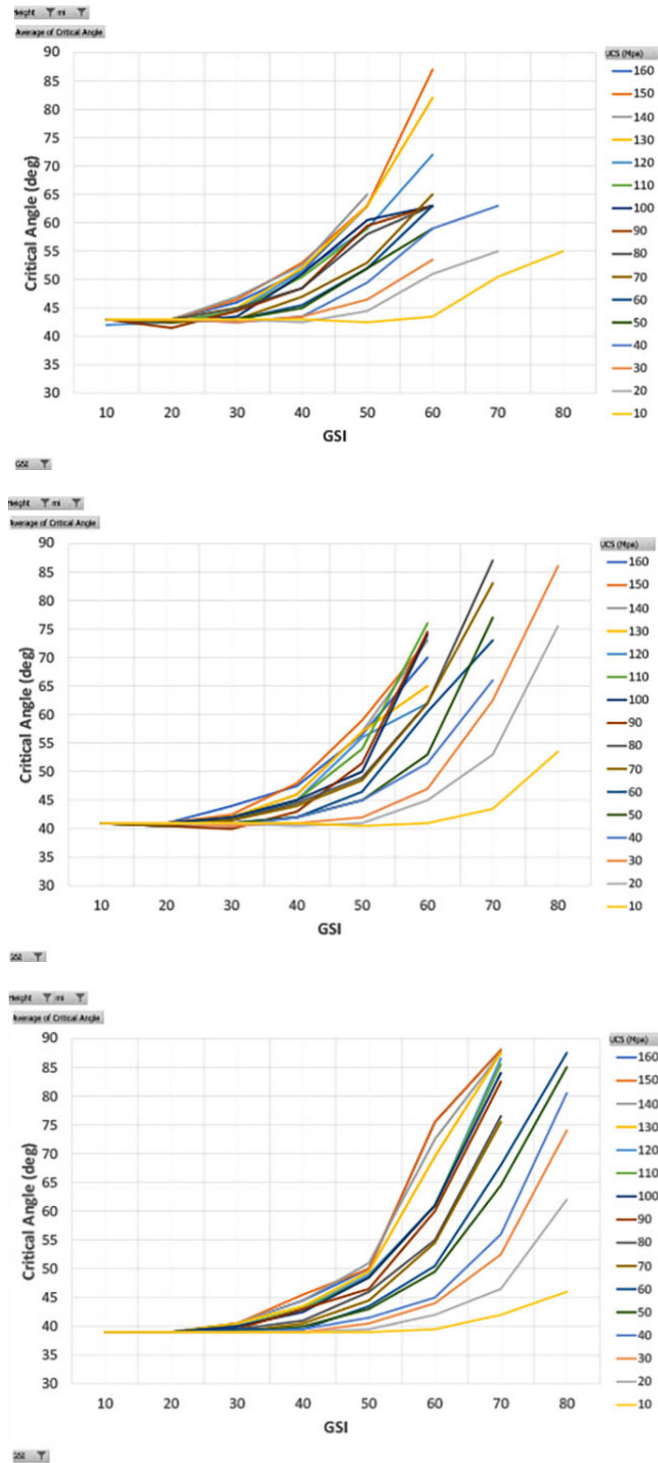


Figure 12—Average critical angle vs. GSI (200 m slope height left, 300 m slope height centre, 500 m slope height right)

Similarly, the effect of the Hoek-Brown m_i value was evaluated and is shown for different m_i values plotted in Figure 15, representing a σ_{ci} value of 100 MPa and a D value of 0.7.

Figure 15 shows that the ELEMCO cohesion intercept is less sensitive to changes in m_i value than the D value. For GSI values above 65 and below 50 the ELEMCO cohesion intercept is not sensitive to m_i .

In summary, this section showed that the 3 077 (3 840 -445 - 318) sets of matching ELEMCO and Hoek-Brown parameters resulting from the LE to FE comparison can be used to obtain

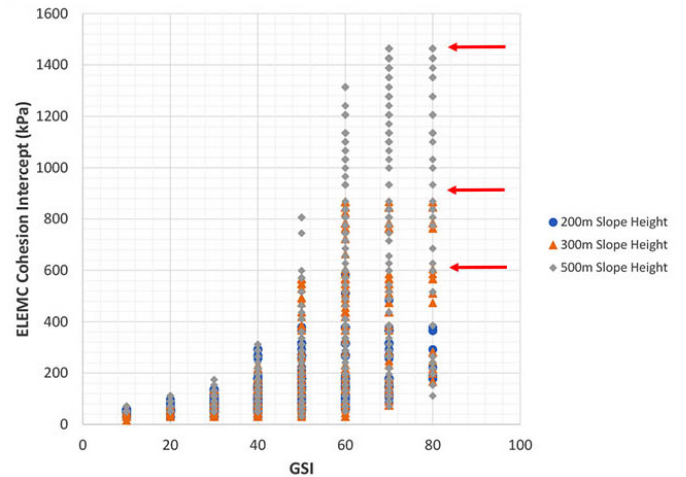


Figure 13—Equivalent LE cohesion vs. GSI

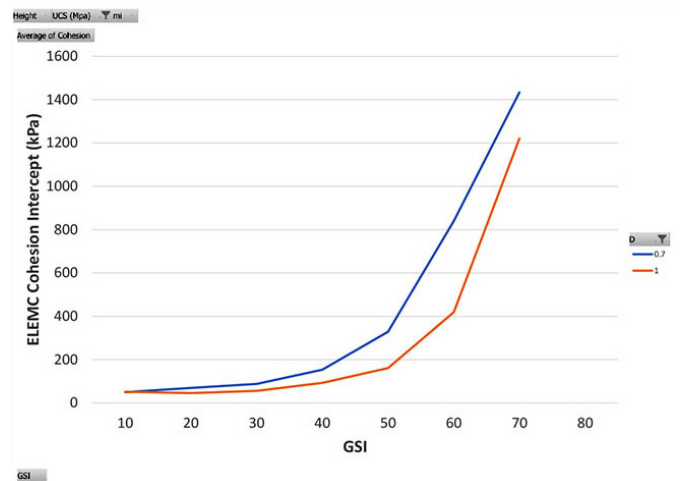


Figure 14—Example curve for 100 MPa σ_{ci} for $D = 0.7$ and 1

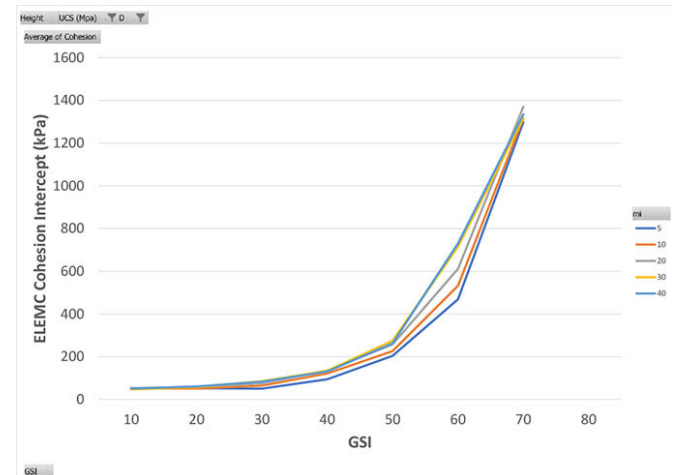


Figure 15—Example curve for 100 MPa σ_{ci} showing effect of m_i values

ELEMCO parameters based on Hoek-Brown parameters. The results show that the ELEMCO cohesion intercepts are sensitive to the Hoek-Brown D values, but less sensitive the Hoek-Brown m_i value. The ELEMCO friction angles could be determined by the simplified method presented already and so were not considered in this section. The next section analyses the results to determine regression equations that can be used to provide ELEMCO parameter estimates.

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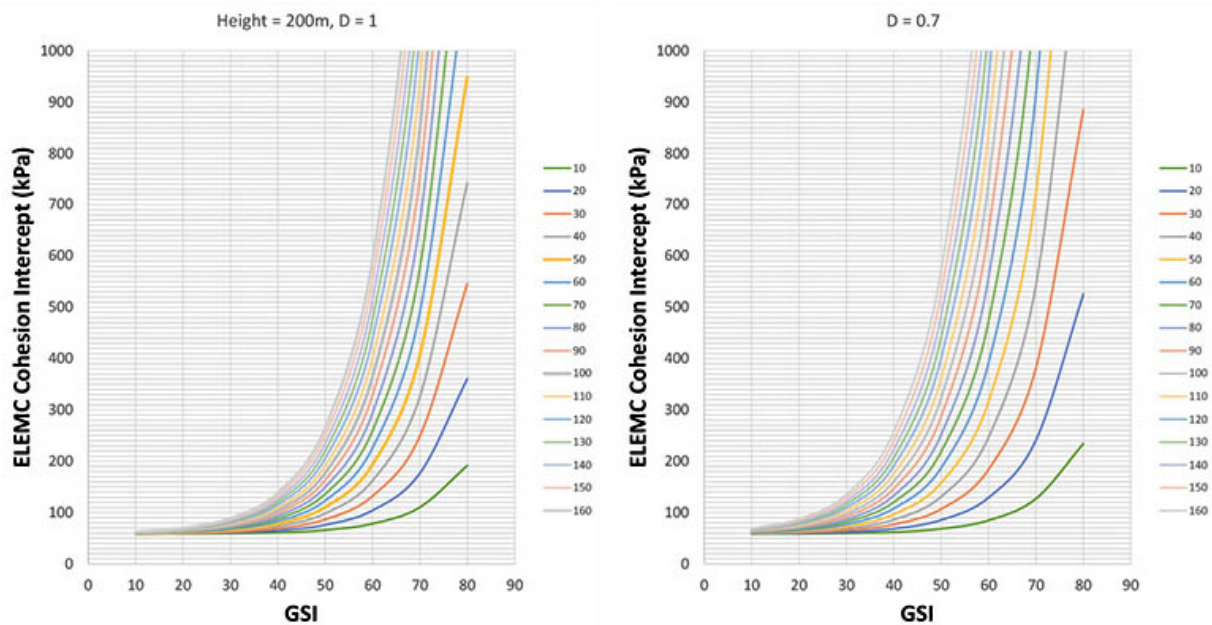


Figure 16—The 200 m regression curves for different σ_{ci} values

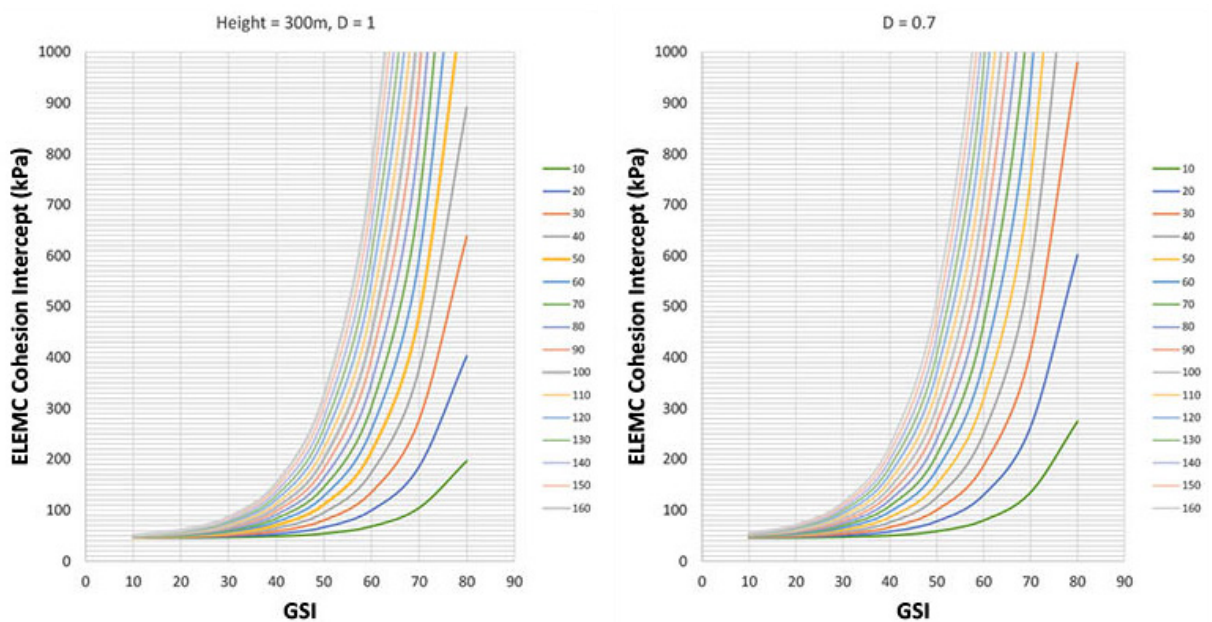


Figure 17—The 300 m regression curves

Regression curves

The true value of using the Hoek-Brown to ELEM approach is that it allows for ongoing calibration of the design curves as more case studies become published, by updating the Hoek-Brown D parameter in the regression equations. It is the authors' view that as more large open pit failures are reported, the additional data will facilitate fine tuning of the results that can then be fed back into the design of slopes. Towards this calibration, the logistic regressions used to fit the initial database are shown in Equation 3 and Equation 4 with regression coefficients compiled in Table 1.

Figure 16, Figure 17, and Figure 18 show the resulting cohesion design charts based on the Hoek-Brown to ELEM analyses for slope heights of 200 m, 300 m, and 500 m, and D values of 0.7 and 1. The design charts were created by smoothing the results to remove irregularities caused by the granularity in the results, while still maintaining the integrity of the overall pattern.

Table 1

Coefficients for calculating L

Slope height (m)	D	a	b	k	x_0
200	1	405.27	1.1783	0.095	120
	0.7	316.59	1.4062		
300	1	398.04	1.2409	0.095	120
	0.7	551.61	1.2789		
500	1	540.17	1.1665	0.095	120
	0.7	496.33	1.3394		

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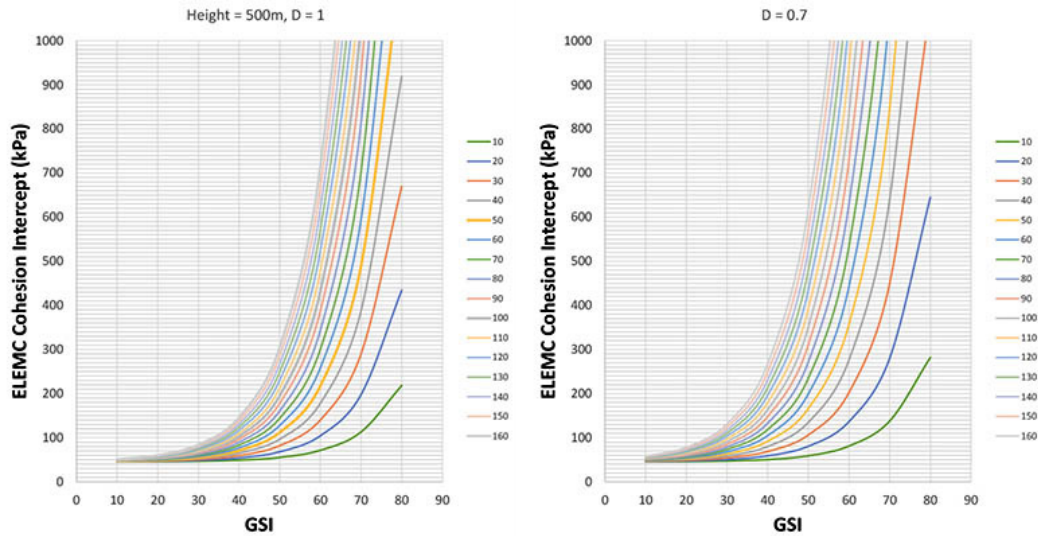


Figure 18—The 500 m regression curves for different σ_{ci} values

The Logistics curve was used for the ELEMCOhesion intercept (in kPa) with trial-and-error fit to match all combinations of the Hoek-Brown parameters. The results equation for ELEMCO is:

$$c = \frac{L}{1 + e^{-k(GSI - x_0)}} \quad [3]$$

Where L is given by:

$$L = a \times \sigma_{ci}^b \quad [4]$$

The coefficients needed are presented in Table 1 that follows.

As a mathematical regression could not be used due to the number of variables and the complexity of the shape, a regression coefficient could not be calculated, but a series of manual 2-dimensional approximate relationships were developed. These approximate relationships are reasonably close to the analysed results, especially for GSI values below 60, but differences do occur. As a result, it will always be preferable to repeat this process for the specific material parameters encountered in a project. This will also allow the measured value of m_i to be included in project specific calibration of properties if more accurate ELEMCO parameters are needed.

Discussion

This paper sets out to compare the use of strain softening shear strength parameters, which more accurately represent rock mass behaviour than peak strength parameters typically used, against the use of equivalent limit equilibrium shear strength parameters (ELEMCO) that provide the same critical slope angles and back break distances as the strain softening parameters. The results of the comparison demonstrate some important lessons. These are discussed in the following.

The equivalent ELEMCO parameters as determined in the previous chapter are roughly in the same order of magnitude as the back analysed parameters proposed by Hoek and Bray (1981). This is valid for both the friction angles (30° to 40° for the equivalent LE values vs. 30° to 40° for the back analysed parameters) and the cohesion values (up to 1500 kPa for the equivalent LE values vs. up to 550 kPa for the back analysed values). The cohesion values appear somewhat different at first glance, however, the 1480 kPa result is for a slope with values for $\sigma_{ci} = 150$ MPa value and GSI = 80.

These parameters are so strong that they are not represented in the Hoek and Bray (1981) database. Limiting the GSI value to a more common 60, and the σ_{ci} to 100MPa, provides an ELEMCOhesion intercept estimate of 700kPa, which is not far off the Hoek and Bray (1981) database. Similarly, a comparison with the back analysed correlation presented in Singh and Goel (2011) back analysed values shows a similar agreement.

The models provided are correlated with the Hoek Brown peak shear strength criterion in combination with strain softening analysis using a residual friction angle of 38° . This makes them more realistic than estimates determined using the Hoek-Brown peak shear strength criterion alone. However, as the curve fitting process still has room for improvement, and only considers correlation against Hoek-Brown parameters, these models will still benefit from calibration against back analysis. The authors envisage investigating such a calibration for future publication, for this purpose the model was set up in such a way that readers can calibrate the model against their own data by changing the coefficients a and b in Table 1 representing different D values.

The results showed that the Hoek-Brown m_i value only has a minor role to play as demonstrated in Figure 15, which means it can be argued that for smaller projects, or early design stages, the value of m_i can be ignored, or triaxial testing postponed until later stages of the project. Perhaps more value will be had by using the available budget for more UCS testing.

The model can be used in three ways:

- Equivalent LE friction angles can be read off Figure 8 directly, and equivalent cohesion values can be read off Figures 15 to 17 once the appropriate slope height, GSI, σ_{ci} and D is selected.
- The equivalent LE friction angles can be calculated using the Equation 1, and the equivalent cohesion can be determined by selecting the appropriate coefficients in Table 1, calculating L using Equation 4 and then the equivalent cohesion using Equation 3.
- An unintended benefit that arose from the results is that for existing slopes where cracks had formed, a rough estimate of the equivalent LE cohesion can be made using Figure 10. Interestingly, this relationship is agnostic of slope angle and slope height.

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Using this approach assumes the user agrees that Hoek-Brown is correct, an m_i value of 20 is appropriate and agrees with the RS2 modelling approach used, and the residual friction angle of 38° . Should the user not accept these assumptions or would prefer to calibrate their LE analysis using project specific data or more rigorous stress analysis, the approach described in this paper can be replicated. The important part of the approach is that the critical angles and backbreak must match in corresponding models, not the FS values.

It is to be noted that, this research did not simulate the effect of ground water on the results, this is left for potential future publication.

Conclusion

In conclusion, this paper fulfilled its stated objectives by deriving equivalent LE Mohr-Coulomb parameters for open pit slopes based on strain softening finite element analysis. The results demonstrated that the equivalent LE Mohr-Coulomb parameters provide more realistic slope angles than those obtained when using Hoek-Brown peak shear strengths in limit equilibrium software. It is therefore suggested that more appropriate limit equilibrium shear strengths will be obtained by calibrating against strain softening numerical models for a project using the methodology presented in this paper. In the absence of strain softening models, the simplified results presented in this paper can be considered to provide early estimates of the ELEM friction angle and cohesion intercept.

Following the work presented in this paper, several shortcomings that leave room for further investigation are:

- The ELEM parameters are correlated with elastic linearly plastic Hoek-Brown parameters, however, in reality rock masses are not linearly plastic. Different results may be obtained if correlated with strain softening models that provide a graded transition from peak to residual strengths.
- The ELEM parameters in this paper are easily calibratable but have not been calibrated yet. Further work in this regard is needed.
- There is also room for improvement in regression analysis of the database. The authors have exhausted their resources to find these curve fits, but ideally, access to software that can handle regression with 8 or more variables is needed.
- Groundwater was also not investigated and needs to be considered.

We look forward to seeing these issues investigated in future. With this in mind, interested readers are encouraged to collect databases containing the Hoek-Brown parameters: GSI, σ_{ci} , m_i , and D values corresponding to back analysed ELEM cohesions and friction angles for both failed and stable slopes, and contact the authors.

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