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Shear Strength of Closely Jointed Porphyry Rock Masses

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ABSTRACT: *Stability analyses of slope failures in open pit mines over a 15-year period have been re-analyzed by the author to assess whether a quantitative relationship can be developed between rock structure models and rock-mass shear strength. From a review of a large number of case histories, a set of 16 documented slope failures were chosen for analysis of structure-strength correlation. All of the analyzed slope failures occurred in closely jointed rocks with RMR values of less than 50. The exponentially increasing cohesion model originally proposed by Jaeger during the 1971 Rankine lecture was adopted to model shear strength for each rock mass. Input parameters for the new system include RQD, intact rock shear strength, joint roughness, joint shear strength and joint-set statistics. The proposed system was used to evaluate rock strength anisotropy and to estimate values of rock-mass shear strength along strongly developed joint systems. The resulting database does show that a general correlation can be made between rock-mass shear strength and joint-set data. The estimates of rock-mass shear strength made from the structural model correlate well with previously published data, including the method of strength estimation that has been proposed by Hoek and Brown using RMR and GSI.*

1. INTRODUCTION

The estimation of rock-mass strength continues to remain one of the more formidable issues that confronts engineers designing structures in rock. As many engineers are aware, the choice of rock-mass shear strength for slope stability analysis requires significant engineering judgment, although several methods have been proposed for assessing rock-mass shear strength from basic geologic data (Bieniawski (1989) [1], Hoek and Brown (1980) [2], Hoek, Wood, and Shah (1992) [3], Robertson (1988) [4]). With carefully constructed laboratory testing programs, both strength and deformability of rock matrix and rock joints can be adequately assessed. However, the methodology used in calculating rock-mass strength from the tested rock matrix and joint shear strengths is a subject that requires additional research.

Although numerical modeling and *in situ* test data may ultimately provide tools that are capable of providing reliable estimates of the state of shear strength mobilization in rock masses, consideration should also be given to past experience with failed and stable rock structures. Empirical methods based on careful observation also have the potential for establishing methods that can be used for evaluating rock-mass strength for design. Dr. Richard Call had long advocated the incorporation of rock structure analysis into the formulation of rock-mass strength, and to this end, the author has sought to carefully develop additional correlations between mobilized rock-mass friction and rock structure data for rock slope design in mining projects. The emphasis in this paper is on the development of rock-mass strength parameters for specific orientations parallel to persistent joint sets. It is not the intent of this paper to propose a new method for estimating a general shear strength criterion for all directions through a rock mass, but rather to supplement the published case history database and provide a simple method of anisotropic rock-mass strength evaluation for preliminary slope design in using structural data as a primary input.

2. STATEMENT OF THE PROBLEM

Frequently, thick zones of low-quality rock are geographically associated with many ore zones, presenting the mining engineer with ground stability problems in both underground and open pit excavations. Regional-scale tectonics, contact metamorphism, and post-mineralization alteration all contribute to the degradation of many rocks within and surrounding both base metal and precious metal deposits. Many of these rocks possess a Rock Mass Rating (RMR) of less than 40, with a drill-hole RQD of less than 30 percent. At many of the mines, rocks that must be excavated in the process of mining can be very intensely fractured (RQD less than 10 percent), and often contain clay alteration along the rock joint surfaces and within the rock matrix. These highly fractured rocks are weak and prone to displacement.

In these closely jointed rocks, structural mapping of the local outcrops along mine benches typically identifies five to eight joint-set orientations at any one locality. Regional mapping data throughout the porphyry copper deposits published by many authors indicate that this is typical for most of the porphyry rocks (Titley, 1982) [5]. The significance of these data is quite clear: the jointing and joint patterns in these rocks are not "randomly" oriented. In regions within the mine, there are certain orientations along which jointing has been very strongly developed, and these strong joint orientations present a direction along which the rock mass is significantly weaker than "average." The strong joint-set orientations are generally the result of plate tectonics on a local and regional scale, and are often related to the hypogene mineralization process. Rock-mass strength models, which use general rock-mass parameters and which do not account for the anisotropic character of the rock mass, may or may not lead to conservative designs. In order to properly evaluate rock-mass strength for design of excavations in the closely jointed porphyry rocks, it has been found necessary to incorporate the structural model (commonly referred to as the rock fabric) into the evaluation of rock-mass shear strength.

3. A ROCK-MASS SHEAR STRENGTH MODEL FOR CLOSELY JOINTED ROCKS

As one moves from the small scale of the rock matrix to the intact rock block and then to the rock mass, the shear strength of rock is reduced. This is commonly known as the scale effect, and is generally attributed to the increasing number of discontinuities that are present in the rock at the larger scale, although fundamental strain energy relations can also explain the reduction of strength at the larger scale (Farmer, 1986) [6]. It is assumed that for most rock masses, the size of a representative sample is primarily a function of the rock jointing patterns (spacing and length). Based on both rock-joint modeling and practical experience, this representative size lies somewhere between 5 and 20 meters depending on the actual character of the structures in the rock. At that scale, it is proposed that the strength of the rock mass is dependent on three primary factors:

(i) The strength of the joints
(ii) The strength of the intact rock blocks between joints
(iii) The degree of interlocking between intact rock blocks

For a massive rock with few or no joints (RMR=100), the rock-mass shear strength approaches that of the intact rock blocks. However, experience indicates that for a very closely jointed rock (RMR<40), the rock-mass shear strength is reduced to within one order of magnitude of the cohesion strength of the rock joints. Therefore, the intensity of jointing, the character of the rock between joints, and the degree of block interlock are all critical geologic properties for estimating rock-mass shear strength. However, since the joint patterns in the porphyry rocks are not random, the degree of interlocking between blocks of rock is quite dependent on joint orientation, hence, the strength of the rock mass can not be described by a general Mohr-Coulomb strength model. In these cases, it is preferable to implicitly incorporate the structural model into the formulation of rock-mass shear strength for slope stability analysis.

It is evident in 2D and 3D joint system modeling that the intensity of joints in the rock mass (joint area per unit volume or joint length per unit area) is a function of the length to spacing ratio. A simple illustration of this principle is shown on Figure 1, which shows the changes in joint intensity in a two dimensional joint trace model developed for a single joint set. As the length to spacing ratio increases, there is a non-linear increase in joint intensity irrespective of whether the negative-exponential, Weibull, or log-normal distributions are employed for modeling joint length and spacing. For the more commonly used negative-exponential or Weibull models, the increase in joint intensity with increasing joint length to spacing ratio takes the form of a power or exponential curve.

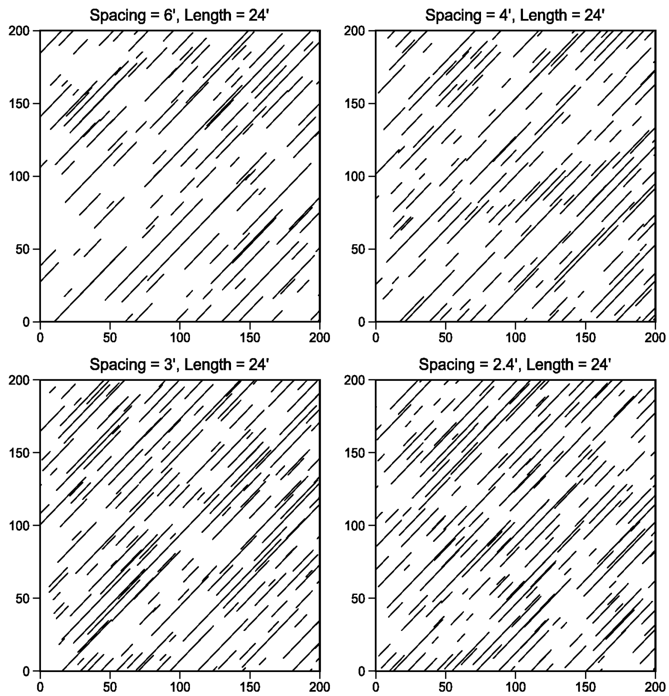

Figure 1. Intensity of Joints as a Function of the Length to Spacing Ratio

Since the joints represent major weakness planes in the rock mass, these features often control the deformation of the rock mass. The nature of the porphyry rock fabric lends itself to stepped failure surfaces where deformation occurs along a complex path consisting of a series of joints, which individually are separated by one or more blocks of rock. The rock-mass bridges between joints provide a strong interlocking effect upon the rock mass. However, it is not clear how the intact rock strength within the bridges between joints along stepped failure paths is mobilized, even in carefully controlled laboratory tests (Bro, 1992) [7]. The assumption that the intact rock strength is fully mobilized within the rock bridges between joints would appear to be an oversimplification, but certainly some intact rock strength is mobilized in the bridges. Simple physical models tested in the laboratory indicate that deformation within the bridges can occur with intact rock shear, rock block rotation, intact rock tensile failure, and simple tensile failure along cross cutting structures.

Model studies tend to show a reduction in shear strength when the maximum shear stress direction is closely aligned with a persistent joint set. In these relatively simple experiments using orthogonal joint systems (rock mass simulated by square blocks), compressive strengths of the block models range from a high of 50 percent of the intact rock block strength to a low of 10 to 13 percent of the intact rock block strength, depending on the loading direction relative to the jointing. As the principal joint set becomes aligned with the maximum shear stress direction ($45 + \varphi/2$), shear behavior changes from a discrete path consisting of block and joint shear to a shear zone within the model. As more joint orientations are cut into the model, the size of the shear zone increases, and deformation is accommodated by block rotation as well as shear distortion. In response to the behavior evident in the model studies, Ladanyi and Archambault (1970) [8] proposed three types of shear failure for closely jointed rock masses: kink band, shear zone, and shear plane.

(i) **Kink Band –** When the normal stress is low, and the rock is very closely jointed, very little intact rock breakage occurs along the shear surface. For rock joints at low normal stress, intact rock asperities are not sheared through but are ridden over. In a rock mass, a similar phenomenon exists where the rock bridges are ridden over. In this case, the rock bridges fail by shear distortion and block rotation. The effect of this is an increase in the mobilized friction angle of the rock mass with a corresponding increase in the dilation angle at "failure." This effect is similar to that modeled by N. Barton [9] for rough joints at low normal stresses. For rock masses, kink band failure of the rock bridges becomes more significant when the rock mass is intensely fractured and the individual pieces of intact rock in the rock bridges are poorly interlocked.

(ii) **Shear Zone –** As normal stress increases or the rock bridges become more interlocked, the shear behavior between the individual joints becomes more like that of a shear zone. Within the shear zone, rock-mass failure occurs due to a complex combination of intact rock breakage, block rotation, and shear distortion.

(iii) **Shear Plane –** At higher normal stress, the rock mass within the rock bridges becomes confined, increasing the effective interlocking between individual rock pieces. Eventually, enough interlocking develops to prevent significant block rotation within the bridges, so shear failure can only occur as a shear plane through the intact rock blocks between joints. In this case, dilation angles at "failure" are low, and the ultimate shear strength is developed within the rock bridges.

Barton's published results for tests on models of 250 to 5000 blocks also showed similar kink band - shear zone behavior in biaxial loading. However, the boundaries between these three shear behaviors is poorly understood and difficult to quantify. It is likely that the combination of these three types of shear behavior accounts for the strong non-linear relationship between normal stress and shear strength for rock masses at low stress levels.

Experience has shown that not accounting for the non-linear relationship between strength and stress at low normals can lead to non-conservative results in slope design (Call, 1985) [10]. In agreement with Call, the author's experience indicates that non-linear shear strength models are needed in slope design. After a number of years of experimentation with several mathematical models of shear strength, the non-linear shear strength law proposed by Jaeger (1971) [11] was adopted by the author for rock-mass shear strength evaluation of the porphyry rocks:

$$\tau = (C_f - C_m)*\{1-\exp(-b\sigma_n^{m})\} + \mu_f\sigma_n + C_m \qquad (1)$$

where

- τ = mobilized shear strength of the rock mass in a specific orientation
- σ_n = normal stress
- μ_f = coefficient of friction for the fully interlocked rock mass
- C_f = ultimate cohesion strength for the fully interlocked rock mass
- C_m = minimum cohesion strength for the fully disturbed rock mass
- m = joint shear strength mobilization parameter
- b = rock-mass interlock factor

In the model, it is proposed that there is a state of full rock-mass interlock in which the mobilization of intact rock strength is at a maximum. This state is similar to Ladanyi's shear plane case. However, this state is not necessarily achievable at low normal stresses, where shear zone and kink band behavior is believed to be more prevalent for the rock bridges. This theory of shear strength is analogous to the simple sawtooth model for joint shear strength where blocks ride over asperities at low normal stress and shear through asperities at high normal stress. The proposed exponentially increasing cohesion model attempts to model this behavior as a function of increasing normal stress.

4. DEVELOPMENT OF THE ROCK-MASS STRENGTH MODEL

To evaluate the shear strength of the closely jointed porphyry rock masses, the following procedure has been followed for a specific joint-set orientation:

(i) The maximum amount of intact rock strength that can be mobilized along a specific persistent joint orientation is evaluated on the basis of the structural model (joint-set length and spacing), and the rock-mass shear strength for the fully interlocked or fully mobilized case is developed from the combined joint and intact rock shear strengths.

(ii) An estimate of the stress at which the transition to a fully interlocked condition occurs is made with general rock-mass parameters (RQD, number of well developed joint sets).

(iii) The extent of interlocking that is developed at stress levels below the transition stress to a fully interlocked condition is estimated on the basis of joint condition (joint roughness and basic residual friction angle).

Figures 2, 3, and 4 present the data required to develop a rock-mass shear strength along a persistent joint orientation using the proposed model.

Step 1 - Estimating the Fully Interlocked Shear Strength

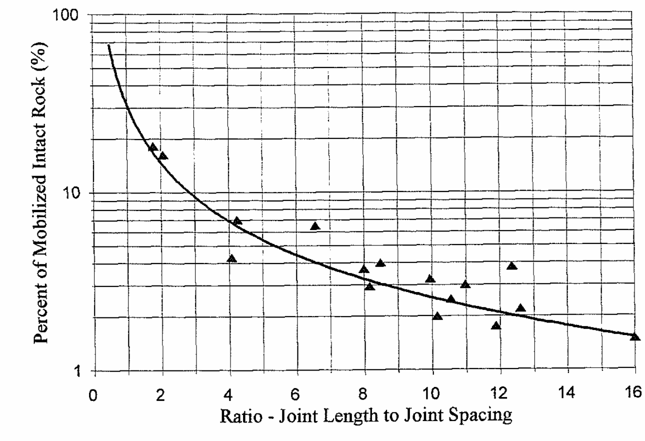

Figure 2. Estimating the maximum percent intact rock strength that can be mobilized.

Although it is difficult to predict the exact character (shear plane, shear zone, or kink band) of the shear strength mobilized in the rock bridges between joints, it is evident from structural models that the percentage of rock bridges along a persistent joint orientation decreases as joint intensity increases. Assuming that the percentage of intact rock strength that is mobilized between joints is proportional to the joint intensity, an estimate of the percentage of intact rock strength mobilized along a failure path would exponentially increase as the joint length to spacing ratio decreases. Using the relationship between joint intensity and joint length to spacing ratio as a guide, an empirical model of the maximum amount of intact rock shear strength that can be mobilized along a persistent joint orientation has been developed by the author with careful back-analysis of failed excavation geometries. The author's best estimate of the maximum percentage of intact rock that can be mobilized along a persistent joint orientation is shown on Figure 2 as a function of the joint-set mean length to mean spacing ratio. A simple power regression to the back-analysis data has yielded an approximate relationship of:

$$\text{Max. Percent Intact Rock} = 0.315(L/S)^{-1.094} * 100\% \quad (2)$$

where

L = mean length of the persistent joint set using the negative-exponential or Weibull model

S = mean spacing of the persistent joint set using the negative-exponential or Weibull model

The range of data used to develop this curve included length to spacing ratios from 1.8 to 14.6. If the length to spacing ratio falls outside of this range, then considerable caution is advised. In addition, since there is some scatter to the data (as one would expect from geologic data), some engineering judgment is needed in estimating the percent of intact rock along the failure path.

Once an estimate of the maximum percentage of intact rock that can be mobilized has been estimated (A), the fully interlocked (shear plane case) rock-mass shear strength can then be calculated using a weighting method where:

$$\text{Rock-mass strength} = (1-A) * \text{joint shear strength} + (A) * \text{intact rock shear strength} \quad (3)$$

Step 2 - Estimating the Stress Required to Develop Full Interlock

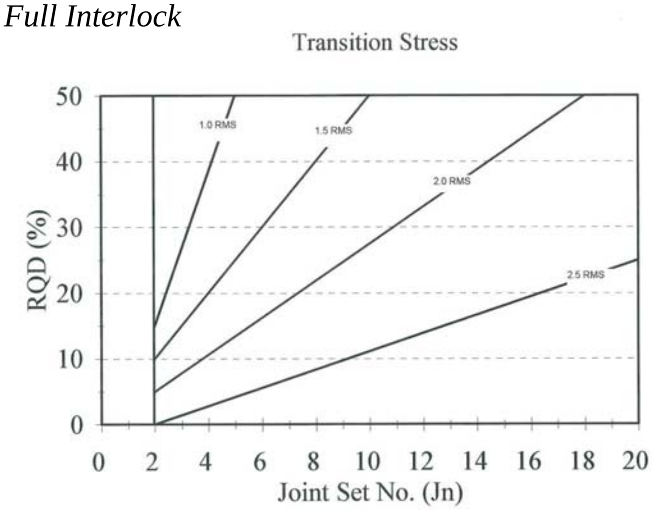

Figure 3. Transition stress to a fully interlocked state as a ratio (R) of Laubscher's RMS.

The transition stress at which the full interlock is developed has also been evaluated empirically. A useful method of relating this stress to the strength of the rock is presented on Figure 3, where the transition stress is evaluated as a ratio of the fully interlocked rock-mass compressive strength. Laubscher, in his 1990 paper [12], refers to this fully interlocked compressive strength as the RMS. As the number of joints in a rock mass increase, the average block sizes in the rock mass decrease. As the number of well developed joint sets increase, more degrees of freedom are available for rock block movement. In both cases, this is believed to result in a looser, less interlocked rock mass, which requires higher levels of stress to develop mobilization of intact rock strength within the bridges between joints.

For evaluating the transition stress as a function of rock-mass quality, RQD was chosen as an index of fracture frequency and the Joint Set Number from the Q classification system (Barton, Loset, Lien, and Lunde, 1974) [13] was chosen for representing the effect of multiple joint sets on rock-mass strength. Experience has shown that the {RQD/Jn} parameter is very useful for evaluating the effect of rock fabric on the strength of the rock mass. Figure 3 requires some engineering judgment to find the transition stress to a fully interlocked condition. Once the ratio (R) of transition stress to RMS has been chosen from the charts, the transition stress can be calculated using the obliquity relations:

$$\text{transition stress} = R * \{2 * C_f * \tan(45 + \varphi_f/2)\} \quad (4)$$

where

R = the simple numerical ratio from Figure 3

C_f = fully interlocked cohesion for the Mohr-Coulomb strength model

φ_f = fully interlocked friction angle for the linear Mohr-Coulomb strength model

Step 3 - Estimating the Minimum Rock-Mass Cohesion

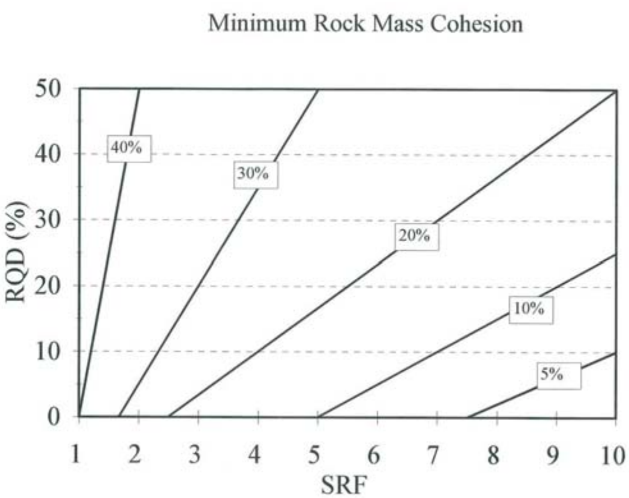

Figure 4. Minimum Rock-Mass Cohesion as a Percentage of Fully Interlocked Rock Cohesion (C_f)

Because most joint failure paths are not continuous, there remains some cohesion effect between joints even at very low normal stress levels. This minimum cohesion effect can be seen in many benched slopes within open pit mines. Experience indicates that some residual shear strength exists at zero normal stress even for a path with very well developed, persistent jointing. The strength (C_m) at such low stress along a joint path is affected by the interlocking effect of the rock blocks between joints. This is difficult to quantify and requires some knowledge of the geology as well as some engineering judgment.

Step 4 - Estimating the Degree of Intact Rock Strength Mobilization at Low Stress Levels

Once the ultimate or fully interlocked shear strength, the stress level at which the fully interlocked strength occurs, and the residual interlocked shear strength at zero normal stress have all been evaluated, the remaining factor left to be determined for the shear strength model is the manner in which the interlocking and intact rock strength is mobilized at lower stresses. This is accomplished with a curve factor, which controls the shape of the shear strength envelope at lower normal stresses. Experience indicates that the key to the interlocking mobilization is the roughness of the joints along the failure path. Since rougher joints cause stronger dilation in shear than planar joints, interlock is more easily developed for the rougher joints. At lower stresses, where joint wall strength and intact rock strength are not fully mobilized, the roughness of the joints is a large factor in the shear strength model.

One method of evaluating the curve factor is to use the results of direct shear tests conducted on natural joints. As discussed by Jaeger (1971), Barton (1976), and Call (1985), joint shear strength at low normal stresses often takes the form of a power curve. One simplified approach to modeling this non-linear strength envelope is a simple power law of the form:

$$\tau = K\sigma^m \quad (5)$$

When test data are available within the range of normal stress that is of interest, the simple power law has proven to be a very useful model of rock joint shear strength for slope stability analysis (Call, personal communication). Jaeger reported shear strength data that fell within the range of $\tau = 1.2\sigma^{0.9}$ and $\tau = 5.2\sigma^{0.7}$. Experience with shear strength testing conducted or commissioned by the author has indicated that “m” factors can range from 0.60 to 0.95 for natural fractures tested at normal stresses between 0.1 and 2 .0 MPa.

For evaluating rock-mass strength using the proposed model, it is recommended that joint shear test data be used to evaluate the curve factor. If test data are unavailable, it is also possible to develop the curve factor from a regression analysis of the shear strength predicted by the Bandis-Barton model for normal stresses less than the joint wall compressive strength (JCS). For any data set, a regression can be performed to determine the best fit to the simple power law by the least squares method. This can be readily accomplished by linearizing the power law equation by taking the logarithms.

5. EXAMPLE APPLICATIONS OF THE MODEL

For evaluation of the key parameters in the model, the following data are required:

(i) The basic friction angle of the rock joints along the failure path (φ_j)

(ii) The small-scale roughness of the joints along the failure path (Jr, JRC)

(iii) The shear strength of the intact rock (φi, Ci)

(iv) An estimate of the “m” coefficient for the simple power strength law ($\tau = K\sigma^m$) for the joints along the failure path

(v) An estimate of fracture intensity for the rock mass (RQD), and the number of well developed joints present in the rock mass (Jn)

(vi) The mean joint length and spacing for the joint set of interest

Example #1 – Evaluating Critical Weakness Orientations in the Rock Mass

In one area of a copper mine, a series of normal faults strike parallel to the pit slope. These faults have a dip range between 45 and 60 degrees. The average dip is 50 degrees. The faults are clay filled and will clearly be unstable if daylighted by the mining at the toe of the pit slope (Fig. 5). However, it is also possible that the stresses developed at the toe of the slope could result in a slope failure even if the faults are not daylighted by the mining if the rock mass at the toe of the slope is highly fractured and susceptible to

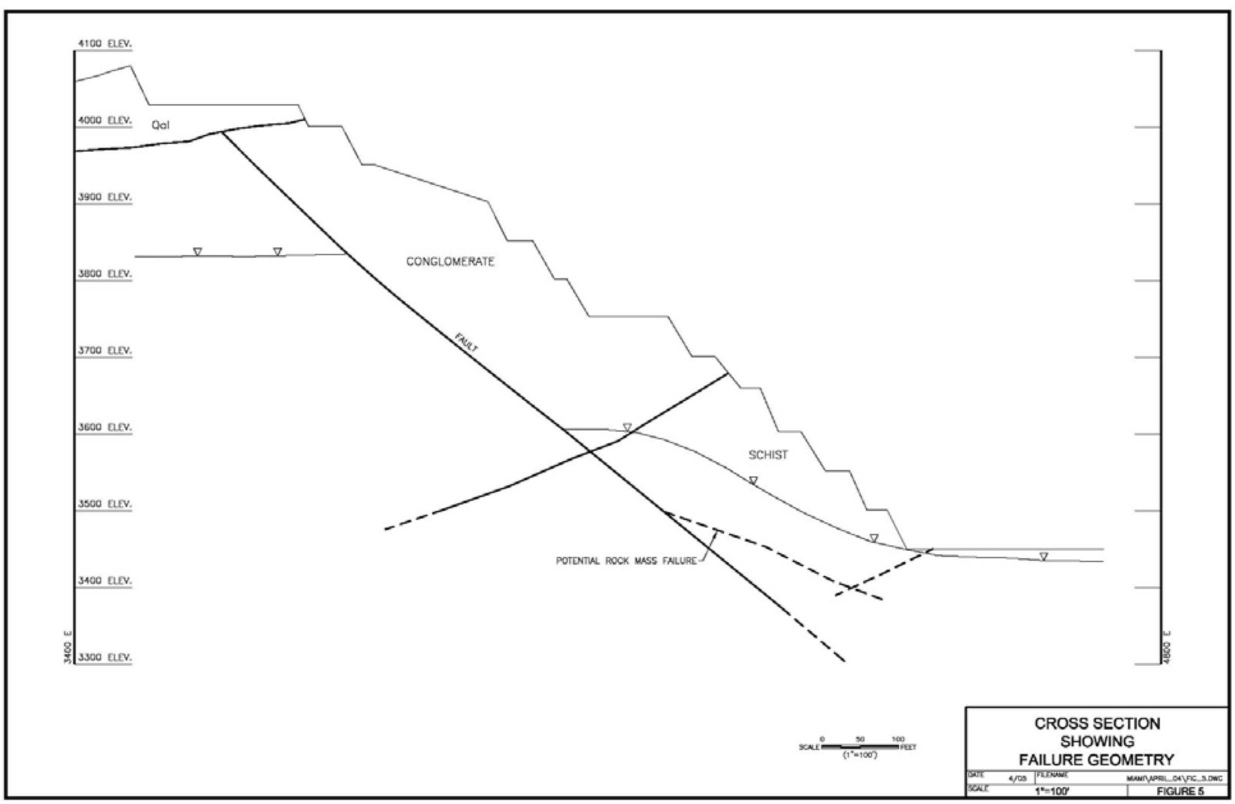

Figure 5. Cross section showing failure geometry

rock-mass failure. In order to evaluate this possibility the rock-mass strength at the toe of the slope must be estimated before stability analysis can be undertaken.

Detailed joint mapping of the rock mass at the toe of the slope indicates that a number of joint sets are present in the rock. As can be seen in a lower hemisphere projection of poles to the mapped structures (Fig. 6), there are a large number of kinematically viable failure paths along joint sets at the toe of the slope. Depending on the intensity of the jointing, some of these joint-set orientations will be weaker than others.

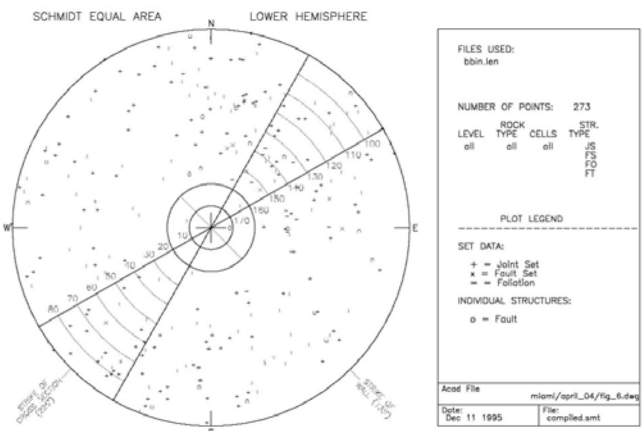

Figure 6.

In order to assess the critical failure path orientations for the rock mass between the toe of the pit and the fault, an analysis of the fully interlocked shear strength was undertaken. For a 2D analysis along the critical cross section, jointing that was striking within 20 degrees of the strike of the pit slope was examined. This set of subparallel structures was divided further into 10-degree dip increments for statistical analysis. Within each 10-degree pie the mean length and spacing of the joint-set data was calculated.

Using equation three with the fundamental data for the rock, the percentage of intact rock mobilization was estimated for each dip region, and plotted as both a rose diagram and a simple histogram for the cross section.

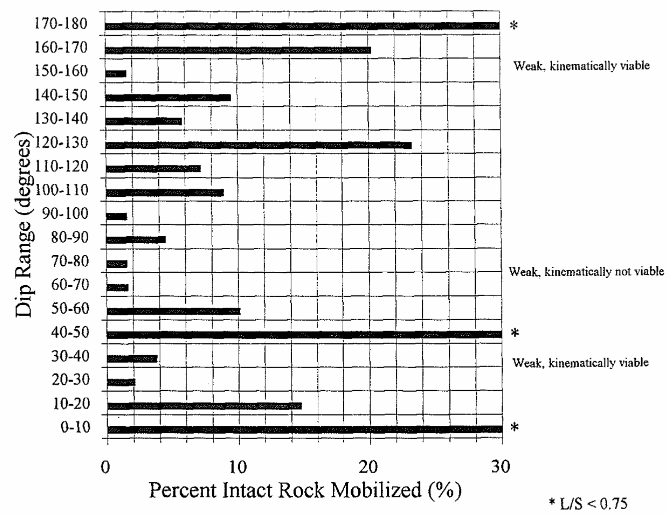

Figure 7. Evaluation of critical weakness plane orientations

The histogram (Fig. 7) shows the directions along which the rock mass is expected to be weakest. These directions represent good trial failure surfaces for the stability analysis, and the structures in those orientations were examined in more detail, leading to a change in the slope design that was successfully implemented.

Example #2 – Shear Strength of a Very Closely Jointed Arkose

The key parameters for this rock are:

- Compressive strength of intact rock:
  41.2 mpa
- Shear strength of intact rock:
  φ_i = 46 degrees, C_i = 8.3 mpa
- Residual friction angle of joints:
  29 degrees
- Roughness angle for joints:
  JRC = 3-6 degrees
- Peak joint shear strength:
  $\tau = 1.18\sigma^{0.80}$
- RQD = 15-20 %
- Three to five well developed joint sets:
  Jn = 15

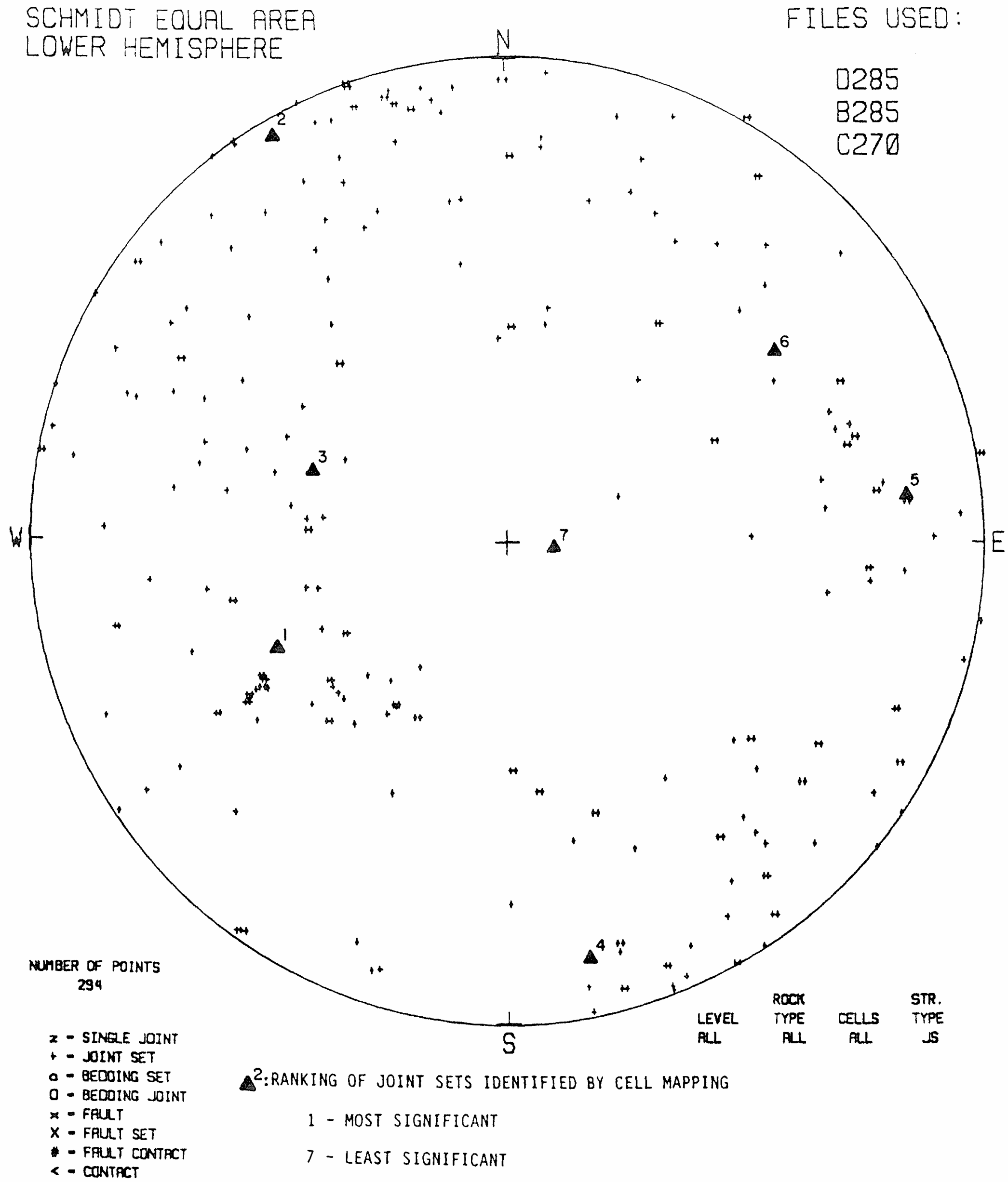

Figure 8. Lower hemisphere projection of joint data in arkose

A lower hemisphere Schmidt plot of structures mapped within the arkose in 1990 is shown on Figure 8. A very strongly developed joint set parallel to the wall (joint set #1 on Figure 8) was suspected as a controlling factor in rock slope movement. This set had an average dip direction of 065 degrees, an average dip of 32 degrees, and a joint roughness of 3-6 degrees. The average direction of movement for six prisms located within the active slide area was an azimuth of 062 degrees with a plunge of 27 degrees. Based on detailed joint mapping of bench outcrops in the area, the mean length of the joint set was estimated to be 5.60 meters, with a mean joint spacing of 0.51 meters.

Given a mean length to spacing ratio of 11 for the joint set, the approximate percentage of intact rock that can be mobilized along the joint orientation was estimated using:

$$\text{Max. Percent of Intact Rock} = .315 * 11^{-1.09} * 100\% \cong 2.3\% \quad (6)$$

For the case of a residual joint shear strength of 40KPa cohesion and 29-degree friction angle with three degrees of roughness, the best estimate of the fully interlocked rock-mass shear strength for 2.3 percent intact rock strength mobilization is:

$$C_f = 0.023 * 8.3\text{MPa} + (1.0 - 0.023) * .04\text{MPa} = 0.23\text{Mpa} \quad (7)$$

$$\mu_f = 0.023 * \tan(46) + (1.0 - 0.023) * \tan(29+3) = 0.63 \quad (8)$$

For a fully interlocked rock mass the RMS would be:

$$\text{RMS} = 0.23\text{MPa} * 2 * \tan(45+32.2/2) = 0.83\text{Mpa} \quad (9)$$

For a joint-set number (Jn) of 15 and a drill-hole RQD of 15-20 percent, Figure 3 would indicate that the transition stress to a fully interlocked condition would be approximately 2.5 times the RMS.

$$\text{Transition stress} = 2.5 * 0.83 \text{ MPa} = 2.07 \text{ Mpa} \quad (10)$$

To calculate the shear strength at normal stresses below the transition stress (partial interlocking) use the equation:

$$\tau = C_f * \{1-\exp(-b\sigma_n^{\ m})\} + \mu_f \sigma_n \quad (11)$$

where

$C_f =$ 0.23 Mpa
$\mu_f =$ 0.63
$m =$ 0.80

To evaluate "b", we must solve for the term $\{1-\exp(-b\sigma_n^{\ m})\}$ using a value of 0.99 when the normal stress is equal to the transition stress.

$$\{1-\exp(-b2.07^{0.80})\} = 0.99 \quad (12)$$

therefore

$$b \cong 2.57 \quad (13)$$

The final resulting shear strength model for the rock mass along joint set #1 is:

$$\tau\ (\text{MPa}) = 0.23 * \{1-\exp(-2.57\sigma_n^{\ 0.80})\} + 0.63\sigma_n \quad (\sigma_n = \text{MPa}) \quad (14)$$

Example # 3 – A Comparison with the Hoek & Brown Strength Model

For comparison purposes, rock-mass strengths for the arkose were also developed using the Hoek and Brown strength model. For the known rock-mass parameters, the likely range for RMR is between 31 and 36 for a ground water rating of 7 (moist). It is assumed that the "m" factor for intact rock is 14. For an RMR of 33, the adjusted "m" and "s" values for the rock mass can be estimated using:

$$m = 14*\exp\{ (33 - 100) / 28 \} = 1.28 \quad (15)$$
$$s = \exp\{ (33-100) / 9 \} = 5.84\text{ E-04} \quad (16)$$

for an "undisturbed" rock mass, and

$$m = 14*\exp\{ (33 - 100) / 14 \} = 1.17\text{ E-02} \quad (17)$$
$$s = \exp\{ (33-100) / 6 \} = 1.41\text{ E-05} \quad (18)$$

for a "disturbed" rock mass.

Using these adjusted values of "m" and "s" for the rock mass, along with a compressive strength of 41.2 MPa for the intact rock, two shear strength envelopes have been developed using the method of Hoek and Brown for the arkose.

For comparison purposes, a series of strength envelopes have been developed for the rock using different joint length to spacing ratios. Figure 9 shows the relationship between the strength envelopes developed using the method proposed here, and the Hoek and Brown strengths. The strengths developed by the model presented in this paper compare quite well to the strength envelopes developed using Hoek and Brown (1980).

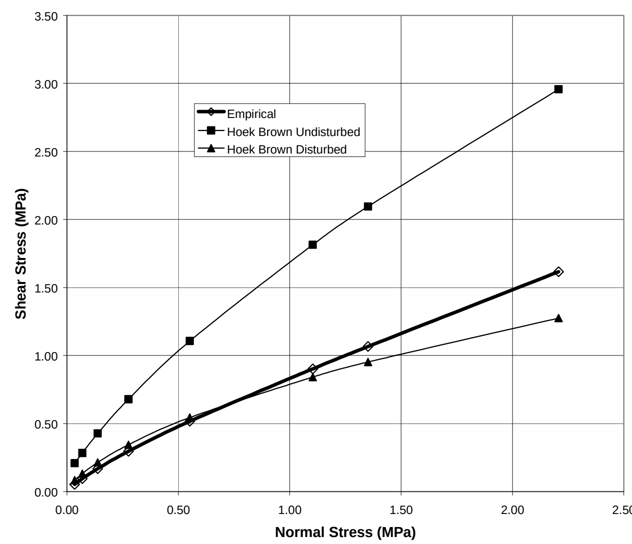

Figure 9.

The predicted shear strength envelope for the rock mass along the strong joint set #1 orientation falls between those predicted by the Hoek and Brown model using the published RMR correlations for disturbed and undisturbed rock masses.

6. CONCLUSIONS

A method of rock-mass strength evaluation has been developed incorporating intact rock strength, fracture shear strength, and rock structure data. The exponentially increasing cohesion strength model has been adapted for estimating the rock-mass shear strength envelope. This non-linear shear strength model has been found to work well for rock slope stability analysis and slope design at several mining projects.

Within this evaluation method, the author has placed a strong emphasis on characterization of the geologic structure, since the author believes that this is critical for estimating the shear strength of jointed rock masses. Assuming that the joint shear strength is a base case for a rock mass, the mobilized strength above that predicted for individual structures is assumed to be due to mobilization of intact rock strength and the interlocking of rock blocks along a discontinuous joint path. The author believes that this interlocking effect is the key to predicting the shear strength of jointed rock masses.

The strength due to interlocking has many components and is an area requiring more research. At this time, the interlocking between discontinuous joints can be empirically estimated for specific orientations on the basis of joint spacing and length data, albeit with some uncertainty. It is the intent of the author to improve this method of shear strength estimation through additional research targeted at the issue of shear strength mobilization within rock bridges between individual joints.

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TR
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