Pergamon

PII: S0148-9062(97)00314-8

Int. J. Rock Mech. Min. Sci. Vol. 35, No. 1, pp. 63–68, 1998

Printed in Great Britain
0148-9062/98 $19.00 + 0.00

Technical note

Reliability of Hoek-Brown Estimates of Rock Mass Properties and their Impact on Design

EVERT HOEK

INTRODUCTION

Hoek and Brown [1] presented a procedure for estimating the strength and deformation characteristics of isotropic jointed rock masses. When applying this procedure to rock engineering design problems, most users consider only the "average" or mean properties. In fact, all of these properties exhibit a distribution about the mean, even under the most ideal conditions, and these distributions can have a significant impact upon the design calculations.

This technical note examines the reliability of a slope stability calculation and a tunnel support design calculation. In each case, the strength and deformation characteristics of the rock mass are estimated by means of the Hoek-Brown procedure, assuming that the three input parameters are defined by normal distributions.

INPUT PARAMETERS

In the Hoek-Brown criterion, the Geological Strength Index (GSI) is the most important input parameter in terms of the relation between the strength and deformation properties determined in the laboratory and those assigned to the field scale rock mass. In earlier versions of the criterion, Bieniawski's RMR [2] was used for this scaling process.

Figure 1 can be used for estimating the value of GSI from field observations of blockiness and discontinuity surface conditions. Included in this figure is a cross-hatched circle representing the 90% confidence limits of a GSI value of 25 ± 5 (equivalent to a standard deviation of *ca* 2.5). This represents the range of values which an experienced geologist would assign to a rock mass described as *blocky/disturbed* or *disintegrated* and *poor*. Typically, rocks such as flysch, schist and some phyllites may fall within this range of rock mass descriptions.

In the author's experience, some geologists go to extraordinary lengths to try to determine an "exact" value of GSI (or RMR). Geology does not lend itself to such precision and it is simply not realistic to assign a single value. A range of values, such as that illustrated in Fig. 1 is more appropriate. In fact, in some complex geological environments, the range indicated by the cross-hatched circle may be too optimistic.

The two laboratory properties required for the application of the Hoek-Brown criterion are the uniaxial compressive strength of the intact rock (σ_{ci}) and the intact rock material constant m_i. Ideally these two parameters should be determined by triaxial tests on carefully prepared specimens as described by Hoek and Brown [1].

It is assumed that all three input parameters can be represented by normal distributions as illustrated in Fig. 2. The standard deviations assigned to these three distributions are based upon the author's experience of geotechnical programs for major civil and mining projects where adequate funds are available for high quality investigations. For preliminary field investigations or "low budget" projects, it is prudent to assume larger standard deviations for the input parameters.

OUTPUT PARAMETERS

The values of the friction angle ϕ, the cohesive strength c, the uniaxial compressive strength of the rock mass σ_{cm} and the deformation modulus E of the rock mass were calculated by the procedure described by Hoek and Brown [1]. The Excel add-on program @RISK (Palisade Corporation, Newfield, NY, U.S.A.) was used for a Monte Carlo analysis in which 1000 calculations were carried out for randomly selected values of the input parameters. The results of these calculations were analysed using the program BESTFIT (Palisade Corporation) and it was found that all four output parameters could be adequately described by the normal distributions illustrated in Fig. 2.

In several trials it was found that the output parameters ϕ, c and σ_{cm} were always well represented by normal distributions. On the other hand, for GSI values of >40, the deformation modulus E was better represented by a lognormal distribution.

P.O. Box 75516, North Vancouver, British Columbia, Canada. V7R 4X1

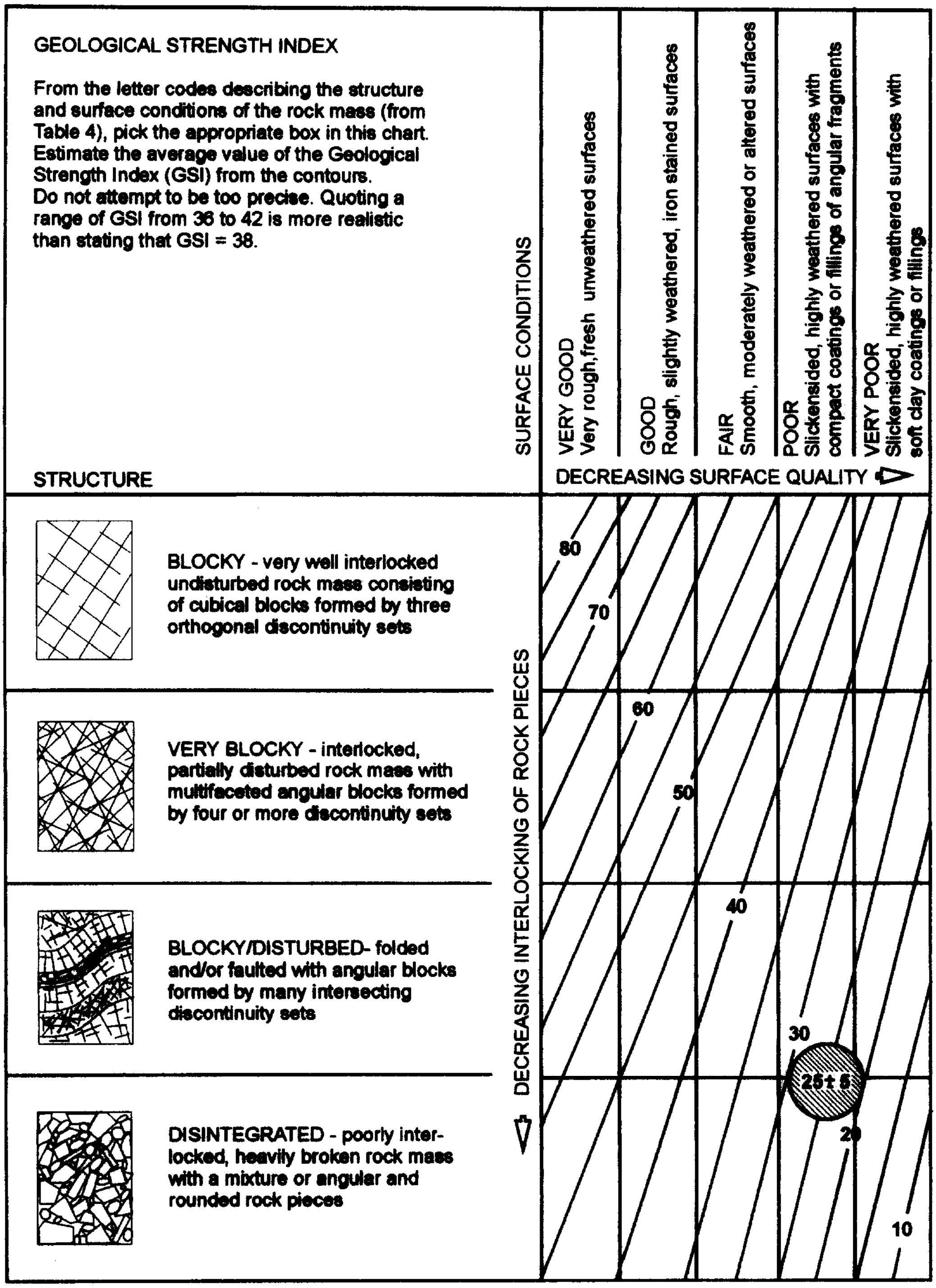

Fig. 1. Estimate of GSI based on geological descriptions.

SLOPE STABILITY CALCULATION

In order to assess the impact of the variation in output parameters, illustrated in Fig. 2, a calculation of the factor of safety for a homogeneous slope was carried out using Bishop's circular failure analysis in the program SLIDE (Rock Engineering Group, University of Toronto, Toronto, Ontario, Canada M4E 3B5). The geometry of the slope and the phreatic surface, the rock mass properties and the critical failure surface for the "average" properties are shown in Fig. 3.

The distribution of the factor of safety was determined by Rosenbleuth's Point Estimate method [3, 4] in which the two values are chosen at one standard deviation on either side of the mean for each variable. The factor of safety is calculated for every possible combination of point estimates, producing 2^m solutions, where m is the number of variables considered. The mean and standard deviation of the factor of safety are then calculated from these 2^m solutions.

This calculation of the mean and standard deviation is given in Table 1. Based upon the fact that the two

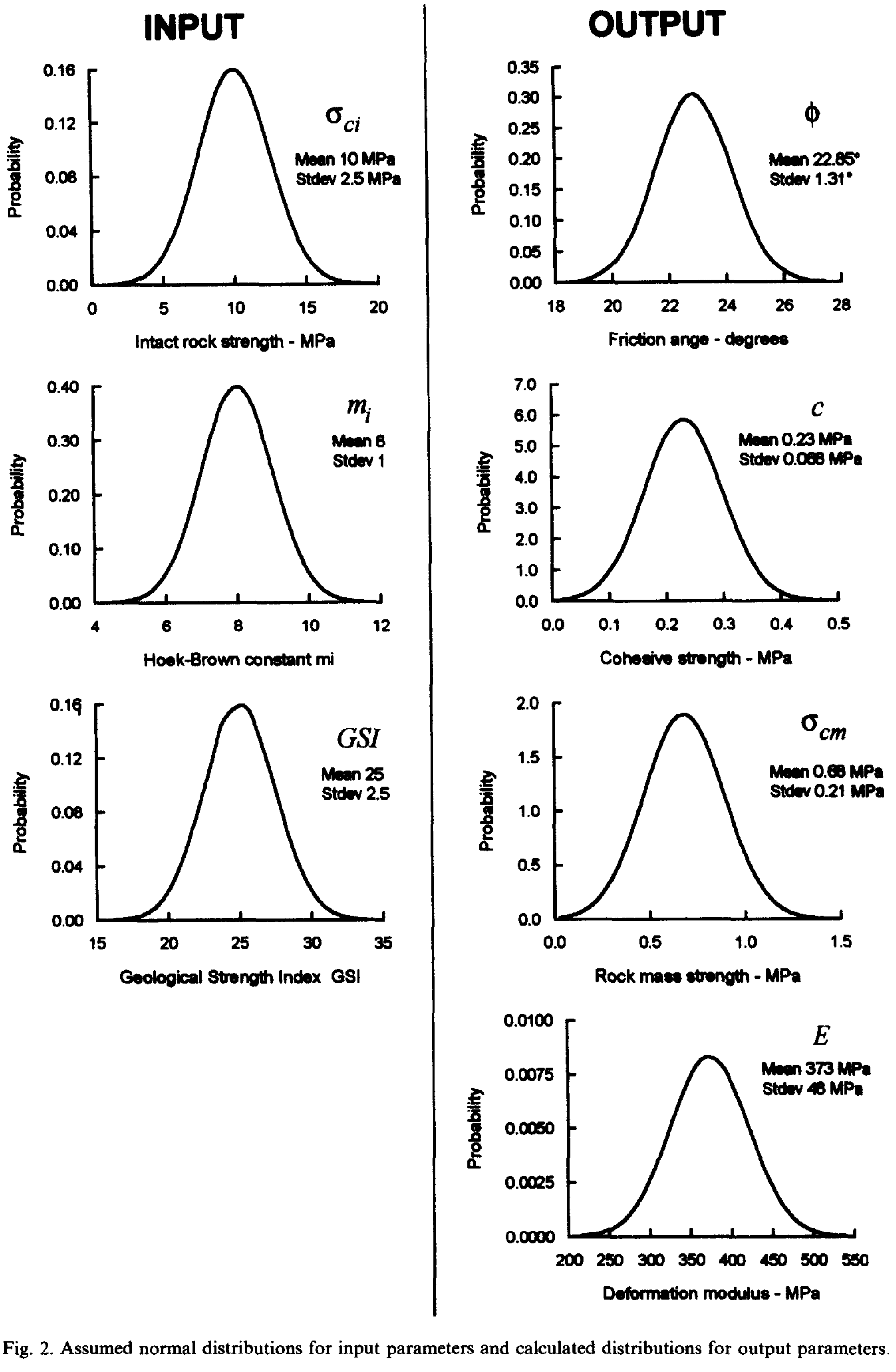

Fig. 2. Assumed normal distributions for input parameters and calculated distributions for output parameters.

Table 1. Calculations for Rosenbleuth's point estimate method using ±1 SD

Case	Friction angle (°)	Cohesion	Safety factor	$(\bar{SF}-SF_i)^2$
ϕ_-, c_-	21.19	0.162	1.215	0.00922
ϕ_+, c_+	24.16	0.298	1.407	0.00922
ϕ_-, c_+	21.19	0.298	1.217	0.00884
ϕ_+, c_-	24.16	0.162	1.406	0.00912
		Sums	5.245	0.0364

Mean safety factor $= \bar{SF} = \frac{1}{n}\sum_{i=1}^{n} SF_i = 1.31$.
Standard deviation $= S^2 = \frac{1}{n-1}\sum_{i=1}^{n}(\bar{SF} - SF_i)^2 = 0.11$.

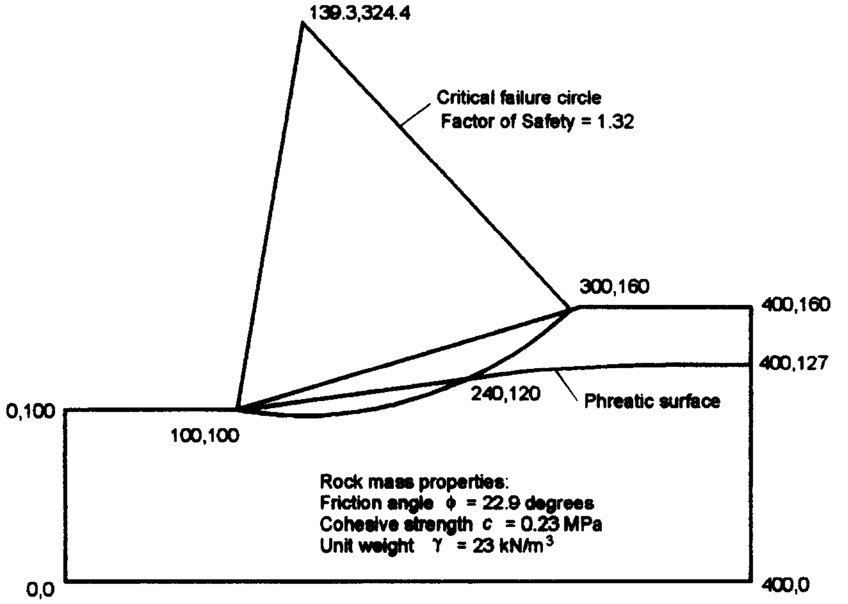

Fig. 3. Slope and phreatic surface geometry, rock mass properties and critical failure surface for a homogeneous slope.

variables included in this analysis are defined by normal distributions and considering the form of the equations used to calculate the factor of safety, it is reasonable to assume that the factor of safety will be adequately represented by a normal distribution. This distribution is illustrated in Fig. 4.

The mean factor of safety for this slope is 1.3 which is a value frequently used in the design of slopes for open pit mines. It is interesting that the probability of failure, given by the portion of the distribution curve for SF < 1, is very small. This suggests that, for a high quality geotechnical investigation such as that assumed in this study, a safety factor of 1.3 is adequate to ensure stability under the assumed conditions.

TUNNEL STABILITY CALCULATIONS

Consider a circular tunnel of radius r_o in a stress field in which the horizontal and vertical stresses are both p_o. If the stresses are high enough, a "plastic" zone of damaged rock of radius r_p surrounds the tunnel. A uniform support pressure p_i is provided around the perimeter of the tunnel. This situation is illustrated in Fig. 5.

Assuming that the rock mass fails with zero plastic volume change, the critical stress level p_{cr} at which failure initiates is given by [5]:

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1+k} \tag{1}$$

where

$$k = \frac{1+\sin\phi}{1-\sin\phi} \tag{2}$$

Where the support pressure p_i is less than the critical pressure p_{cr}, the radius r_p of the plastic zone and the inward deformation of the tunnel wall u_{ip} are given by:

$$\frac{r_p}{r_o} = \left[\frac{2(p_o(k-1)+\sigma_{cm})}{(1+k)((k-1)p_i+\sigma_{cm})}\right]^{1/(k-1)} \tag{3}$$

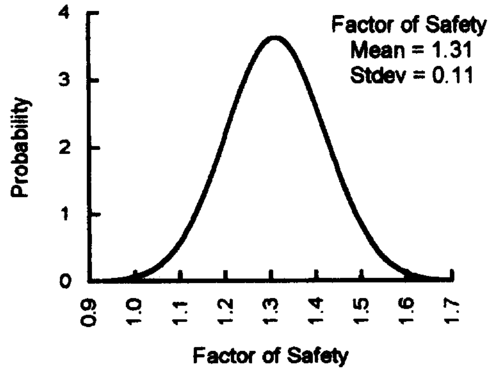

Fig. 4. Normal distribution for factor of safety of slope defined in Fig. 3.

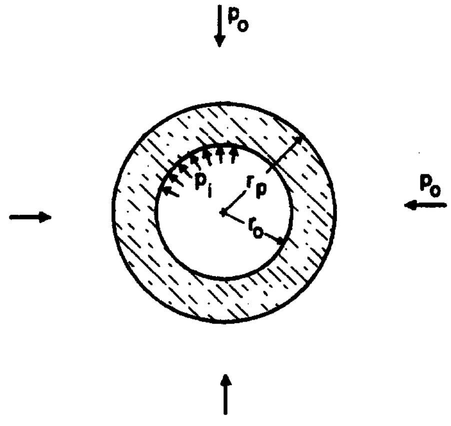

Fig. 5. Development of a plastic zone around a circular tunnel in a hydrostatic stress field.

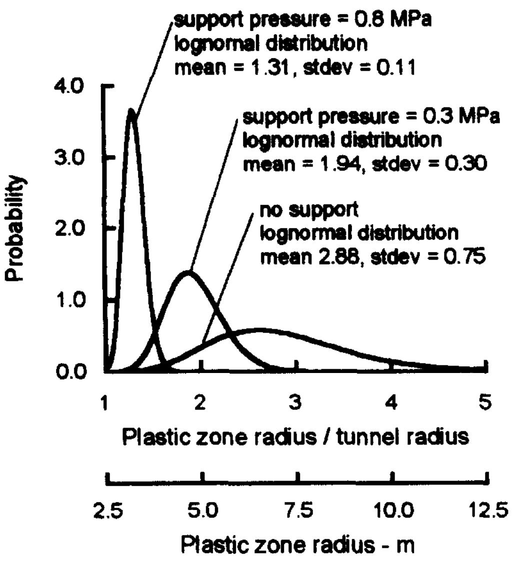

Fig. 6. Lognormal distributions representing the range of plastic zone radii for different support pressures.

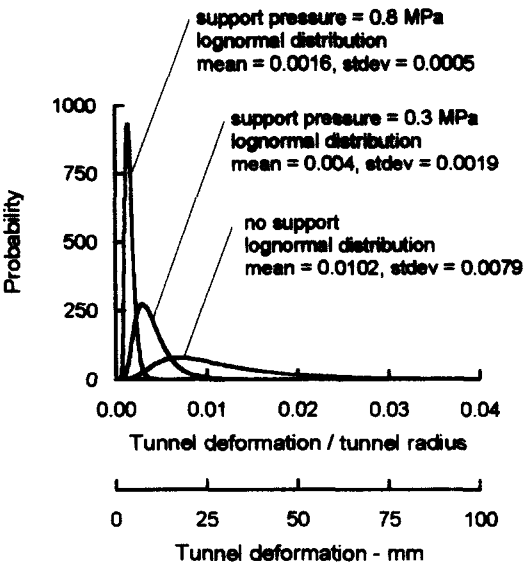

Fig. 7. Lognormal distributions representing the range of tunnel deformations for different support pressures.

$$\frac{u_{ip}}{r_o} = \frac{(1+\nu)}{E}\left[2(1-\nu)(p_o - p_{cr}\left(\frac{r_p}{r_o}\right)^2 - (1-2\nu)(p_o - p_i)\right] \quad (4)$$

In order to study the influence of the variation in the input parameters, a Monte Carlo analysis was performed using the program @RISK in an Excel spreadsheet which had been programmed to perform the analysis defined above. It was assumed that a 5 m diameter tunnel ($r_o = 2.5$ m) was subjected to uniform *in situ* stress of $p_o = 2.5$ MPa. The rock mass properties were defined by the normal distributions for ϕ, c, σ_{cm} and E defined in Fig. 2.

This analysis was carried out for a tunnel with no support. A second analysis was performed for a tunnel with a support pressure of $p_i = 0.3$ MPa which is approximately that which can be achieved with a closed ring of 50 mm thick shotcrete with a uniaxial compressive strength of 14 MPa (after 1 day of curing). This would represent the early support which would be achieved by the immediate application of shotcrete behind the advancing face. A third analysis was performed for a support pressure $p_i = 0.8$ MPa. This is approximately the support which can be achieved in this size of tunnel by a 75 mm thick shotcrete lining with a uniaxial compressive strength of 35 MPa (cured for 28 days). The results of these analyses are summarized graphically in Figs 6 and 7.

Figures 6 and 7 show that the size of the plastic zone and the tunnel deformation can be represented by lognormal distributions. As would be expected, the mean values for the size of the plastic zone and the magnitude of the sidewall displacements are reduced significantly by the installation of support.

What is surprising is the dramatic reduction in the standard deviations with increasing support pressure. This is because of the strong dependence of the size of the plastic zone upon the difference between the critical pressure p_{cr} and the support pressure p_i. A detailed discussion on this dependence is beyond the scope of this technical note and is the subject of ongoing research by the author.

From the results of the analysis described above it is evident that the installation of a relatively simple support system is very effective in controlling the behaviour of this tunnel. Without support there is an *ca* 50% probability of severe instability and possible collapse of the tunnel. A plastic zone diameter of 15 m and a tunnel closure of 50 mm in a 5 m diameter tunnel would certainly cause visible signs of distress. The fact that a relatively thin shotcrete lining can control the size of the plastic zone and the closure of the tunnel provides confirmation of the effectiveness of support.

A word of warning is required at this point. The example described above is for a 5 m diameter tunnel at a depth of *ca* 100 m below the surface. For larger tunnels at greater depths, the plastic zone and the displacements can be significantly larger. The demands on the support system may be such that it may be very difficult to support a large tunnel in poor ground at considerable depth below surface.

CONCLUSIONS

The uncertainty associated with estimating the properties of *in situ* rock masses has a significant impact or the design of slopes and excavations in rock. The examples which have been explored in this technical note show that, even when using the "best" estimates currently available, the range of calculated factors of safety or tunnel behaviour are uncomfortably large. These ranges become alarmingly large when poor site investigation techniques and inadequate laboratory procedures are used.

Given the inherent difficulty of assigning reliable numerical values to rock mass characteristics, it is unlikely that "accurate" methods for estimating rock mass properties will be developed in the foreseeable future. Consequently, the user of the Hoek–Brown procedure or of any other equivalent procedure for estimating rock mass properties should not assume that the calculations produce unique reliable numbers. The simple techniques described in this note can be used to explore the possible range of values and the impact of these variations on engineering design.

Acknowledgements—Professor E. T. Brown reviewed a draft of this technical note and his comments are gratefully acknowledged.

Accepted for publication 20 August 1997

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