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Technical note

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

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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 (σi) and the intact rock material constant m. 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 σcm 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 σcm 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.
**GEOLOGICAL STRENGTH INDEX**

From the letter codes describing the structure and surface conditions of the rock mass (from Table 4), pick the appropriate box in this chart. Estimate the average value of the Geological Strength Index (GSI) from the contours. Do not attempt to be too precise. Quoting a range of GSI from 36 to 42 is more realistic than stating that GSI = 38.

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>SURFACE CONDITIONS</th>
<th>DECREASING SURFACE QUALITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>BLOCKY - very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets</td>
<td>VERY GOOD</td>
<td>DECREASING INTERLOCKING OF ROCK PIECES</td>
</tr>
<tr>
<td>VERY BLOCKY - interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets</td>
<td>GOOD</td>
<td>80</td>
</tr>
<tr>
<td>BLOCKY/DISTURBED - folded and/or faulted with angular blocks formed by many intersecting discontinuity sets</td>
<td>FAIR</td>
<td>70</td>
</tr>
<tr>
<td>DISINTEGRATED - poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces</td>
<td>POOR</td>
<td>60</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Case</th>
<th>Friction angle (°)</th>
<th>Cohesion</th>
<th>Safety factor</th>
<th>(SF - SF̄)²</th>
</tr>
</thead>
<tbody>
<tr>
<td>φ+, c-</td>
<td>21.19</td>
<td>0.162</td>
<td>1.215</td>
<td>0.09922</td>
</tr>
<tr>
<td>φ+, c+</td>
<td>24.16</td>
<td>0.298</td>
<td>1.407</td>
<td>0.09922</td>
</tr>
<tr>
<td>φ-, c+</td>
<td>21.19</td>
<td>0.298</td>
<td>1.217</td>
<td>0.00884</td>
</tr>
<tr>
<td>φ-, c-</td>
<td>24.16</td>
<td>0.162</td>
<td>1.406</td>
<td>0.00912</td>
</tr>
<tr>
<td>Sums</td>
<td>5.245</td>
<td>0.0364</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mean safety factor = SF = \frac{1}{N} \sum_{i=1}^{N} SF_i = 1.31.

Standard deviation = S² = \frac{1}{N-1} \sum_{i=1}^{N} (SF - SF̄)² = 0.11.
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]:

\[
p_{cr} = \frac{2\mu_o - \sigma_{cm}}{1 + k}
\]

(1)

where

\[
k = \frac{1 + \sin \phi}{1 - \sin \phi}
\]

(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 \( u_p \) are given by:

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

(3)
This is because of the strong dependence of the size of the plastic zone upon the difference between the critical pressure $p_c$ 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 on 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.

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REFERENCES