

**INTERNAL REPORT
DIVISION OF ROCK MECHANICS**

**ESTIMATING ROCK MASS STRENGTH USING
THE HOEK-BROWN FAILURE CRITERION
AND ROCK MASS CLASSIFICATION**

**—A REVIEW AND APPLICATION TO THE
AZNALCOLLAR OPEN PIT**

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PREFACE

This report constitutes part of a joint research project between Boliden Mineral AB and Luleå University of Technology, concerning large scale slope stability in open pit mining. The project is aimed at developing a design methodology for large scale rock slopes, with special application to the Aitik open pit in northern Sweden. Financial support for the project is provided by Boliden Mineral AB.

In this report, the history of the Hoek-Brown failure criterion is reviewed, and its validity discussed, with particular application to the design of large scale rock slopes. The report specifically deals with strength estimation for the Aznalcollar open pit mine in Spain. These strength estimates will serve as input to numerical modeling and limit equilibrium analysis of the footwall stability at Aznalcollar, which will be described in a subsequent report.

The report has greatly benefited from the comments by Mr. Norbert Krauland, senior consultant at Boliden Mineral AB, and Dr. Erling Nordlund, Associate Professor at the Division of Rock Mechanics, Luleå University of Technology, for which they both are gratefully acknowledged.

Luleå, August 1997

Jonny Sjöberg

ABSTRACT

In this report, the history of the Hoek-Brown failure criterion is reviewed. The validity of the criterion and the methodology of parameter estimation are discussed, with particular application to the design of large scale rock slopes.

From the review it was clear that the continuous update of the original Hoek-Brown failure criterions has not been complemented by equal efforts to verify the same. Furthermore, the data supporting some of these revisions have not been published, making it difficult to judge the validity of these. One particular issue is the use of the categories *undisturbed* and *disturbed rock mass* when determining parameters in the failure criterion, for which clear guidelines are lacking. Also, the modified Hoek-Brown criterion gives physically questionable values for the rock mass strength and should be used with caution in low-stress environments. It is recommended that the original Hoek-Brown criterion instead be used routinely.

Many analysis methods require the use of the Mohr-Coulomb failure criterion with strength parameters cohesion and friction angle. In determining equivalent cohesion and friction angle for the curved Hoek-Brown failure envelope, it is important to use stress ranges relevant to the particular slope. Ideally, the stress state should be determined through numerical stress analysis assuming elastic conditions, which gives an upper bound to the actual state of stress in a slope.

The Hoek-Brown strength criterion was applied to estimate the strength of the rock mass at the Aznalcollar open pit. It was shown that by assuming *disturbed rock mass*, a good agreement was found between estimated strength values and back-calculated strengths from observed slope failures in the footwall. These values are probably conservative and representative of the residual strength of the rock mass. They can as such be used in limit equilibrium methods for assessing the factor of safety for fully developed failure. Similarly, the category *undisturbed rock mass* can be considered equivalent to the peak strength of the rock mass, which could be applied in numerical models. This hypothesis must, however, be verified by additional data from slope failures before more precise guidelines can be formulated.

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1 INTRODUCTION

The strength of jointed rock masses is notoriously difficult to assess. Laboratory tests on core samples are not representative of a rock mass of significantly larger volume. On the other hand, *in situ* strength testing of the rock mass is seldom practically or economically feasible. Back-analysis of observed failures can provide representative values for large scale rock mass strength, but obviously, this is only possible for cases in which rock mass failure has occurred. The more general problem of forward strength prediction for large scale rock masses remains as one of the great challenges in rock mechanics.

The currently more or less accepted approach to this problem is to use the Hoek-Brown failure criterion and estimate the required parameters with the help of rock mass classification. This methodology is relatively well established but not without its difficulties. This became apparent within a currently ongoing project concerned with slope stability analysis of the Aznalcollar open pit mine in Spain. Part of this project work involved determining representative strength parameters for subsequent use in stability analysis (limit equilibrium analysis and numerical modeling). As previous work in this area using the Hoek-Brown criterion gave inconclusive and sometimes contradictory results, it was deemed necessary to critically review and discuss the approach to strength estimation.

The objective of this report is thus to:

1. Review the history of the Hoek-Brown failure criterion, along with the suggested guidelines for parameter estimation, and discuss the ambiguities and uncertainties associated with the different versions of the Hoek-Brown criterion.
2. Apply the Hoek-Brown criterion to estimate the strength of the Aznalcollar hangingwall and footwall rocks, and compare these values with estimates provided by others (e.g., Golder Associates, UK), as well as with back-calculated strength values.
3. Discuss the implications of using the Hoek-Brown failure criterion for strength assessment and provide some thoughts on how to make better use of strength estimates for rock slope stability analysis.

In Chapters 2 and 3, the Hoek-Brown failure criteria are described, along with methods to estimate equivalent strength properties for the rock mass. The Aznalcollar pit is dealt with in Chapter 4, and in Chapter 5, some conclusions and recommendations for future applications, especially for stability analysis of rock slopes, are given.

2 FAILURE CRITERIA AND PARAMETER ESTIMATION

2.1 The Original Hoek-Brown Failure Criterion

The Hoek-Brown failure criterion is an empirical criterion developed through curve-fitting of triaxial test data. The conceptual starting point for the criterion was the Griffith theory for brittle fracture but the process of deriving the criterion was one of pure trial and error. The original Hoek-Brown criterion was proposed in 1980 (Hoek and Brown, 1980) and is defined as

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_c + s\sigma_c^2} \quad (2.1)$$

where

- m = constant depending on the characteristics of the rock mass,
- s = constant depending on the characteristics of the rock mass,
- σ_c = uniaxial compressive strength of the intact rock material,
- σ_1 = major principal stress at failure, and
- σ_3 = minor principal stress at failure.

The uniaxial compressive strength for the rock mass, $\sigma_{c, \text{rockmass}}$, can be expressed by setting $\sigma_3 = 0$ in Equation 2.1 thus obtaining

$$\sigma_{c, \text{rockmass}} = \sigma_c \sqrt{s}. \quad (2.2)$$

The uniaxial tensile strength of the rockmass, $\sigma_{t, \text{rockmass}}$, can be found by setting $\sigma_1 = 0$ in Equation 2.1, thus yielding

$$\sigma_{t, \text{rockmass}} = \frac{\sigma_c}{2} \left(m - \sqrt{m^2 + 4s} \right). \quad (2.3)$$

There is no fundamental relation between the constants in the failure criterion and the physical characteristics of the rock mass. The justification for choosing this particular formulation was the good agreement with observed rock fracture behavior (Hoek, 1983). Also, since the authors were most familiar with the design of underground excavations, it was chosen to formulate the criterion in terms of principal stresses. This is, however, a problem for certain applications, e.g., slope stability analysis in which the shear strength of a failure

surface as a function of the normal stress is required. Procedures to deal with this obstacle will be discussed in Chapter 3 of this report.

For intact rock, $s = 1$ and $m = m_i$. Values for m_i can be calculated from laboratory triaxial testing of core samples at different confining stress, or extracted from reported test results. Hoek and Brown (1980) provided a relative thorough compilation of such data. In essence, they found trends suggesting that rock types could be grouped into five classes, with $m_i = 7, 10, 15, 17$ and 25 , respectively, see Appendix 1, Table A1.1. Hoek and Brown did, however, note that the scatter within each group was considerable.

For jointed rock masses, $0 \leq s < 1$ and $m < m_i$. The values for each of these parameters can be difficult to assess since this also requires triaxial testing. At that time there were very few such data sets available for rock masses (this is probably also true today). Hoek and Brown (1980) reported results from, in their opinion, one of the most complete sets of triaxial test data conducted on *Panguna andesite* from the Bougainville open pit in Papua New Guinea. Since these test data form the basis for estimating values for m and s , it is worthwhile to describe them in more detail. The tested rock samples varied in composition and different testing methods were employed, according to the following list, reproduced from Hoek and Brown (1980):

- *Intact Panguna andesite.* Tests on 25 mm and 50 mm core samples.
- *Undisturbed core samples.* 152 mm diameter samples of jointed rock (joint spacing typically 25 mm) recovered using triple tube coring technique, and tested triaxially.
- *Recompacted graded samples.* Samples taken from bench faces, scaled down and compacted back to near *in situ* density, and tested in a 152 mm diameter triaxial cell.
- *Fresh to slightly weathered Panguna andesite.* Samples taken from the open pit mine and compacted to near *in situ* density and then tested in a 571 mm diameter triaxial cell.
- *Moderately weathered Panguna andesite.* Samples taken from the mine and compacted to near *in situ* density and then tested in a 571 mm diameter triaxial cell.
- *Highly weathered Panguna andesite.* Compacted samples tested in a 152 mm diameter triaxial cell.

It is worth noting that most of these tests were done on recompacted samples from the mine, which may not be totally representative of the *in situ* rock mass conditions. Nevertheless, these tests did provide a means of assessing the influence of increasing size and decreasing quality (more weathering) on the triaxial strength. The triaxial tests showed a decrease in the values of m and s with increasing degree of jointing and weathering. The value of m ranged from 0.278 to 0.012, with $m_i = 18.9$, and s was in the interval of 0.0002 to 0.

It was realized that extensive testing of larger size samples is not feasible in most practical projects. Hoek and Brown therefore proposed the use of rock mass classification techniques to estimate the value of the two parameters m and s . Using the test data on the *Panguna andesite*, and assuming those samples were representative of the rock mass (i.e. scale independent), they fitted two lines relating m/m_i and s to classification ratings. Both the *CSIR*, or *RMR*, (Bieniawski, 1976) and the *NGI* (Barton, Lien and Lunde, 1974) classification schemes were utilized for this. As pointed out by Hoek and Brown, these relations suffer from the limited amount of test data. For example, the relation between s and *RMR* was based on only two data points. Hoek and Brown extrapolated their findings and constructed a table with *approximate* relations between the values of m and s and classification ratings, see also Hoek (1983) and Appendix 1, Table A1.1. Hoek and Bray (1981) also reproduced the same table in the book on slope design.

2.2 The Updated Hoek-Brown Failure Criterion

In 1988, Hoek and Brown presented an update to the original Hoek-Brown failure criterion. One small update was to formulate Equation 2.1 in terms of effective stress, assuming that the effective stress law ($\sigma' = \sigma - u$, where u is the water pressure) applied. In fact, Hoek and Brown (1980) also discussed this and concluded that the effective stress law was applicable. The 1988 update also focused on ways to determine the constants m and s and techniques for estimating the equivalent cohesion, c , and friction angle, ϕ , of the material. The latter will be discussed in more detail in Chapter 3.

Based on the attempts by Priest and Brown (1983), the following updated empirical relations to calculate the constants m and s were presented (Brown and Hoek, 1988, Hoek and Brown, 1988):

Undisturbed (or Interlocking) Rock Masses

$$m = m_i e^{\frac{RMR-100}{28}} \quad (2.4)$$

$$s = e^{\frac{RMR-100}{9}} \quad (2.5)$$

Disturbed Rock Masses

$$m = m_i e^{\frac{RMR-100}{14}} \quad (2.6)$$

$$s = e^{\frac{RMR-100}{6}} \quad (2.7)$$

where

m_i = the value of m for the intact rock, and
 RMR = Rock Mass Rating (Bieniawski, 1976).

The introduction of the categories *undisturbed* and *disturbed rock mass*, respectively, was prompted by experiences of Hoek and Brown from practical use of the criterion for a few years. The predicted strengths using the original relations (Table A1.1, Appendix 1) had proved to be too conservative (giving too low strengths) for most applications. From this it was concluded that the specimens of *Panguna andesite* in reality probably were disturbed and the particle interlocking destroyed (Hoek and Brown, 1988). The previously suggested relations between classification ratings and values for m and s (Hoek and Brown, 1980) were therefore viewed as representative of a *disturbed rock mass* (Equations 2.6 and 2.7). Such strength values were considered reasonable when used for:

- i. slope stability studies in which the rock mass is usually disturbed and loosened due to the excavation of the slope (in particularly the boundaries of the slope),
- ii. underground excavations in which the rock has been loosened by poor blasting practice, and
- iii. waste dumps and embankments (Brown and Hoek, 1988; Hoek and Brown, 1988).

For underground applications in which the confining stress would not permit the same degree of loosening as would occur in a slope, the category *undisturbed rock mass* was introduced. This would apply to all cases in which the interlocking between particles and blocks is still significant. Back-calculation of rock mass strengths from a number of cases gave the relations in Equations 2.4 and 2.5. A summary is also provided in Appendix 1, Table A1.2. Unfortunately, the data supporting these revisions and the development of Equations 2.4 and 2.5 have not been published.

The value of *RMR* is obtained through rock mass classification using the *CSIR* classification system (*RMR*-system) according to Bieniawski (1976). It should be noted that for the purpose of using *RMR* to estimate *m* and *s*, dry conditions should be assumed (a rating of 10 in Bieniawski's system). Also, no adjustments for joint orientation should be made. The effect of joint orientation and groundwater conditions should instead be accounted for in the stability analysis. Finally, since there are several versions of the *RMR*-system and parameters are weighted differently in subsequent versions of the classification system, the above equations require the use of the 1976 version of Bieniawski's system.

2.3 The Modified Hoek-Brown Failure Criterion

Hoek, Wood and Shah (1992) stated that when applied to jointed rock masses, the original Hoek-Brown failure criterion gave acceptable strength values only for cases where the minor principal stress has a significant compressive value. For low confining stress, the criterion in general predicted too high axial strengths and also a finite tensile strength. For a jointed rock mass, the true tensile strength is generally very low, if not zero. A modified criterion that satisfied the condition of zero tensile strength was presented in 1992 by Hoek, Wood and Shah (1992) as

$$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} \right)^a \quad (2.8)$$

where

σ'_1 = major principal effective stress at failure,

σ'_3 = minor principal effective stress at failure,

m_b = the value of the constant *m* for broken rock, and

a = constant for broken rock.

The constant m_b in Equation 2.8 is equivalent to the constant m in Equation 2.1. Hoek, Wood and Shah (1992) provided tables for estimating the value of the constants a , the ratio m_b/m_i and the constant m_i for intact rock, based on a simplified description of the rock mass. The rock mass was described in terms of rock structure and surface condition for discontinuities. Rock structure comprised four classes: blocky, very blocky, blocky/seamy and crushed, whereas surface conditions ranged from very good to very poor (four classes), see also Appendix 1, Table A1.3.

2.4 The Generalized Hoek-Brown Failure Criterion

In the book by Hoek, Kaiser and Bawden (1995) a general form of the Hoek-Brown failure criterion was given. With notations as defined earlier, this is written

$$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} + s \right)^a \quad (2.9)$$

For intact rock, i.e. $s = 1$ and $m_b = m_i$, Equation 2.9 can be written as

$$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_i \frac{\sigma'_3}{\sigma_c} + 1 \right)^{1/2} \quad (2.10)$$

For rock masses of good to reasonable quality with relatively tight interlocking, the constant a is equal to 0.5, thus reducing Equation 2.9 to the Equation 2.1 (the original criterion). For poor quality rock masses, the modified Hoek-Brown criterion is more applicable, which is obtained by setting $s = 0$ in Equation 2.9, thus reducing it to Equation 2.8.

The constant m_i can be determined from triaxial tests on intact rock or, if test results are not available, from the tabulated data provided by Hoek, Kaiser and Bawden (1995), see Appendix 1, Table A1.4. To estimate the value of parameters m_b , s and a , the following relations were suggested by Hoek, Kaiser and Bawden (1995).

For $GSI > 25$ (*Undisturbed rock masses*)

$$m_b = m_i e^{\frac{GSI-100}{28}}, \quad (2.11)$$

$$s = e^{\frac{RMR-100}{9}}, \quad (2.12)$$

$$a = 0.5. \quad (2.13)$$

For $GSI < 25$ (*Undisturbed rock masses*)

$$s = 0, \quad (2.14)$$

$$a = 0.65 - \frac{GSI}{200}, \quad (2.15)$$

where GSI is the *Geological Strength Index*.

GSI is similar to RMR but incorporates also newer versions of Bieniawski's original system (Bieniawski, 1976, 1989). Hence, the following relations were developed (Hoek, Kaiser and Bawden, 1995).

For $RMR_{76'} > 18$:

$$GSI = RMR_{76}. \quad (2.16)$$

For $RMR_{89'} > 23$:

$$GSI = RMR_{89'} - 5. \quad (2.17)$$

For both versions, dry conditions should be assumed, i.e. assigning a rating of 10 in RMR_{76}' , and a rating of 15 in RMR_{89}' , for the groundwater category in each classification system. Also, no adjustments for joint orientation should be made.

For rock masses with $RMR_{76}' < 18$ and $RMR_{89}' < 23$, the RMR -system cannot be used, since these are the minimum values that can be obtained in each of these versions, respectively. For these cases, i.e. very poor quality rock masses, the NGI -index (Barton, Lien and Lunde, 1974) should be used instead. In using this classification system to estimate GSI , the joint water reduction factor (J_w) and the stress reduction factor (SRF) should both be set to 1. The thereby obtained modified Tunneling Quality Index (Q') can be related to GSI as

$$GSI = 9 \ln Q' + 44. \quad (2.18)$$

Hoek, Kaiser and Bawden (1995) also presented a table from which values for the constants a , the ratio m_b/m_i and the constant m_i for intact rock, can be determined based on the previously mentioned simplified description of the rock mass in terms of rock structure and surface condition for discontinuities, see Appendix 1, Table A1.5.

2.5 Strength of Schistose Rock

For schistose or layered rocks such as slates and shales, Hoek and Brown (1980) proposed an extension of the original Hoek-Brown failure criterion. A set of empirical equations were used to modify the material constants m and s as follows

$$m = m_i \left(1 - Ae^{-\theta^4}\right), \quad (2.19)$$

$$s = 1 - Pe^{-\zeta^4}, \quad (2.20)$$

$$\theta = \frac{\beta - \xi_m}{A_2 + A_3\beta}, \quad (2.21)$$

$$\zeta = \frac{\beta - \xi_s}{P_2 + P_3\beta}, \quad (2.22)$$

where

m_i = the value of m for intact rock,

A = constant,

P = constant,

β = the inclination of the discontinuity surfaces (schistosity or layering) to the direction of the major principal stress, σ_1 (see also Figure 2.1).

ξ_m = the value of β at which m is minimum,

ξ_s = the value of β at which s is minimum,

A_2 = constant,

A_3 = constant,

P_2 = constant, and

P_3 = constant.

The values of m and s can be calculated by means of linear regression of test results for the rock in question, in the process determining the constants in Equations 2.19 to 2.22. Hoek and Brown (1980) gave some examples of this technique and showed good agreement with actual test data. A major drawback is the many constants whose values must be determined before applying the criterion. This obviously requires extensive laboratory test data.

Hoek (1983) used a slightly different approach. Assuming that the shear strength of the discontinuity surfaces in schistose rocks is defined by an instantaneous friction angle, ϕ_j , and an instantaneous cohesion, c_j , the axial strength, σ_1 , of a triaxial specimen containing inclined discontinuities at an angle of β is given by

$$\sigma_1 = \sigma_3 + \frac{2(c_j + \sigma_3 \tan \phi_j)}{(1 - \tan \phi_j \tan \beta) \sin 2\beta} \quad (2.23)$$

Equation 2.23 defines the joint strength, but must be used in combination with e.g., Equation 2.1 for the intact rock strength, to fully describe the strength of the schistose material for different inclinations of the schistosity. For low values of β , Equation 2.23 will yield strength values that are higher than those predicted by Equation 2.1. For high values of β , Equation 2.23 will give negative strengths. The physical meaning of this is that for both these cases, the strength is instead governed by the intact rock strength according to Equation 2.1, see also Figure 2.1.

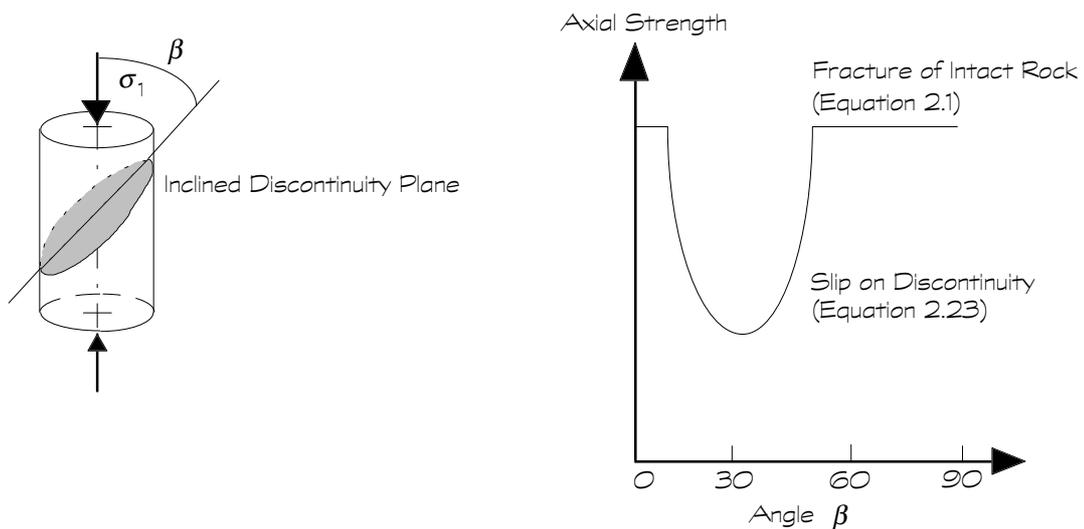


Figure 2.1 Strength of a rock specimen containing an inclined discontinuity (after Hoek, 1983).

To determine the strength envelope in Figure 2.1, the instantaneous friction angle and cohesion for the discontinuity surfaces must be known. The discontinuity strength can be described using the original Hoek-Brown failure criterion, but with m - and s -values for the joint (discontinuity), m_j and s_j , respectively. Provided that these values can be determined (e.g., from shear tests), the instantaneous friction angle and cohesion can be calculated (compare Section 3.2.2 and Equations 3.5 and 3.6) for the normal stress acting on the surface:

$$\sigma_n = \frac{1}{2}(\sigma_1 + \sigma_3) - \frac{1}{2}(\sigma_1 - \sigma_3)\cos 2\beta \quad (2.24)$$

Since σ_1 is the strength to be determined, an iterative procedure is necessary. This involves the following steps (Hoek, 1983):

- i. Calculate the axial strength, σ_1 , for the intact rock using Equation 2.1.
- ii. From this, calculate the normal stress, σ_n , using Equation 2.24.
- iii. Calculate the axial strength of the schistose rock using Equation 2.23.
- iv. Use this value for σ_1 to calculate a new value for the normal stress, σ_n , again using Equation 2.24.
- v. Repeat this procedure until the difference between successive values of σ_1 is less than 1 %, which normally requires three to four iterations.

This procedure also gives good agreement with actual test results on anisotropic rock. The main disadvantage is the iterative procedure, which slows down calculations. Also, m_j and s_j must be determined for the discontinuity surfaces, which requires more test results, preferably direct shear or triaxial test data. Obviously, if the cohesion and friction angle of the discontinuity surface is known, these values can be used directly in Equation 2.23, thus avoiding the need for an iterative procedure.

2.6. Use of the Hoek-Brown Failure Criterion

Hoek, Kaiser and Bawden (1995) summarized the rock mass conditions for which the Hoek-Brown failure criterion can be applied, as shown in Figure 2.2. Note that the criterion strictly only is applicable to intact rock or to heavily jointed rock masses that can be considered homogeneous and isotropic. For cases in which the rock mass behavior is controlled by a single discontinuity or joint set, a criterion that describes the shear strength of joints should be used instead (e.g., the Barton shear strength criterion or the Mohr-Coulomb criterion applied for discontinuities).

There are, however, no strict guidelines as to when the Hoek-Brown failure criterion can be applied. This must be based on judgement of potential anisotropy of the rock mass, block size in relation to size of the excavation, and mode of failure (structural control versus rock mass failure). Proposed qualitative guidelines for judging whether the rock is isotropic or not were given by Helgstedt (1997), but following these recommendations still involves a lot of subjectivity.

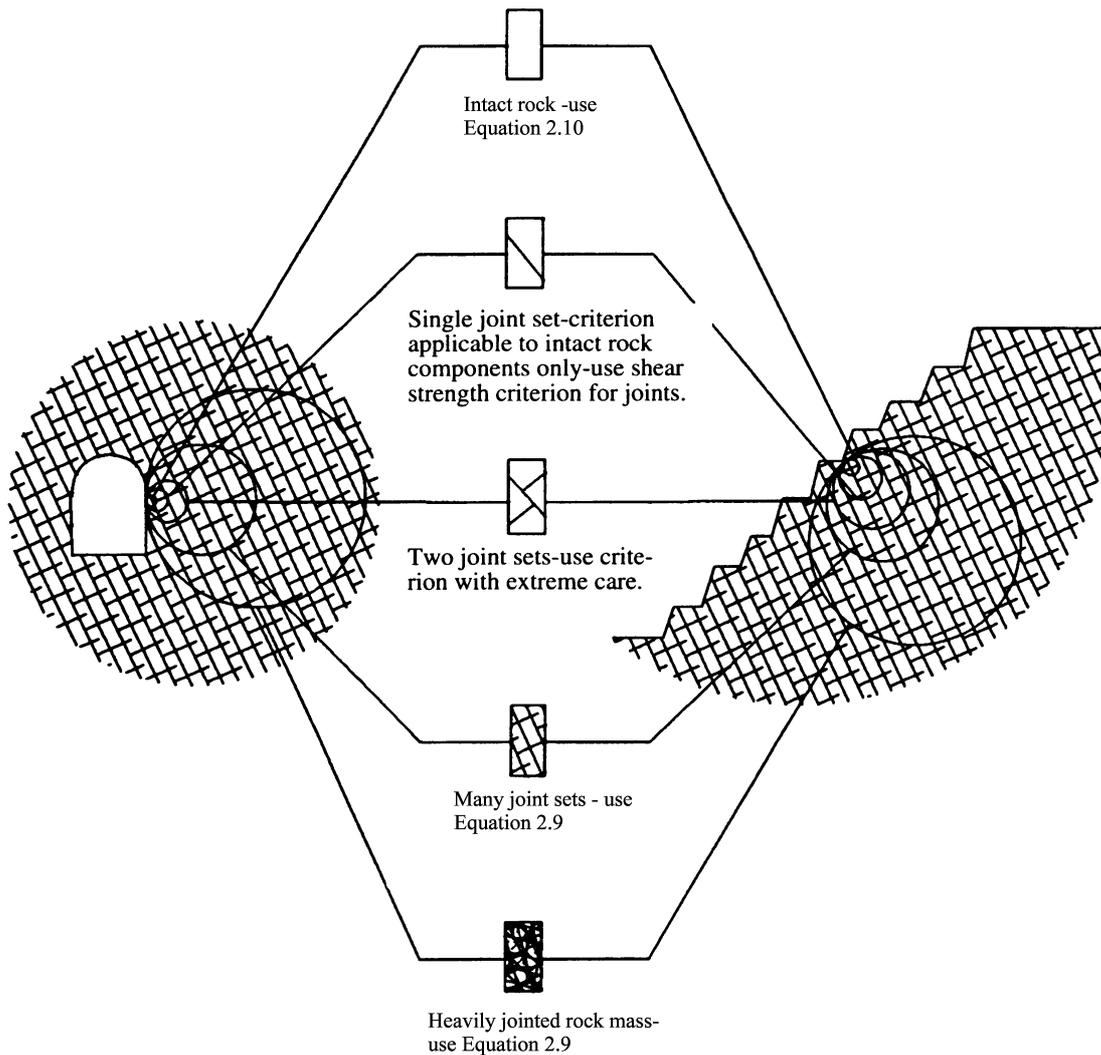


Figure 2.2 Rock mass conditions under which the Hoek-Brown failure criterion can be applied. From Hoek, Kaiser and Bawden (1995).

The case of schistose rocks was treated in Section 2.5. Both approaches for determining the strength of anisotropic rock have distinct disadvantages, and are difficult to apply in practice. Consequently, there are few examples of when such analyses have been conducted. Instead,

the original Hoek-Brown criterion has often been used with some adjustment of the material constants m and s , based on jointing and rock mass quality. Thus, an "equivalent" strength for the intact rock and the discontinuities combined is achieved.

2.7 Discussion

2.7.1 *Validity and Basis for Criterion*

The Hoek-Brown criterion is the most widely used failure criterion for estimating the strength of jointed rock masses, despite its lack of theoretical basis and the very limited amount of experimental data that went into the first development of the criterion. Only one rock type (*Panguna andesite*) was tested, although of various quality (compare Section 2.1). There was only one test series of "jointed rock mass"; namely the 152 mm undisturbed core samples. The triaxial tests on larger samples (571 mm size) were recompacted samples, not necessarily equivalent to actual *in situ* conditions. The lack of data became particularly obvious in the development of the original relations between classification ratings (*RMR*) and the parameters m and s . This was also clearly stated by Hoek and Brown (1980) as a major limitation.

Nevertheless, the rock mechanics community has gladly adopted these *approximate* equations suggested by Hoek and co-workers. This is somewhat surprising; something which also was noted by Hoek (1994b) who stated that he originally only intended the criterion to be used for initial and preliminary estimates of the rock mass strength. Hoek should also be complemented for his attempts at providing a tool for predicting the triaxial strength of rock masses. It was probably the apparent lack of such a criterion that led to the large acceptance of the criterion. Even as of today, there are no good alternatives available for practical design work. Also, the concept of a curved failure envelope similar to what is found for intact rock seemed fair and reasonable, although there were not much data around supporting this also for rock masses.

The widespread use of the Hoek-Brown failure criterion has not been complemented by equally increasing efforts to verify the same. There are very few reported cases in which the application of the Hoek-Brown failure criterion has been verified against actual observations of failure. It appears that many engineers have been busy applying the failure criterion, without taking the time to assess its validity. Some verification of the criterion was provided in a recent study by Helgstedt (1997) that compared predicted strengths with back-calculated values from a dam foundation and a large scale natural slope, as well as from tests on rockfill.

Helgstedt (1997) concluded that the Hoek-Brown criterion consistently predicted too high shear strengths for these cases. All these cases were rock masses of poor to medium quality with GSI in the range of 22 to 55.

There is, however, little merit in trying to develop a completely new empirical criterion. Rather, more efforts should be focused on trying to tie predicted Hoek-Brown strengths with field estimates and back-analyzed strengths from failed construction elements. Only by **publishing** such data is it possible to improve on the current lack of verification (as pointed out by Hoek at several occasions). In the following, some other problems associated with using the Hoek-Brown failure criterion in practice are discussed.

2.7.2 *The Original versus the Modified Criterion*

Consider the differences between the original and the modified Hoek-Brown criterion (Equations 2.1 and 2.8, respectively). These two equations are plotted in Figures 2.3 and 2.4 for a set of parameters representative of a poor quality rock mass, shown below. This is also the type of rock mass for which the modified criterion is best suited ($GSI < 25$). The values for parameters m_b , s and a were determined using Equations 2.11 to 2.15, as:

$$\sigma_c = 25 \text{ MPa}$$

$$m_i = 10$$

$$GSI = 20$$

$$m_b = 0.5743$$

$$\text{Original criterion (Equation 2.1): } s = 0.00138 \quad a = 0.5$$

$$\text{Modified criterion (Equation 2.6): } s = 0 \quad a = 0.55$$

In general, the two failure envelopes do not differ very much when compared to each other (Figure 2.3). However, if we look at little closer at these two, in particular for low compressive and tensile minor principal stresses, the differences are clearer, see Figure 2.4. Firstly, the modified criterion does not predict any tensile strength for the rock mass. This is typical of a granular material, or a rock mass without any interlocking. For rock masses with multiple joint sets providing some interlocking of blocks, it is conceivable that at least very small tensile stresses can be transferred through the rock mass. It is, however, relatively common to assume zero tensile strength for the rock mass when conducting design analysis, since this will error on the safe side. The lack of a tensile strength in the modified criterion can thus be considered reasonable and acceptable from a practical perspective.

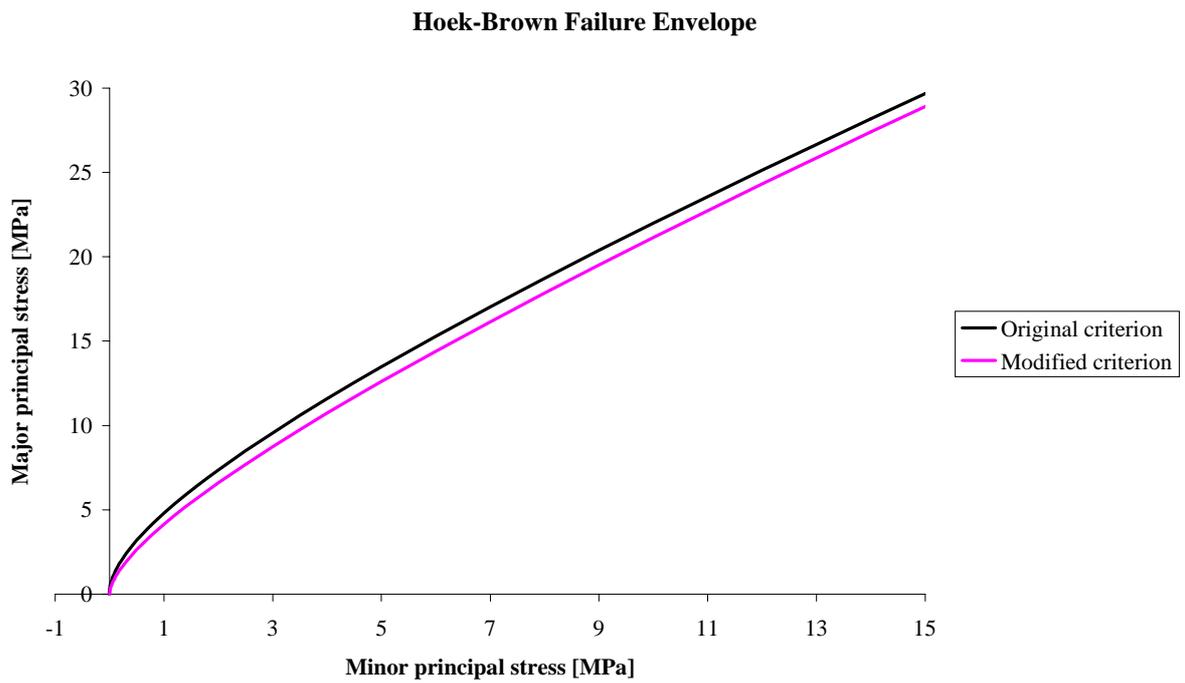


Figure 2.3 Hoek-Brown failure envelopes for a poor quality rock mass.

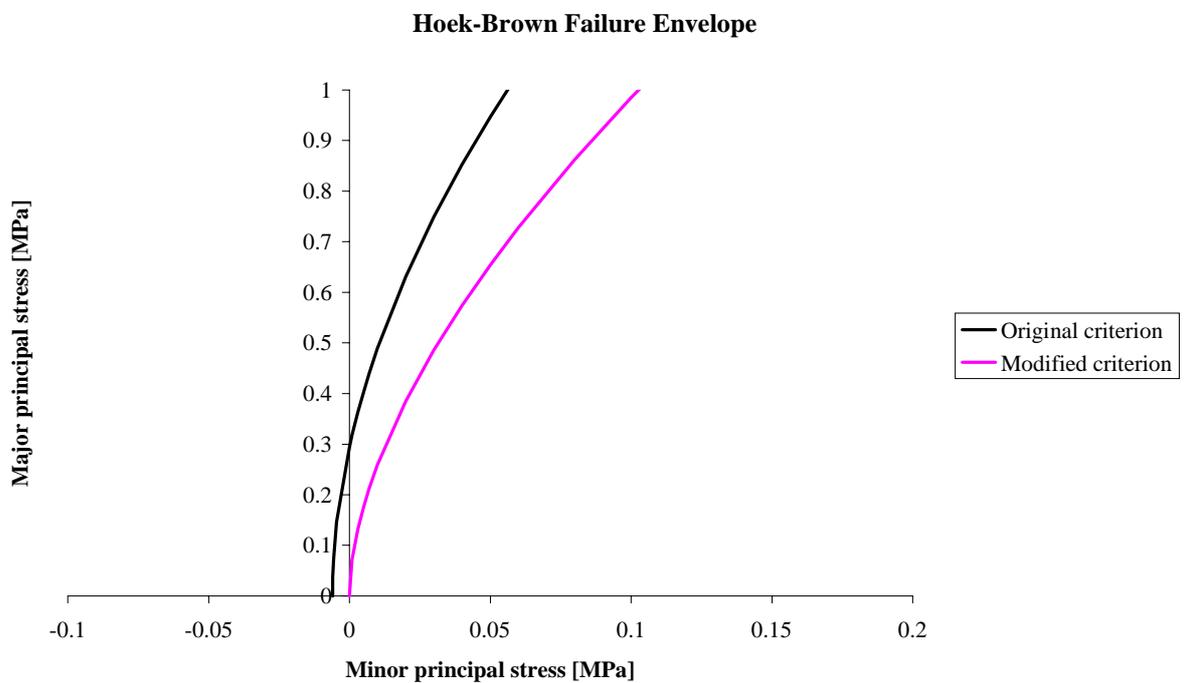


Figure 2.4 Hoek-Brown failure envelopes for a poor quality rock mass, and for low minor principal stress.

Secondly and perhaps more importantly, the modified criterion also predicts that the uniaxial compressive strength of the rock mass is zero (no intercept with the vertical axis in Figure 2.4). This is odd and in reality means that unless there is some confinement acting on the rock, the rock mass has no strength whatsoever.

In this particular case, the predicted uniaxial tensile and compressive strengths are very low as a consequence of the assumed very poor quality rock mass. For practical purposes, the relatively small difference between the two curves in Figures 2.3 and 2.4 might not be very significant. For higher values of GSI and σ_c the discrepancies between the modified and the original criterion are much larger. This is probably one reason to why the modified criterion should only be used when $GSI < 25$. However, if the differences between the original and modified criterion is small for $GSI < 25$, as indicated in this study, it can be questioned what the reason is for using the modified criterion at all?

In my opinion, the lack of unconfined strength for the rock mass in the modified Hoek-Brown criterion is not physically correct, even for very weak rock masses. The modified criterion should therefore be used with caution in any situation where very low or no confining stresses are anticipated. Since the differences in general are quite small, one could probably use the original criterion routinely without any great loss of accuracy.

2.7.3 Parameter Estimation

The many revisions and additions to the original Hoek-Brown failure criterion can lead to some confusion as to what to use for practical applications. As the authors have gained more experience and collected additional data over the years, the latest version and associated methods for parameter estimation should normally be considered the most reliable. There are, however, several ambiguities in the latest published data (Hoek, Kaiser and Bawden, 1995), compared to earlier versions.

Values for m_i

If test results for determining the constant m_i for intact rock are not available, it is possible to estimate the value of m_i from tabulated values. The most updated version of suggested m_i -values is given in Appendix 1, Table A1.4 (Hoek, Kaiser and Bawden, 1995). As Helgstedt (1997) pointed out, these values only represent the average m_i -values for the different rock types. In reality the scatter of the m_i -values can be very large, as shown in Table 2.1. Tabulated values should therefore be used with great care.

Table 2.1 Mean and standard deviation of m_i for a few selected rock types, based on data from Doruk (1991). From Helgstedt (1997).

Rock type	Number of test sets	Range of m_i -values	Average m_i -value	Standard deviation of m_i -value	Suggested m_i -value (Hoek, Kaiser and Bawden, 1995)
Dolomite	8	5.2 - 18.2	11.4	4.3	8-10
Granite	18	7.9 - 42.6	25.3	9.5	33
Limestone	26	3.9 - 51.7	11.2	9.4	8-10
Marble	14	4.7 - 16.0	8.0	3.2	9
Mudstone	7	8.6 - 46.6	19.2	14.5	9
Quartzite	7	5.6 - 28.4	18.2	7.3	24
Sandstone	57	4.7 - 35.5	16.0	8.6	19

Undisturbed versus Disturbed Rock Mass

In the latest version of the criterion published in the book by Hoek, Kaiser and Bawden (1995), only the category *undisturbed rock masses* is considered. This could be due to the fact that this book deals with underground excavations, for which the degree of confinement is considerable and *undisturbed rock mass* could be assumed (compare Section 2.2).

However, in an unpublished note that preceded the final publication, Hoek (1994a) stated that:

"The effects of blast damage, stress relief in excavated slopes and other processes which may disturb the interlocking of the rock pieces making up the rock mass should be taken into account by using a lower RMR value."

This means that the category *disturbed rock* does not exist at all, but should be accounted for by a lower *RMR*. Whether this is also Hoek's intention in the finally published book is impossible to say. The lack of sufficient description is bothersome and a source of confusion for the potential user.

Several consultants working in the field of slope design (Stacey, 1996) tend to agree that the category *disturbed rock* is, although slightly over-conservative, most applicable for rock slope design. On the other hand, Brown and Hoek (1988) stated that they used values for *undisturbed rock mass* for preliminary design calculations of **both** slopes and underground

excavations. Unfortunately, virtually nothing has been published to support either approach, making it difficult to judge the validity and applicability of Equations 2.4 to 2.7.

It can be argued that large scale slope failures have failure surfaces that are deep and therefore subjected to relatively high confinement. Using numerical modeling it can be shown that a relatively large region close to the slope face will be destressed compared to the virgin stress state (Sjöberg, 1996). However, the stress conditions within a slope will vary substantially from very low stress regimes at the crest with both the minor and major principal stresses being low, to relatively high major and minor principal stresses at the slope toe. The degree of confinement therefore depends on the location of the failure surface and what portion of the failure surface is considered, see also Figure 2.5.

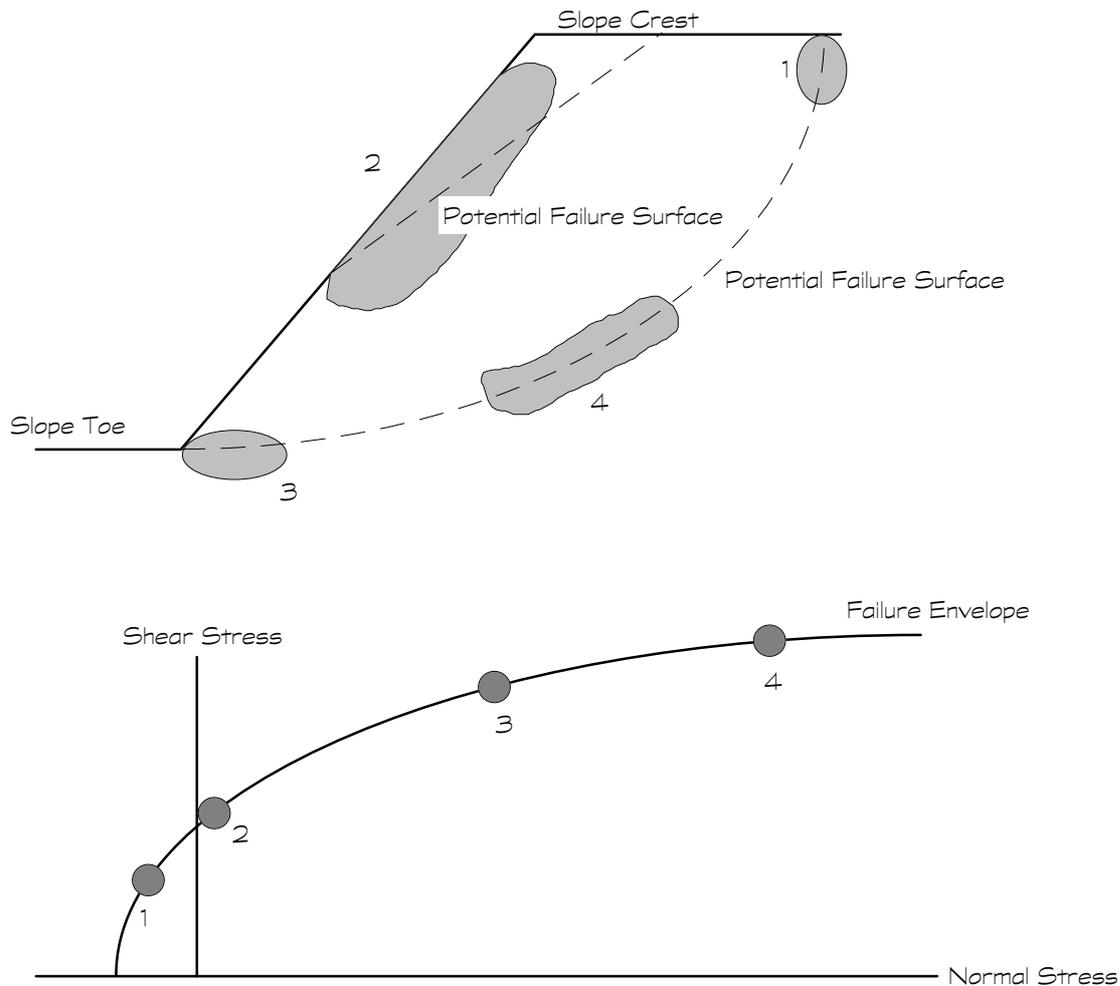


Figure 2.5 Schematic illustration of the stress state at different points along two potential failure surfaces in a rock slope, and on a corresponding curved failure envelope.

The difference between estimated strength values using *undisturbed* and *disturbed rock masses* can be huge, depending on the value *RMR* (or *GSI*). Consider two cases, one with an *RMR* of 80 and one with an *RMR* of 35 (good and poor rock, respectively). The calculated uniaxial compressive strengths for the rock mass, using the updated Hoek-Brown criterion are shown in Table 2.2. The ratio between the calculated strengths for *undisturbed* and *disturbed rock masses* is also shown. We find that the difference can be quite large, up to 6 times for *RMR* = 35. A similar strength reduction could be achieved by reducing the *RMR*-value as shown in Table 2.2.

Table 2.2 Calculated uniaxial compressive strength for the rock mass, using parameters for *undisturbed* and *disturbed rock mass*, respectively.

	<i>RMR</i> = 35	<i>RMR</i> = 80
Uniaxial compressive strength: <i>Undisturbed rock mass</i> [MPa]	0.675	65.8
Uniaxial compressive strength: <i>Disturbed rock mass</i> [MPa]	0.111	37.8
$\sigma_{c,Undisturbed} / \sigma_{c,Disturbed}$	6.08	1.74
Adjusted <i>RMR</i> to achieve $\sigma_{c,Undisturbed} = \sigma_{c,Disturbed}$	3	70
<i>RMR</i> -difference	-32	-10

While this is a reasonable strategy for relatively good rock quality (high *RMR*), the reduction is unrealistically large for lower *RMR*-values. Whether this is because the *disturbed* values are simply too low is difficult to say. These values were, after all, developed already in the original criterion, thus forming the basis for the entire failure criterion. This also highlights the problem with the limited amount of data forming the basis for the criterion. In any case, and for all practical applications, simply reducing the *RMR*-value to obtain strength values corresponding to *disturbed* rock can only be recommended for rock masses exhibiting high *RMR*-values.

Finally, a small note on the relation between *disturbed* and *undisturbed* rock strengths. This ratio is constant for a constant *RMR*-value, compare Equations 2.4 and 2.6. In reality, this means that it is independent of rock type (does not depend on m_i), which might not necessarily reflect actual conditions. Some rock types, e.g., igneous rocks with high intrinsic interlocking, could exhibit a larger reduction in strength if this interlocking is destroyed (becoming *disturbed*), compared to e.g., clastic, sedimentary rocks.

Use of Rock Mass Classification

The use of the *RMR* or *NGI* classification systems is in itself coupled with uncertainties. Neither system can be said to fully characterize the rock mass and several of the factors included are very subjective and difficult to assess quantitatively, which might also influence the strength estimates.

It should also be noted that in particular the *CSIR* classification system includes several parameters that implicitly define rock mass strength. The intact uniaxial compressive rock strength, σ_c , is one example of a parameter that thus is included twice in the strength estimate for the rock mass. The implications of this are probably limited, considering the other uncertainties in the process of parameter estimation as discussed above. It is also realized that by using well-known and established classification systems, it is easier to gain acceptance of the criterion and perhaps more importantly, to have a common basis upon which improvements and enhancements can be made.

Summary

Taken together, these facts show that the procedure for estimating parameters for the Hoek-Brown failure criterion is far from clean cut. Depending on what version of the failure criterion one uses, large differences in final strength values can be obtained, simply by choosing values for *undisturbed* or *disturbed rock mass*. In particular for rock slopes, these discrepancies are troublesome and a strong word of caution is therefore justified. Ideally, values obtained should be correlated to field estimates, e.g., through back-analysis, before used in stability analyses of critical slope sections. An alternative interpretation of the categories *undisturbed* and *disturbed* rock mass will be discussed in Chapter 4 of this report.

3 EQUIVALENT COHESION AND FRICTION ANGLE

3.1 The Mohr-Coulomb Failure Criterion

The Mohr-Coulomb failure criterion is defined as

$$\tau = c + \sigma'_n \tan \phi \quad (3.1)$$

or

$$\sigma'_1 = \sigma'_c + \sigma'_3 \frac{1 + \sin \phi}{1 - \sin \phi} \quad (3.2)$$

where

- τ = the shear stress at failure,
- σ'_n = the effective normal stress,
- c = the cohesion of the rock mass, and
- ϕ = the friction angle of the rock mass.

Equations 3.1 and 3.2 are both straight lines in the τ - σ'_n - and σ_1 - σ_3 -plane, respectively, compared to the curved Hoek-Brown failure envelope (compare Figures 2.2 and 3.1).

The uniaxial compressive and tensile strength for a Mohr-Coulomb material, σ_c and σ_t , respectively, can be expressed as

$$\sigma_c = \frac{2c \cdot \cos \phi}{1 - \sin \phi}, \quad (3.3)$$

$$\sigma_t = -\frac{2c \cdot \cos \phi}{1 + \sin \phi}. \quad (3.4)$$

The tensile strength predicted from Equation 3.4 is often quite high, in particular for low friction angles. Furthermore, Equation 3.1 does not have any physical meaning when the normal stress becomes negative. Based on this, it is customary to use a lower value for the tensile strength by specifying a "tension cut-off" for the failure envelope, see Figure 3.1.

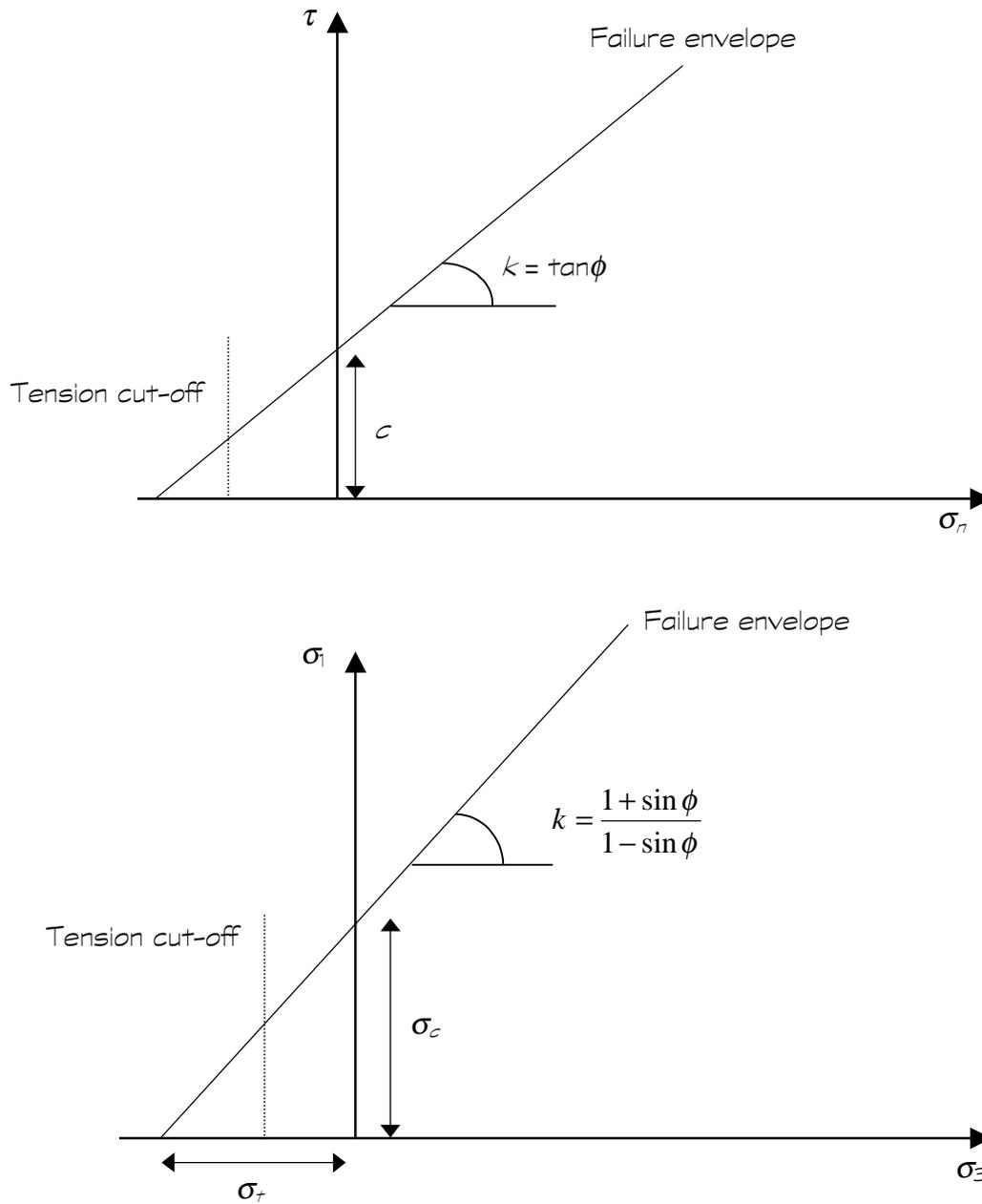


Figure 3.1 Mohr-Coulomb failure envelope in the τ - σ_n -plane and the σ_1 - σ_3 -plane.

3.2 Methods for Determining Equivalent Cohesion and Friction Angle

3.2.1 Introduction

In most of the currently available design methods for rock mechanics analysis, such as limit equilibrium methods and numerical models, the strength is expressed in terms of the Mohr-Coulomb failure criterion. There are a few, but increasing, number of numerical methods

which also can handle curved failure envelopes and even the Hoek-Brown criterion for plasticity calculations. Some of these methods, e.g., *FLAC* (Itasca, 1995), "convert" a curved failure envelope to a linear Mohr-Coulomb envelope by determining the tangent to the curved envelope at the calculated stress in each element and for each calculation step. Consequently, the Mohr-Coulomb cohesion and friction angle will vary throughout the model. For many practical applications, it is necessary to approximate the curved Hoek-Brown failure envelope with a single set of strength parameters for the straight-line Mohr-Coulomb failure envelope. Consequently, we need some methodology for determining an equivalent cohesion and friction angle for the Hoek-Brown failure envelope.

For applications in which only the major and minor principal stress and their relation to the strength of the rock is of importance, determining equivalent cohesion and friction angle can be done by finding the tangent to the Hoek-Brown envelope at a specified minor principal stress. An alternative would be to calculate a regression line over a specified interval for the minor principal stress, thus determining "average" values for cohesion and friction angle. Both approaches will be outlined below.

In other applications, notable slope stability analysis using limit equilibrium methods, the shear strength of a failure surface under specified normal stress is required. For this it is necessary to express the Hoek-Brown failure criterion in terms of shear and normal stresses. Once this has been done, equivalent cohesion and friction angle can be determined either from the tangent at a specified normal stress or from a regression line over an anticipated normal stress range. The conversion of the Hoek-Brown criterion to a corresponding Mohr envelope differs, depending on whether we adopt the original or the modified Hoek-Brown criterions. In the following, these two are therefore treated separately.

It is important to realize that by using a linear failure envelope in place of curved, the final result of a stability analysis can be affected. For slopes, Charles (1982) showed that when applying limit equilibrium analysis to rotational shear failures, a curved failure envelope resulted in the critical failure surface being more deep-seated. A drastic change in failure mechanism is, however, not likely when converting to a linear failure envelope, as long as a stress range appropriate for the problem at hand is used. Choice of stress range for determining equivalent strength parameters will be discussed at the end of this Chapter.

3.2.2 Cohesion and Friction Angle Derived from the Original Hoek-Brown Criterion

Cohesion and Friction Angle for a Specified Normal Stress

The Mohr failure envelope corresponding to the original Hoek-Brown failure criterion was derived by Dr. John Bray of Imperial College and is given by (Hoek, 1993, Hoek and Brown, 1988, Hoek, 1990) as

$$\tau = (\cot\phi_i - \cos\phi_i) \frac{m\sigma_c}{8}, \quad (3.5)$$

where ϕ_i is the instantaneous friction angle at the given value of τ and σ_n , see Figure 3.2. Londe (1988) also presented a similar solution.

The value of the instantaneous friction angle is given by

$$\phi_i = \arctan\left(\frac{1}{\sqrt{4h \cos^2 \theta - 1}}\right), \quad (3.6)$$

$$\theta = \frac{1}{3} \left(90 + \arctan \frac{1}{\sqrt{h^3 - 1}} \right), \quad (3.7)$$

$$h = 1 + \frac{16(m\sigma_n' + s\sigma_c)}{3m^2\sigma_c}. \quad (3.8)$$

The instantaneous cohesion, c_i , can be calculated from

$$c_i = \tau - \sigma_n' \tan \phi_i. \quad (3.9)$$

Calculations of the uniaxial tensile strength using the above formulation will yield a slightly different value than from using Equation 2.3. This is because the radius of curvature of the Mohr envelope on the tensile side is not necessarily the same as the radius of the Mohr circle defining the uniaxial tensile strength (Hoek and Brown, 1988). For most practical applications, this difference is small and can be neglected. Furthermore, if assuming that the tensile strength of the rock mass is zero, this problem disappears completely.

Cohesion and Friction Angle for a Specified Minor Principal Stress

For a specified value of σ_3 , the corresponding major principal stress at failure, σ_1 , is calculated from Equation 2.1 (see also Figure 3.2). The friction angle can be found from (Hoek, 1990)

$$\sigma_n = \sigma_3' + \frac{(\sigma_1' - \sigma_3')^2}{2(\sigma_1' - \sigma_3') + \frac{1}{2}m\sigma_c}, \quad (3.10)$$

$$\tau = (\sigma_n' - \sigma_3') \sqrt{1 + \frac{m\sigma_c}{2(\sigma_1' - \sigma_3')}}}, \quad (3.11)$$

$$\phi_i = 90 - \arcsin\left(\frac{2\tau}{\sigma_1' - \sigma_3'}\right), \quad (3.12)$$

and the cohesion is then calculated from Equation 3.9.

Cohesion and Friction Angle for Equal Compressive Strength

For the condition that the uniaxial compressive strength for the Hoek-Brown failure criterion (Equation 2.2) and the Mohr-Coulomb criterion (Equation 3.3) are the same (see Figure 3.2), the friction angle can be calculated from (Hoek, 1990)

$$\sigma_n' = \frac{2s}{4\sqrt{s} + m}, \quad (3.13)$$

$$\tau = \sigma_n' \sqrt{1 + \frac{m}{2\sqrt{s}}}, \quad (3.14)$$

$$\phi_i = 90 - \arcsin\left(\frac{2\tau}{\sigma_c \sqrt{s}}\right). \quad (3.15)$$

Finally, the cohesion can be calculated from Equation 3.9. In reality, this approach corresponds to finding the tangent at $\sigma_3 = 0$.

Regression Line over a Stress Range

Once a set of (σ_3, σ_1) pairs have been determined, it is also possible to determine equivalent cohesion and friction angle over a larger range of the minor principal stress, by conducting a regression analysis on these data points. A least square straight-line fit gives the following relations for the slope of the regression line, k ,

$$k = \frac{n \sum \sigma_1' \sigma_3' - (\sum \sigma_1' \sum \sigma_3')}{n \sum (\sigma_3')^2 - (\sum \sigma_3')^2}, \quad (3.16)$$

where the summations are done over the number of (σ_3, σ_1) pairs, n .

Using Equations 3.16 and 3.2, the friction angle can be calculated as

$$\phi = \arcsin \frac{k-1}{k+1}. \quad (3.17)$$

The intercept with the σ_1 -axis will give the uniaxial compressive strength for the rock mass as

$$\sigma_{c,rockmass} = \frac{\sum (\sigma_3')^2 \sum \sigma_1' - \sum \sigma_3' \sum \sigma_1' \sigma_3'}{n \sum (\sigma_3')^2 - (\sum \sigma_3')^2}. \quad (3.18)$$

The equivalent cohesion is calculated, using Equation 3.3, as

$$c = \frac{\sigma_{c,rockmass} (1 - \sin \phi)}{2 \cos \phi}. \quad (3.19)$$

A standard linear regression will typically yield a compressive strength (from Equation 3.18) which is lower than that predicted by the Hoek-Brown criterion (Equation 2.2). An alternative method of curve fitting is to fix the intercept with the σ_1 -axis at the value given by Equation 2.2 (equal uniaxial compressive strengths for the Hoek-Brown and the Mohr-Coulomb criteria), and then calculate the slope of the corresponding regression line which best fit the data points, using the least square method, see Figure 3.2. This gives

$$k = \frac{\sum \sigma_1' \sigma_3' - \sigma_c \sqrt{s} \sum \sigma_3'}{\sum (\sigma_3')^2}, \quad (3.20)$$

where the summation is carried out over the number of data pairs.

The corresponding friction angle and cohesion can then be determined from Equations 3.17 and 3.19.

It is also possible to carry out the regression in the τ - σ_n -plane. This is done by calculating the shear stress, τ , for a specified normal stress, σ_n , using, in order, Equations 3.8, 3.7, 3.6 and 3.5. Once a set of $(\sigma_n - \tau)$ pairs have been calculated, the regression analysis is equivalent to that described above and using Equations 3.18 and 3.20 with σ_3 substituted with σ_n , and σ_1 substituted with τ . The friction angle can then be calculated from the slope of this regression line (see also Figure 3.1) as

$$\phi = \arctan k , \quad (3.21)$$

and the cohesion is found from the intercept with the vertical axis.

ORIGINAL HOEK-BROWN FAILURE CRITERION

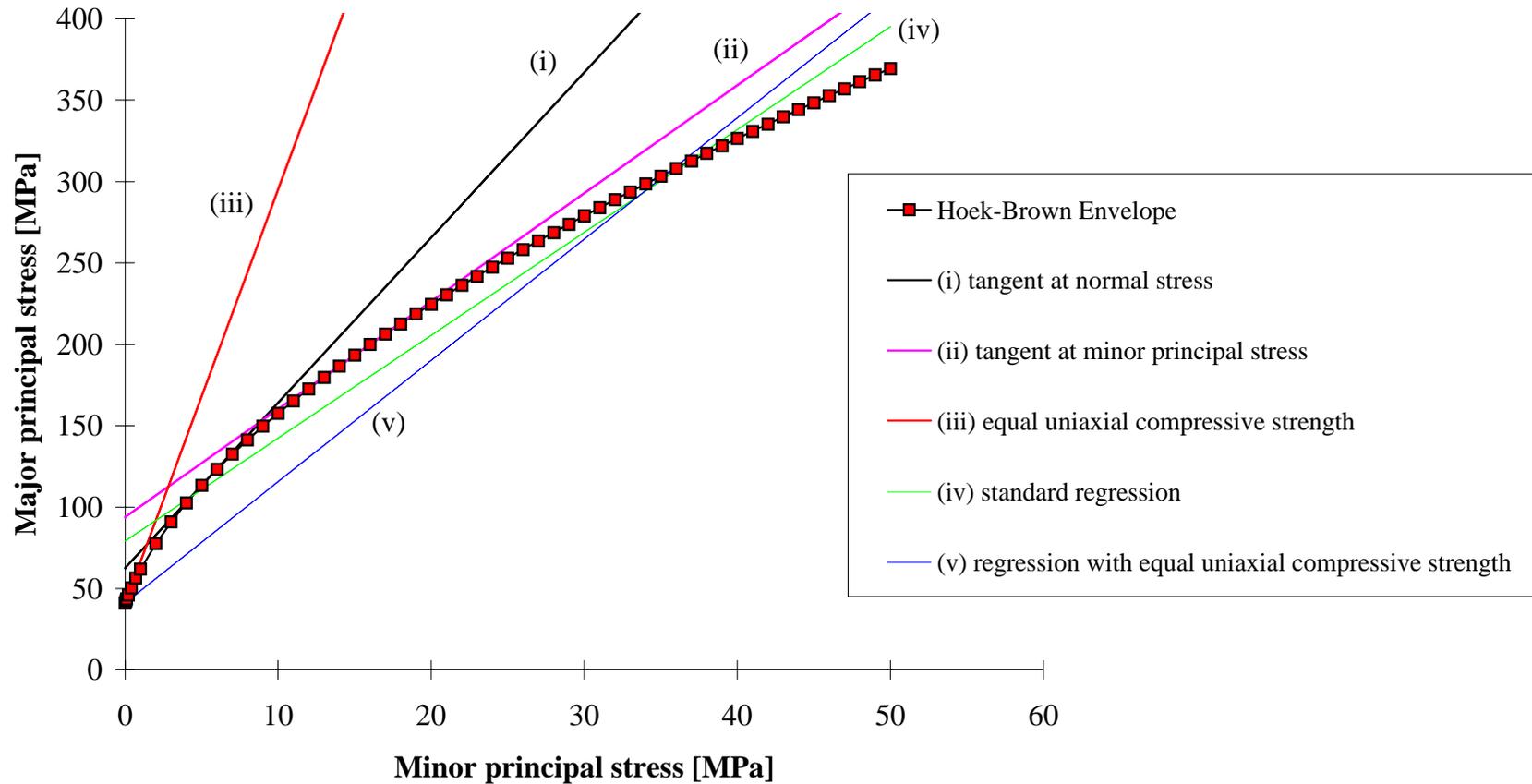


Figure 3.2 Example of different methods for determining equivalent cohesion and friction angle from the original Hoek-Brown failure criterion: (i) tangent at $\sigma_n = 15$ MPa, (ii) tangent at $\sigma_3 = 15$ MPa, (iii) equal uniaxial compressive strength for the Hoek-Brown and Mohr-Coulomb failure criteria, (iv) linear regression over $\sigma_3 = 0 - 50$ MPa, (v) linear regression over $\sigma_3 = 0 - 50$ MPa with equal uniaxial compressive strength for the Mohr-Coulomb and the Hoek-Brown failure criterion.

3.2.3 Cohesion and Friction Angle Derived from the Modified and Generalized Hoek-Brown Criterion

For the case of the modified Hoek-Brown failure criterion, a closed form solution for the corresponding Mohr envelope cannot be derived. A general analytical solution for the Mohr envelope was published by Balmer (1952) in which the normal and shear stresses are expressed as follows

$$\sigma'_n = \sigma'_3 + \frac{\sigma'_1 - \sigma'_3}{\partial\sigma'_1/\partial\sigma'_3 + 1}, \quad (3.22)$$

$$\tau = (\sigma'_n - \sigma'_3) \sqrt{\partial\sigma'_1/\partial\sigma'_3}. \quad (3.23)$$

The derivative $\partial\sigma'_1/\partial\sigma'_3$ can be calculated from the general formulation of the Hoek-Brown failure criterion (Equation 2.9) (Hoek and Brown, 1980; Hoek, Wood and Shah, 1992; Hoek, Kaiser and Bawden, 1995) using the following relations:

For $GSI > 25$, when $a = 0.5$

$$\frac{\partial\sigma'_1}{\partial\sigma'_3} = 1 + \frac{m_b \sigma_c}{2(\sigma'_1 - \sigma'_3)}. \quad (3.24)$$

For $GSI < 25$, when $s = 0$

$$\frac{\partial\sigma'_1}{\partial\sigma'_3} = 1 + a(m_b)^a \left(\frac{\sigma'_3}{\sigma_c} \right)^{a-1}. \quad (3.25)$$

Equation 3.24 represents the original Hoek-Brown failure criterion, and the technique described below is equally applicable for this formulation. Once a set of (σ_n, τ) values have been determined from Equations 3.22 to 3.25, average cohesion and friction angle can be calculated by linear regression analysis over the normal stress range. This gives the following expressions for c and ϕ :

$$\phi = \arctan \left(\frac{n \sum \sigma'_n \tau - (\sum \tau \sum \sigma'_n)}{n \sum (\sigma'_n)^2 - (\sum \sigma'_n)^2} \right), \quad (3.26)$$

$$c = \frac{\sum \tau}{n} - \frac{\sum \sigma'_n}{n} \tan \phi . \quad (3.27)$$

If we instead want to do the regression over a specified range of minor principal stress, we can use Equations 3.16 to 3.20 to calculate equivalent cohesion and friction angle.

The instantaneous friction angle and cohesion for a specified minor principal stress (rather than over a range of stress) can be calculated from the modified Hoek-Brown criterion, by fitting a tangent to the Hoek-Brown envelope. The slope of this tangent is found from Equation 3.2 (see also Figure 3.1), which in turn must be equal to the derivative in Equation 3.25:

$$k = \frac{1 + \sin \phi}{1 - \sin \phi} = \frac{\partial \sigma'_1}{\partial \sigma'_3} . \quad (3.28)$$

From this we can solve for ϕ , which yields

$$\phi = \arctan \frac{k - 1}{k + 1} = \frac{\partial \sigma'_1 / \partial \sigma'_3 - 1}{\partial \sigma'_1 / \partial \sigma'_3 + 1} . \quad (3.29)$$

The cohesion can then be calculated from Equation 3.19.

3.3 Discussion

The choice of method to use for determining equivalent cohesion and friction angle is largely a matter of taste. An equivalent cohesion and friction angle at a specified normal or minor principal stress gives the most accurate values, but only for that specific stress state. By using a regression line over a larger stress range, average values applicable to a wider range of conditions are obtained. This could, however, lead to an underestimate of the strength for low stress and an overestimate of strength values for high stress, see Figure 3.2.

In any case, the choice of stress state at which the equivalent cohesion and friction angle is to be evaluated should be based on the anticipated stress in the rock mass around the excavation at question as was also pointed out by Charles (1982). The stress range will thus be different for different types of slopes or underground excavations. An upper bound estimate of the stress state can be found from an elastic stress analysis for the opening or slope. In reality, and in particular after failure has occurred, the actual stress state will be lower than that

predicted from a purely elastic analysis. Using the elastic stress range in the estimation of equivalent cohesion and friction angle could thus give slightly lower friction angle and slightly higher cohesion, depending on the curvature of the actual Hoek-Brown failure envelope.

Here, one word of caution is necessary. For the modified and generalized Hoek-Brown criterion, Hoek, Kaiser and Bawden (1995) gave an algorithm and a spreadsheet solution for determining c and ϕ using Equations 3.22 to 3.27. In this spreadsheet, the stress range over which the regression is to be conducted is given by

$$\frac{\sigma_3}{\sigma_c} = \frac{1}{2^i} \quad (3.30)$$

where i varies from 9 to 1 in increments of -1.

This was done to capture the pronounced curvature of the Hoek-Brown envelope at low normal stresses (Hoek, Wood and Shah, 1992). This gives a maximum value for σ_3 of $0.5\sigma_c$. If this approach is used routinely, the stress range will vary depending on the uniaxial compressive strength of the rock. This is acceptable in cases where the actual stress state is unknown. However, for the case of a rock mass comprising several rock types with varying strength, it will lead to different regression intervals for each rock type, even if these rocks in reality are subjected to the same stress state after excavation or mining. In conclusion, a better approach is to use the same stress range for all parameter estimates and determine this stress range from elastic stress analysis of the construction element.

4 STRENGTH OF THE ROCK MASS AT THE AZNALCOLLAR OPEN PIT

4.1 Mine Description

The Aznalcollar open pit mine is situated in southern Spain, just northwest of the city of Seville. Open pit mining started in the early 1970's and commenced until September 1996. The mine was acquired by Boliden Mineral AB in 1987 and has since been operated by Andaluza de Piritas SA (APIRSA), a subsidiary of Boliden Mineral AB. Production during the last few years of operation amounted to approximately 2.3 Mton of ore annually with a stripping ratio of up to 8.7:1 (waste:ore).

The lead-zinc orebody strikes east-west and dips to the north at angle of 45° to 60° (becoming steeper with depth). The dominant footwall rock types are slates and schists with well-developed cleavage. The cleavage discontinuities dip between 45° and 70°, running subparallel to the orebody. Phyllites and quartz veins are also found in the footwall. The hangingwall comprises slates, tuffs (pyroclastics), felsites and dacites/rhyolites.

At the end of mining, the pit measured approximately 1300 by 700 meters with a pit depth of 270 meters. Overall slope angles varied from 30° to 38°. Despite the relative moderate slope height and the flat slope angles, the mine has suffered several large scale failures of the footwall slope. In the central portion of the footwall, failures occurred in 1983, 1987, 1988, 1989 and 1992. Slope monitoring using aerial photography, surface displacement stations and inclinometers, enabled failure surfaces to be determined, as shown in Figure 4.1. The slope has been moving continuously, with displacement rates varying from less than a few mm per day up to 1680 mm/day (July 1992).

Final mining of the eastern footwall of the pit involved steepening of the overall slope angle. To secure mining, an extensive monitoring program was installed. This included additional inclinometers, extensometers, a hydraulic leveling system, surface displacement measurement, and piezometers for monitoring groundwater pressures. Mining was successfully completed in the fall of 1996, but subsequent to this, two large scale failure surfaces developed in the eastern footwall resulting in huge scarps in the footwall ramp. The pit is currently being backfilled with waste material from the nearby Los Frailes open pit.

The hangingwall of the Aznalcollar mine has proven to be significantly stronger than the footwall. Despite steeper slope angle, the hangingwall did not experience any failure up until mine closure.

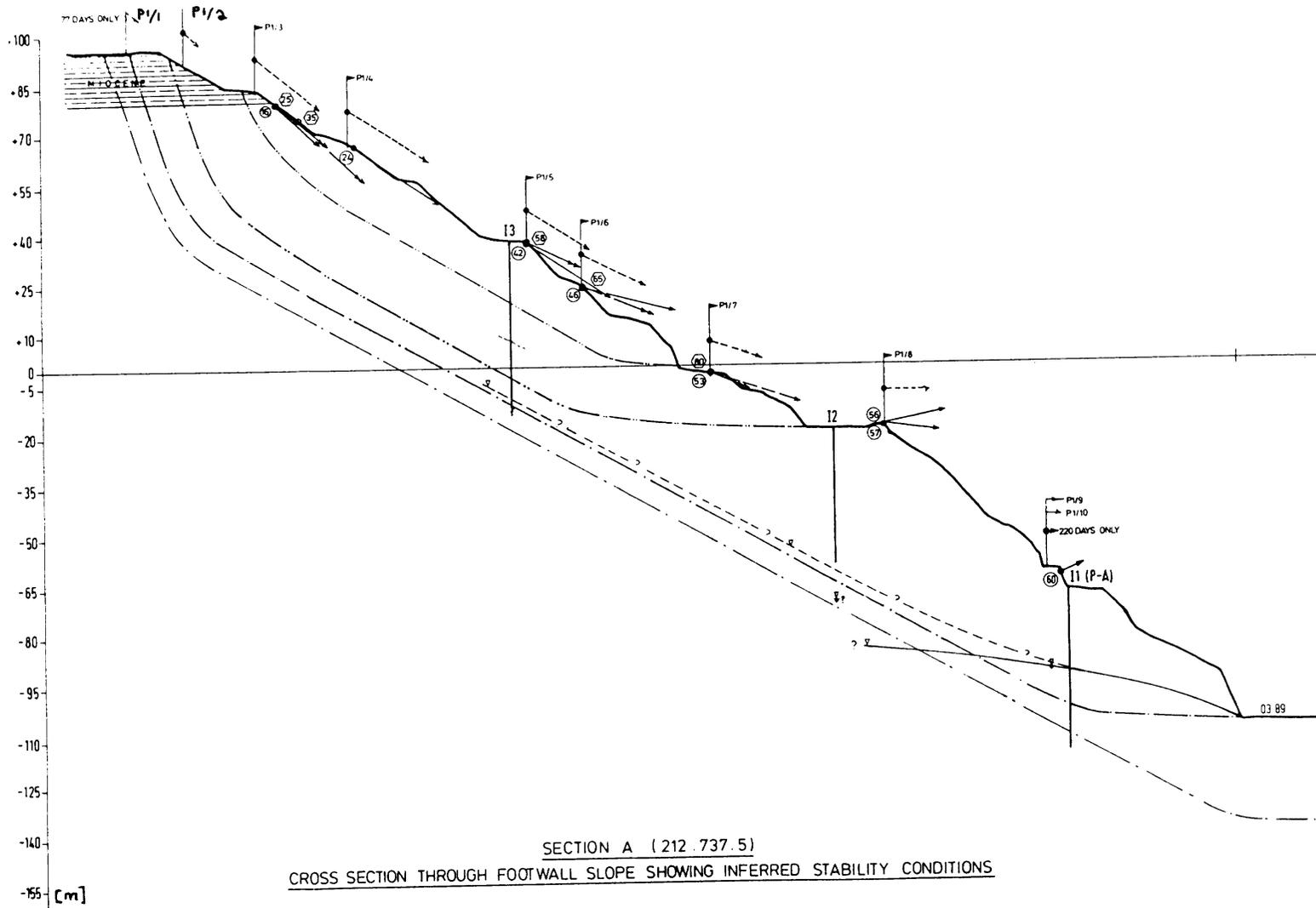


Figure 4.1 Cross section through the central portion of the footwall of the Aznalcollar open pit, showing inclinometer locations (I1, I2, I3), monitoring points (P1/1 to P1/9) and displacement vectors, and inferred failure surfaces (from Golder Associates, 1989a).

4.2 Aznalcollar Hangingwall Strength

The strength of the hangingwall rocks at the Aznalcollar open pit mine was estimated by Golder Associates (UK) (1996a), using the generalized Hoek-Brown failure criterion and Equations 3.22 to 3.27, and 3.30, see also Section 3.2.3. The results are reproduced in Table 4.1. Only values for *undisturbed rock mass* were used, in accordance with Hoek, Kaiser and Bawden (1995). The various values for m_i , σ_c , and GSI correspond to the estimated variation of these parameters.

Note that different stress ranges (in terms of σ_3 and σ_n) are used for the different rock types. This is a consequence of adopting the procedure suggested by Hoek, Kaiser and Bawden (1995), as discussed in Section 3.3.

Table 4.1 Estimated rock mass strength for the Aznalcollar hangingwall, assuming *undisturbed rock mass*. From Golder Associates UK Ltd. (1996a).

Rock Type	m_i	σ_c [MPa]	GSI	σ_{n-min} [MPa]	σ_{n-max} [MPa]	σ_{3-min} [MPa]	σ_{3-max} [MPa]	ϕ [°]	c [MPa]
Pyrite Ore	17	100	76	0	29.9	0	12.5	47.8	5.7
Pyrite Ore	17	70	74	0	20.7	0	8.75	47.5	3.7
Pyroclastics	17	70	67	0	19.9	0	8.75	46.3	2.8
Pyroclastics	17	70	71	0	20.3	0	8.75	47	3.2
Pyroclastics	17	60	83	0	18.9	0	7.5	48.7	4.7
Dacite	17	60	83	0	18.9	0	7.5	48.7	4.7
Slate	9	25	61	0	6.5	0	3.13	38.9	0.8
Slate	9	35	62	0	9.1	0	4.38	39.1	1.2
Slate	9	35	66	0	9.3	0	4.38	39.8	1.4
Slate	9	15	48	0	3.6	0	1.88	36.1	0.3

It is interesting to compare these values with the estimates provided by Golder (1989b). This gave $\phi = 21 - 27^\circ$ and $c = 0.47 - 0.58$ MPa, for a normal stress of 2 MPa, i. e. much lower strength than the values in Table 4.1. Also, Golder (1996b) conducted a back-analysis of the hangingwall, using limit equilibrium analysis. By assuming that the stable slope had a safety factor of between 1.2 and 1.5, this analysis gave the required strengths for stability as $\phi = 30 - 40^\circ$, and $c = 0.12 - 0.83$ MPa, i.e. significantly lower than indicated by Table 4.1.

The varying stress ranges used in Table 4.1 makes direct comparisons between the different rock units difficult. A lower stress range will, however, result in higher friction angles and slightly lower cohesions. This is very pronounced for the slate unit in Table 4.1, for which relatively high friction angles were achieved, despite the low intact rock strength and rock mass quality rating for this rock type.

To compare these values, additional estimates were done by the author using values for *disturbed rock mass* (Equations 2.6 and 2.7). The category *disturbed rock mass* was chosen as this was believed to better reflect conditions in a rock slope, compare Section 2.7.3. The same values for m_i , σ_c , and GSI as in Table 4.1 were used. The anticipated stress state to which the rock mass is subjected was estimated from a linear elastic numerical stress analysis. This analysis showed that stresses vary significantly, depending on what portion of the slope that is considered, compare Figure 2.5. The minor principal stress ranged from nearly zero at the crest to about 11 MPa at the toe of the slope. The interior of the slope exhibited much smaller stress variations, typically $\sigma_3 = 0 - 5$ MPa. It was thus decided to use two stress ranges for the regression analysis, as follows:

- i. Entire slope (toe, crest, interior): $\sigma_3 = 0 - 11$ MPa
- ii. Interior of the slope (excluding the toe): $\sigma_3 = 0 - 5$ MPa

A major difference compared to Table 4.1 is thus that the same stress range is used for all rock types. Equivalent cohesion and friction angle was then calculated by means of linear regression over the specified stress range, using Equations 3.16 to 3.20, see Section 3.2.2. The obtained strength parameters are shown in Tables 4.2 and 4.3.

As expected, lower stresses (smaller stress range) resulted in higher friction angles and slightly lower cohesions. In this particular case, the lower stress range ($\sigma_3 = 0 - 5$ MPa) gave approximately 5° higher friction angle. When comparing Tables 4.2 and 4.3 with Table 4.1 it is found that for the pyrite ore and the dacite, only slightly lower values on c and ϕ were obtained by using values for *disturbed rock mass*. For the pyroclastics, and in particular, the slate, the differences between strength values in Table 4.1 versus Tables 4.2 and 4.3 are much larger. It can be concluded that for rocks with low classification ratings (RMR approximately less than 60), results are extremely sensitive to the choice of *undisturbed* or *disturbed rock mass*.

Table 4.2 Estimated rock mass strength for the Aznalcollar hangingwall assuming *disturbed rock mass*, for a stress range of $\sigma_3 = 0 - 11$ MPa.

Rock Type	m_i	σ_c [MPa]	$GSI =$ RMR	σ_{3-min} [MPa]	σ_{3-max} [MPa]	Standard regression (Equations 3.16-3.19)		Fixed intercept (Equations 3.17-3.20)	
						ϕ [°]	c [MPa]	ϕ [°]	c [MPa]
Pyrite Ore	17	100	76	0	11	42.9	3.66	44.5	2.84
Pyrite Ore	17	70	74	0	11	39.4	2.64	41.4	1.81
Pyroclastics	17	70	67	0	11	35.8	1.92	38.1	1.09
Pyroclastics	17	70	71	0	11	37.9	2.29	40.0	1.45
Pyroclastics	17	60	83	0	11	42.5	3.87	44.1	3.08
Dacite	17	60	83	0	11	42.5	3.87	44.1	3.08
Slate	9	25	61	0	11	19.8	0.81	22.0	0.33
Slate	9	35	62	0	11	22.5	0.99	24.8	0.47
Slate	9	35	66	0	11	24.4	1.17	26.6	0.64
Slate	9	15	48	0	11	11.8	0.41	13.7	0.08

Table 4.3 Estimated rock mass strength for the Aznalcollar hangingwall assuming *disturbed rock mass*, for a stress range of $\sigma_3 = 0 - 5$ MPa.

Rock Type	m_i	σ_c [MPa]	$GSI =$ RMR	σ_{3-min} [MPa]	σ_{3-max} [MPa]	Standard regression (Equations 3.16-3.19)		Fixed intercept (Equations 3.17-3.20)	
						ϕ [°]	c [MPa]	ϕ [°]	c [MPa]
Pyrite Ore	17	100	76	0	5	47.6	2.90	48.8	2.54
Pyrite Ore	17	70	74	0	5	44.7	1.99	46.2	1.62
Pyroclastics	17	70	67	0	5	41.6	1.37	43.5	0.96
Pyroclastics	17	70	71	0	5	43.4	1.68	45.1	1.29
Pyroclastics	17	60	83	0	5	47.1	3.11	48.2	2.78
Dacite	17	60	83	0	5	47.1	3.11	48.2	2.78
Slate	9	25	61	0	5	25.0	0.55	27.2	0.30
Slate	9	35	62	0	5	27.9	0.69	30.0	0.43
Slate	9	35	66	0	5	29.8	0.84	31.8	0.57
Slate	9	15	48	0	5	15.7	0.27	18.0	0.07

On the other hand, the values in Tables 4.2 and 4.3, especially for the slate unit, compare relatively well with the back-calculated strengths reported earlier ($\phi = 30 - 40^\circ$, and $c = 0.12 - 0.83$ MPa) suggesting perhaps that the category *disturbed rock mass* is the more suitable choice in this case. Since no failures have occurred in the hangingwall, this is difficult to prove with any certainty.

4.3 Aznalcollar Footwall Strength

The strength of the Aznalcollar footwall has been the subject of several studies. A summary of estimated strengths is given in Table 4.4. The results from more recent back-analyses of the observed footwall failures using limit equilibrium methods, are shown in Table 4.5. Note that the cohesion here is given in kPa, since the strength values in general are much lower than for the hangingwall rocks.

Table 4.4 Summary of estimated strengths for the Aznalcollar footwall.

Rock Type	ϕ [°]	c [kPa]	Comments	Reference
Schist - rock mass	30-40	?	Back-analysis using limit equilibrium method	Golder (1989a)
Schist - foliation	22-30	0	Estimated	Golder (1989a)
Schist - rock mass	24-40	120	Back-analysis using numerical modeling	Proughten (1991)
Slate - intact rock	30-32	0-2000	Back-analysis using numerical modeling	McCullough (1993)
Slate - dry joints	27	0	Back-analysis using numerical modeling	McCullough (1993)
Slate - wet joints	20-25	0	Back-analysis using numerical modeling	McCullough (1993)
Slate - rock mass Active zone	25-28	0	Back-analysis using limit equilibrium method	Golder (1995)
Slate - rock mass Passive zone	35	35-325	Back-analysis using limit equilibrium method	Golder (1995)

Table 4.5 Strength values from back-analysis of previous failures at the Aznalcollar mine, using different limit equilibrium methods. Analyses with the program *SLIDE* 2.05 (Curran et al., 1995), using Bishop's routine method, were carried out by the author.

Failure Geometry						Limit equilibrium analysis using <i>SLIDE</i> 2.05		Limit equilibrium analysis using Hoek's charts (Krauland, 1995)					
Year	Section	Slope Crests [m]	Slope Toe [m]	Slope Height [m]	Slope Angle [°]	Drained Conditions		Drained Conditions		Chart 3		Chart 5	
						ϕ [°]	c [kPa]	ϕ [°]	c [kPa]	ϕ [°]	c [kPa]	ϕ [°]	c [kPa]
1983	212800	90	5	85	30	20	57	20	70	20	150	20	170
						22	44	25	30	25	100	25	130
								28	20	28	70	28	100
1987-shallow	212850	90	-10	100	32	20	115	20	160	20	310	20	360
						22	95	25	60	25	210	25	280
								28	30	28	150	28	220
1988	213000	95	-85	180	31	20	140	20	160	20	320	20	380
						22	100	25	60	25	220	25	290
								28	30	28	160	28	230
1989	212850	95	-105	200	31	20	150	20	170	20	350	20	420
						22	120	25	70	25	240	25	320
								28	40	28	170	28	250
1989	212737.5	95	-105	200	31	20	145						
						22	105						
1992	212800	85	-125	210	34	20	170	20	210	20	400	20	490
						22	135	25	100	25	280	25	400
								28	60	28	220	28	320
1996	213300	85	-155	240	34	20	235						
						22	175						
	213300	85	-185	270	37	20	300						
1996	213410	85	-185	270	38.5	20	330						
						22	275						

For estimating the strength of the footwall using the Hoek-Brown failure criterion, three sets of classification ratings and estimated uniaxial compressive strengths were used as follows (see also Table 4.4):

1. $RMR = 58$, $\sigma_c = 25$ MPa (Proughten, 1991);
2. $RMR = 58$, $\sigma_c = 50$ MPa (Proughten, 1991; Intecsa, 1988); and
3. $RMR = 44$, $\sigma_c = 35$ MPa (McCulloch, 1993).

In addition, two different values for m_i were used; $m_i = 10$ and $m_i = 11.4$, roughly corresponding to schist and slate. Values for *disturbed rock mass* were used (Equations 2.6 and 2.7) and regression was conducted over the two stress intervals, $\sigma_3 = 0 - 11$ and $\sigma_3 = 0 - 5$ MPa, respectively (Equations 3.16 to 3.20). The resulting strength parameters are summarized in Tables 4.6 and 4.7.

Table 4.6 Estimated rock mass strength for the Aznalcollar footwall assuming *disturbed rock mass*, for a stress range of $\sigma_3 = 0 - 11$ MPa.

Rock Type	m_i	σ_c [MPa]	$GSI =$ RMR	σ_{3-min} [MPa]	σ_{3-max} [MPa]	Standard regression (Equations 3.16-3.19)		Fixed intercept (Equations 3.17-3.20)	
						ϕ [°]	c [kPa]	ϕ [°]	c [kPa]
Schist (1)	10	25	58	0	11	19.2	750	21.5	260
Schist (2)	10	50	58	0	11	23.9	1040	26.2	470
Schist (3)	10	35	44	0	11	15.2	540	17.5	120
Slate (1)	11.4	25	58	0	11	20.1	780	22.5	250
Slate (2)	11.4	50	58	0	11	25.0	1070	27.4	460
Slate (3)	11.4	35	44	0	11	16.0	570	18.3	120

Relatively low strength values are obtained, in particularly for *Set 3* ($RMR = 44$, $\sigma_c = 35$ MPa). Friction angles as low as 15° are probably unrealistic, since this is significantly lower than even the estimated friction for the foliation of $22-30^\circ$, compare Table 4.4.

For *Set 1* and 2, obtained strength values are approximately of the same magnitude as those obtained from back-analyses of failures. For the approach with a fixed intercept with the σ_1 -axis, we find that friction angles are in the range of 22 to 27° with cohesions ranging from 250 to 470 kPa, see Table 4.6. For the lower stress range ($\sigma_3 = 0 - 5$ MPa) we obtain $\phi = 27 - 33^\circ$ and $c = 230 - 420$ kPa (Table 4.7). Back-calculated strengths are in the range of $\phi = 20 -$

28°, $c = 50 - 500$ kPa, depending on what failure occurrence we consider and the assumed groundwater conditions, see Table 4.5. This might indicate that also in this particular case, the values obtained for the category *disturbed rock* compare better to back-calculated failure strengths.

Table 4.7 Estimated rock mass strength for the Aznalcollar footwall assuming *disturbed rock mass*, for a stress range of $\sigma_3 = 0 - 5$ MPa.

Rock Type	m_i	σ_c [MPa]	$GSI =$ RMR	$\sigma_{3-\min}$ [MPa]	$\sigma_{3-\max}$ [MPa]	Standard regression (Equations 3.16-3.19)		Fixed intercept (Equations 3.17-3.20)	
						ϕ [°]	c [kPa]	ϕ [°]	c [kPa]
Schist (1)	10	25	58	0	5	24.4	500	26.8	230
Schist (2)	10	50	58	0	5	29.4	720	31.7	420
Schist (3)	10	35	44	0	5	19.9	360	22.4	110
Slate (1)	11.4	25	58	0	5	25.4	510	28.0	230
Slate (2)	11.4	50	58	0	5	30.6	730	33.0	410
Slate (3)	11.4	35	44	0	5	20.8	370	23.5	110

At this point it is also necessary to consider the pronounced schistosity (or foliation) of the footwall rock at Aznalcollar, as this most likely will result in some strength anisotropy. This strength anisotropy can be assessed using Equation 2.23, as described in Section 2.5. The friction angle of the foliation was estimated to be in the range of 22° to 30° (Golder, 1989a, and Table 4.4). Using values for the rock mass according to *Set 3* ($RMR = 58$, $\sigma_c = 25$ MPa) and a worst case of $\phi_j = 22^\circ$ for the foliation, the resulting strength as a function of the inclination of the foliation relative to the major principal stress was calculated. Here, the minor principal stress was taken to be $\sigma_3 = 5$ MPa and calculations were done both for *disturbed* and *undisturbed rock mass*, see Figures 4.2 and 4.3, respectively.

The strength is reduced for certain orientations of the foliation relative to the major loading direction. Typically, the minimum strength is found for inclinations of around 30° - 35°. The value of the minimum strength is only governed by the discontinuity shear strength which is given by the friction angle for the foliation, ϕ_j , and

$$\sigma_1 = \sigma_3 \frac{1 + \sin \phi_j}{1 - \sin \phi_j} . \quad (4.1)$$

Strength of Schistose Rock, Aznalcollar Footwall
Undisturbed Rock Mass

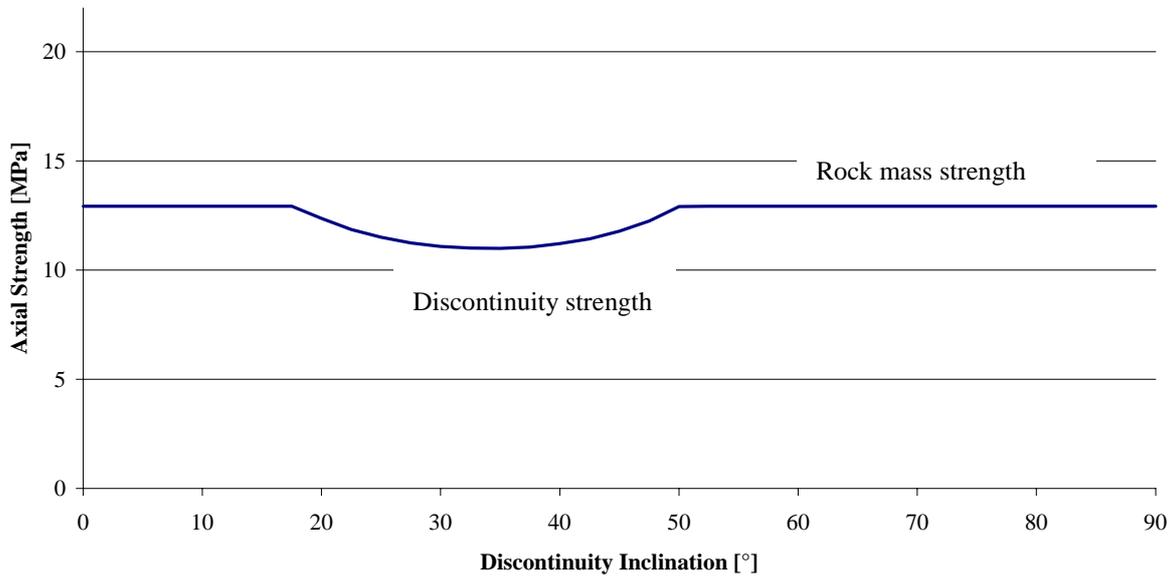


Figure 4.2 Axial strength (σ_1) as a function of the inclination of the foliation (β) relative to the major principal stress, σ_1 . Values for *Set 1* ($RMR = 58$, $\sigma_c = 25$ MPa), *disturbed rock mass*, $\sigma_3 = 5$ MPa, and a foliation friction angle of 22° .

Strength of Schistose Rock, Aznalcollar Footwall
Undisturbed Rock Mass

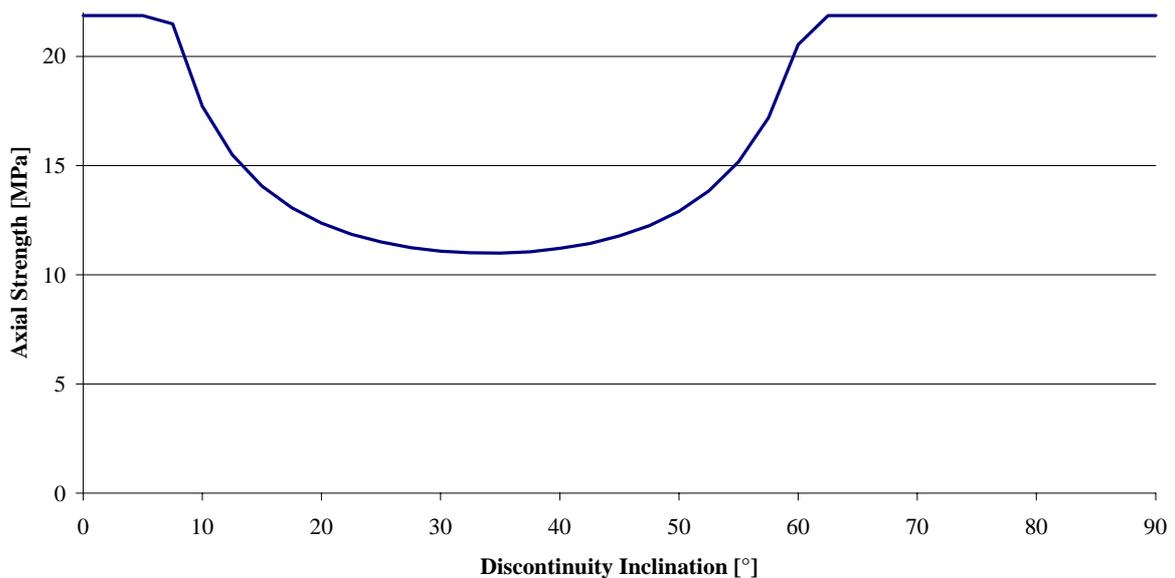


Figure 4.3 Axial strength (σ_1) as a function of the inclination of the foliation (β) relative to the major principal stress, σ_1 . Values for *Set 1* ($RMR = 58$, $\sigma_c = 25$ MPa), *undisturbed rock mass*, $\sigma_3 = 5$ MPa, and a foliation friction angle of 22° .

The inclination between the foliation and the major principal stress will obviously vary from point to point in the pit slope. From numerical elastic stress analysis of the Aznalcollar pit, it was found that close to the slope face the major principal stress is nearly parallel to the face. Farther away from the slope, the major principal stress reorients toward the direction of the major virgin principal stress, in this being horizontal. This is schematically illustrated in Figure 4.4, along with the average dip of the foliation (55°). From this and for a slope angle of $30 - 40^\circ$, we find that the inclination between the foliation and the major principal stress generally is within the interval of $15 - 55^\circ$.

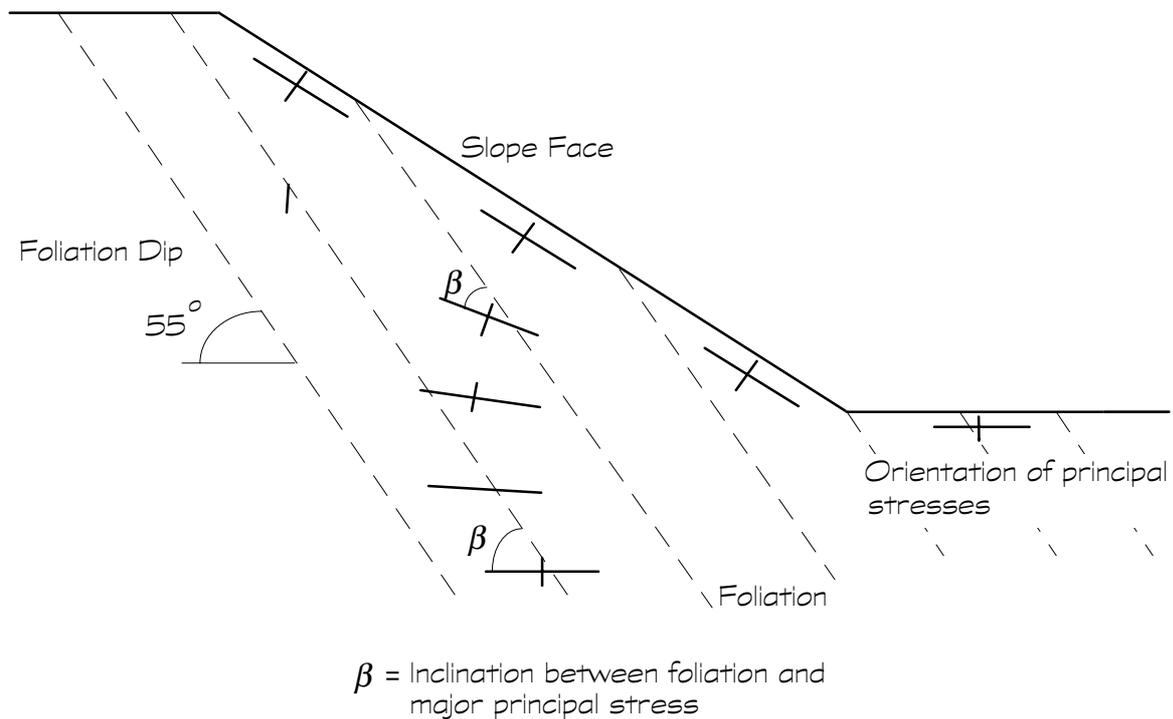


Figure 4.4 Orientation of the foliation and the major and minor principal stresses in the Aznalcollar footwall slope.

This value for the discontinuity inclination (β) falls within the range giving the lowest strength in Figures 4.2 and 4.3. However, the difference between the strength of the discontinuity and the rock strength is relatively small for the case of *disturbed rock mass* (Figure 4.2). This indicates that the rock mass in this case has a strength that is fairly close to that of the foliation. Assuming that the rock mass corresponds to the category of *undisturbed rock mass*, the strength anisotropy is more pronounced. Still, this is for a friction angle for the foliation of only 22° . For higher foliation strength, the differences will be small.

In conclusion, the anisotropy effects are probably relatively small and the obtained strength values in Tables 4.6 and 4.7 can be seen as "average" strengths of the footwall schist including discontinuities.

4.4 Discussion

It can be questioned whether the Hoek-Brown failure criterion is at all applicable for the schistose and foliated rocks at Aznalcollar. This question is especially valid for the schists and slates of the hangingwall and the footwall. The pyrite ore is more or less massive and can be considered isotropic in nature. For the schists and slates, foliation is very dominant but, on the other hand, has not proved to be the controlling factor governing failure. In fact, observed failure surfaces seem to cut across the existing foliation. Although the failure behavior is not yet fully understood, this might indicate that the Hoek-Brown criterion at least could be used for initial estimates of the rock mass strength. Furthermore, in assessing the strength anisotropy caused by the foliation, it was shown that the strength was not much different from the estimated strength for an isotropic rock mass (under the assumption of *disturbed rock mass*).

For the Aznalcollar case, it appears that strength estimates for the category *disturbed rock mass* best fit back-calculated failure strengths. The back-analyzed strengths were obtained from limit equilibrium analysis, which inherently assumes that the shear strength is fully mobilized over the entire failure surface. These strength values are therefore "average" strengths for the entire failure surface in the slope. This could partly explain the apparently low strengths obtained from the back-calculations.

The back-analyzed strengths from limit equilibrium analysis are also representative of post-failure conditions, i.e. failure fully developed with ongoing displacements. These strengths can therefore be regarded as equivalent to the residual rock mass strength. Consequently, the category *disturbed rock mass* could correspond to residual strength conditions. In post-failure conditions, the cohesion of rock mass must, by necessity, be very low, as most of the interlocking along the failure surface has been destroyed. Cohesion values in Tables 4.6 and 4.7 are relatively low but not zero, as perhaps could be expected for residual conditions.

Here, we must remember that cohesion and friction angles merely are parameters by which we are trying to fit observed behavior to a simplistic mechanical model. The friction angle of the rock mass can be interpreted as the friction resistance along pre-existing discontinuities and asperities on these discontinuities (overriding of asperities). The cohesion can be thought

of as the shear resistance of intact rock bridges in the rock mass, or the shear resistance of the asperities on a discontinuity surface (shearing through asperities). A finite cohesion consequently indicates that some amount of intact rock material or local irregularities still exist along the failure surface. Thus, it is conceivable that at least some "apparent" cohesion also can develop along the failure surface, as indicated by Tables 4.6 and 4.7.

Following this trail of thoughts, we could also imagine that the category *undisturbed rock mass* in reality corresponds to pre-failure conditions, i.e. being representative of peak rock mass strength. Once the peak strength is overcome, the strength is reduced to a residual value and failure progresses to the next point in the rock mass. A progressive mechanism cannot be simulated in conventional limit equilibrium methods. Thus, we have no means of back-calculating peak strength values to compare with the category *undisturbed rock mass*. Numerical analysis can be utilized for this purpose, but this is outside the scope of the current report. Data supporting this hypothesis are therefore lacking and more work is required before strict guidelines for when to use *disturbed* versus *undisturbed rock mass* parameters can be formulated.

For the hangingwall, it is more difficult to judge which set of strength parameters is most representative. Since no failures have occurred, the back-analyzed strengths only represent a lower bound for the actual rock mass strength. This lower bound estimate does, however, compare well with strength estimates for the category *disturbed rock mass*. The values obtained in the report by Golder Associates (UK) (1996a) were significantly higher. Also, the fact that different stress ranges were used for different rock types in the regression analysis makes their approach less consistent.

To summarize, it appears that by assuming *disturbed rock mass*, we will obtain strength values that can be viewed as "average" strengths for an entire pit slope. These are probably conservative and representative of the residual strength of the rock mass. Nevertheless, they can be used in limit equilibrium methods for assessing the factor of safety once failure has developed fully. This is also an implicit assumption of all limit equilibrium methods, which results in conservative values on the factor of safety for brittle materials. For a progressive failure mechanism in a brittle rock mass, the peak rock mass strength could perhaps be estimated assuming *undisturbed rock mass* and then applied in a numerical model.

5 CONCLUSIONS AND RECOMMENDATIONS

Stability analysis of rock mechanics construction elements such as pit slopes, require knowledge of the strength parameters for the rock mass. The Hoek-Brown criterion is one of the very few criteria available that can be used for assessing the rock mass strength. In this report it has been shown that the use of the Hoek-Brown failure criterion to estimate strength parameters is lined with difficulties and uncertainties. Nevertheless, this approach is probably the best one available and rather than developing new failure criteria, efforts should be made to improve the Hoek-Brown criterion and the existing relations for parameter estimation.

The use of *undisturbed* versus *disturbed rock mass* in estimating the parameters m and s in the Hoek-Brown failure criterion is not well documented, and the supporting data are lacking. At the same time, for medium to poor quality rock masses (typically $RMR < 60$), the resulting strength values are extremely sensitive to the choice of *undisturbed* or *disturbed rock mass*. In this report, an alternative interpretation of this concept was presented, linking *undisturbed rock mass* with the peak rock mass strength, and *disturbed rock mass* with the residual strength. This appears reasonable from a mechanical perspective, and the data from the Aznalcollar slope failures provide some evidence supporting this hypothesis. However, more work is required to verify this and for formulating guidelines for parameter estimation.

In using the criterion and the proposed methods for estimating parameters it is important to adhere to a few basic rules, as follows:

- i. For rock slopes, use the category *disturbed rock mass* for initial estimates of the rock mass strength. This might be slightly over-conservative but at least for the case of the Aznalcollar pit slope, proved to give more reasonable strength values than when assuming *undisturbed rock mass*. These values are directly applicable for stability analysis of residual strength conditions of slopes.
- ii. Determine equivalent cohesion and friction angles using the same stress state or stress range for all rock types around the excavation. The stress range should be estimated from simple linear-elastic stress analysis of the slope in question. A representative stress range is typically found by examining the stresses in the interior of the slope, excluding the often very high toe stresses.

- iii. Preferably, use a linear regression to estimate c and ϕ over a larger, and more representative, stress range, rather than at a single value of the minor principal or normal stress.
- iv. Use the original Hoek-Brown criterion rather than the modified criterion, even for poor quality rocks. The modified criterion predicts zero uniaxial compressive strength for the rock mass, which appears to be physically incorrect. Aside from this, the strength differences between the two formulations are relatively small, which makes the original criterion equally applicable.
- v. Whenever possible, compare obtained values with back-calculated strength values before applying them in design. This is necessary as there are still much too few published cases supporting the Hoek-Brown failure criterion and in particular, the proposed relations for estimating the parameters m and s .
- vi. Forward prediction of large scale rock mass strength in cases where no failures have occurred is still difficult and uncertain. To improve on this, it is necessary to publish data on estimated and back-calculated strengths for verification and validation of the Hoek-Brown failure criterion.

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APPENDIX 1

Tables for Estimation of Parameters in the Hoek-Brown Failure Criterion

Table A1.1 *The original Hoek-Brown failure criterion (Hoek and Brown, 1980). Approximate equations for principal stress relations and Mohr envelopes for intact rock and jointed rock masses. Note that principal stresses have been normalized with respect to the uniaxial compressive strength of the intact rock.*

TABLE 12 - APPROXIMATE EQUATIONS FOR PRINCIPAL STRESS RELATIONSHIPS AND MOHR ENVELOPES FOR INTACT ROCK AND JOINTED ROCK MASSES					
	CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE <i>dolomite, limestone and marble</i>	LITHIFIED ARGILLACEOUS ROCKS <i>mudstone, siltstone, shale and slate (normal to cleavage)</i>	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE <i>sandstone and quartzite</i>	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS <i>andesite, dolerite, diabase and rhyolite</i>	COARSE GRAINED POLYMINERALLIC IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS <i>amphibolite, gabbro, gneiss, granite, norite and quartz-diorite</i>
INTACT ROCK SAMPLES <i>Laboratory size rock specimens free from structural defects</i> <i>CSIR rating 100+, NGI rating 500</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{7\sigma_{3n} + 1.0}$ $\tau_n = 0.816(\sigma_n + 0.140)^{0.658}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{10\sigma_{3n} + 1.0}$ $\tau_n = 0.918(\sigma_n + 0.099)^{0.677}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{15\sigma_{3n} + 1.0}$ $\tau_n = 1.044(\sigma_n + 0.067)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{17\sigma_{3n} + 1.0}$ $\tau_n = 1.086(\sigma_n + 0.059)^{0.696}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{25\sigma_{3n} + 1.0}$ $\tau_n = 1.220(\sigma_n + 0.040)^{0.705}$
VERY GOOD QUALITY ROCK MASS <i>Tightly interlocking undisturbed rock with unweathered joints spaced at ± 3 metres</i> <i>CSIR rating 85, NGI rating 100</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{3.5\sigma_{3n} + 0.1}$ $\tau_n = 0.651(\sigma_n + 0.028)^{0.679}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{5\sigma_{3n} + 0.1}$ $\tau_n = 0.739(\sigma_n + 0.020)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{7.5\sigma_{3n} + 0.1}$ $\tau_n = 0.848(\sigma_n + 0.013)^{0.702}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{8.5\sigma_{3n} + 0.1}$ $\tau_n = 0.883(\sigma_n + 0.012)^{0.705}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{12.5\sigma_{3n} + 0.1}$ $\tau_n = 0.998(\sigma_n + 0.008)^{0.712}$
GOOD QUALITY ROCK MASS <i>Fresh to slightly weathered rock, slightly disturbed with joints spaced at 1 to 3 metres.</i> <i>CSIR rating 65, NGI rating 10</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.7\sigma_{3n} + 0.004}$ $\tau_n = 0.369(\sigma_n + 0.006)^{0.669}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.0\sigma_{3n} + 0.004}$ $\tau_n = 0.427(\sigma_n + 0.004)^{0.683}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.5\sigma_{3n} + 0.004}$ $\tau_n = 0.501(\sigma_n + 0.003)^{0.695}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.7\sigma_{3n} + 0.004}$ $\tau_n = 0.525(\sigma_n + 0.002)^{0.698}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{2.5\sigma_{3n} + 0.004}$ $\tau_n = 0.603(\sigma_n + 0.002)^{0.707}$
FAIR QUALITY ROCK MASS <i>Several sets of moderately weathered joints spaced at 0.3 to 1 metre.</i> <i>CSIR rating 44, NGI rating 1.0</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.14\sigma_{3n} + 0.0001}$ $\tau_n = 0.198(\sigma_n + 0.0007)^{0.662}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.20\sigma_{3n} + 0.0001}$ $\tau_n = 0.234(\sigma_n + 0.0005)^{0.675}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.30\sigma_{3n} + 0.0001}$ $\tau_n = 0.280(\sigma_n + 0.0003)^{0.688}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.34\sigma_{3n} + 0.0001}$ $\tau_n = 0.295(\sigma_n + 0.0003)^{0.691}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.50\sigma_{3n} + 0.0001}$ $\tau_n = 0.346(\sigma_n + 0.0002)^{0.700}$
POOR QUALITY ROCK MASS <i>Numerous weathered joints spaced at 30 to 500mm with some gouge filling / clean waste rock</i> <i>CSIR rating 23, NGI rating 0.1</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.04\sigma_{3n} + 0.00001}$ $\tau_n = 0.115(\sigma_n + 0.0002)^{0.646}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.05\sigma_{3n} + 0.00001}$ $\tau_n = 0.129(\sigma_n + 0.0002)^{0.655}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.08\sigma_{3n} + 0.00001}$ $\tau_n = 0.162(\sigma_n + 0.0001)^{0.672}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.09\sigma_{3n} + 0.00001}$ $\tau_n = 0.172(\sigma_n + 0.0001)^{0.676}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.13\sigma_{3n} + 0.00001}$ $\tau_n = 0.203(\sigma_n + 0.0001)^{0.686}$
VERY POOR QUALITY ROCK MASS <i>Numerous heavily weathered joints spaced less than 50mm with gouge filling / waste rock with fines</i> <i>CSIR rating 3, NGI rating 0.01</i>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.007\sigma_{3n} + 0}$ $\tau_n = 0.042(\sigma_n)^{0.534}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.010\sigma_{3n} + 0}$ $\tau_n = 0.050(\sigma_n)^{0.539}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.015\sigma_{3n} + 0}$ $\tau_n = 0.061(\sigma_n)^{0.546}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.017\sigma_{3n} + 0}$ $\tau_n = 0.065(\sigma_n)^{0.548}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.025\sigma_{3n} + 0}$ $\tau_n = 0.078(\sigma_n)^{0.556}$

Table A1.2 *The updated Hoek-Brown failure criterion (Hoek and Brown, 1988).*
 Approximate relation between rock mass quality and material constants. From
 Hoek and Brown (1988).

		Table 1 : Approximate relationship between rock mass quality and material constants					
		Disturbed rock mass <i>m</i> and <i>s</i> values		undisturbed rock mass <i>m</i> and <i>s</i> values			
EMPIRICAL FAILURE CRITERION $\sigma'_1 = \sigma'_3 + \sqrt{m\sigma_c\sigma'_3 + s\sigma_c^2}$ $\sigma'_1 = \text{major principal effective stress}$ $\sigma'_3 = \text{minor principal effective stress}$ $\sigma_c = \text{uniaxial compressive strength of intact rock, and}$ $m \text{ and } s \text{ are empirical constants.}$			CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE <i>dolomite, limestone and marble</i>	LITHIFIED ARGILLACEOUS ROCKS <i>mudstone, siltstone, shale and slate (normal to cleavage)</i>	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE <i>sandstone and quartzite</i>	FINE GRAINED POLYMINERALIC IGNEOUS CRYSTALLINE ROCKS <i>andesite, dolerite, diabase and rhyolite</i>	COARSE GRAINED POLYMINERALIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS – <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>
INTACT ROCK SAMPLES							
<i>Laboratory size specimens free from discontinuities</i>	<i>m</i>	7.00	10.00	15.00	17.00	25.00	
	<i>s</i>	1.00	1.00	1.00	1.00	1.00	
CSIR rating: RMR = 100	<i>m</i>	7.00	10.00	15.00	17.00	25.00	
NGI rating: Q = 500	<i>s</i>	1.00	1.00	1.00	1.00	1.00	
VERY GOOD QUALITY ROCK MASS							
<i>Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.</i>	<i>m</i>	2.40	3.43	5.14	5.82	8.56	
	<i>s</i>	0.082	0.082	0.082	0.082	0.082	
CSIR rating: RMR = 85	<i>m</i>	4.10	5.85	8.78	9.95	14.63	
NGI rating: Q = 100	<i>s</i>	0.189	0.189	0.189	0.189	0.189	
GOOD QUALITY ROCK MASS							
<i>Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.</i>	<i>m</i>	0.575	0.821	1.231	1.395	2.052	
	<i>s</i>	0.00293	0.00293	0.00293	0.00293	0.00293	
CSIR rating: RMR = 65	<i>m</i>	2.006	2.865	4.298	4.871	7.163	
NGI rating: Q = 10	<i>s</i>	0.0205	0.0205	0.0205	0.0205	0.0205	
FAIR QUALITY ROCK MASS							
<i>Several sets of moderately weathered joints spaced at 0.3 to 1m.</i>	<i>m</i>	0.128	0.183	0.275	0.311	0.458	
	<i>s</i>	0.00009	0.00009	0.00009	0.00009	0.00009	
CSIR rating: RMR = 44	<i>m</i>	0.947	1.353	2.030	2.301	3.383	
NGI rating: Q = 1	<i>s</i>	0.00198	0.00198	0.00198	0.00198	0.00198	
POOR QUALITY ROCK MASS							
<i>Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock</i>	<i>m</i>	0.029	0.041	0.061	0.069	0.102	
	<i>s</i>	0.000003	0.000003	0.000003	0.000003	0.000003	
CSIR rating: RMR = 23	<i>m</i>	0.447	0.639	0.959	1.087	1.598	
NGI rating: Q = 0.1	<i>s</i>	0.00019	0.00019	0.00019	0.00019	0.00019	
VERY POOR QUALITY ROCK MASS							
<i>Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.</i>	<i>m</i>	0.007	0.010	0.015	0.017	0.025	
	<i>s</i>	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001	
CSIR rating: RMR = 3	<i>m</i>	0.219	0.313	0.469	0.532	0.782	
NGI rating: Q = 0.01	<i>s</i>	0.00002	0.00002	0.00002	0.00002	0.00002	

Table A1.3 *The modified Hoek-Brown failure criterion (Hoek, Wood and Shah, 1992).*
 Estimation of m_b/m_i and a based on rock structure and surface condition.
 From Hoek, Wood and Shah (1992).

MODIFIED HOEK-BROWN FAILURE CRITERION													
$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} \right)^a$ <p> σ'_1 = major principal effective stress at failure σ'_3 = minor principal effective stress at failure σ_c = uniaxial compressive strength of <i>intact</i> pieces in the rock mass m_b and a are constants which depend on the composition, structure and surface conditions of the rock mass </p>		SURFACE CONDITION		VERY GOOD Unweathered, discontinuous, very tight aperture, very rough surface, no infilling		GOOD Slightly weathered, continuous, tight aperture, rough surface, iron staining to no infilling		FAIR Moderately weathered, continuous, extremely narrow, smooth surfaces, hard infilling		POOR Highly weathered, continuous, very narrow, polished/slickensided surfaces, hard infilling		VERY POOR Highly weathered, continuous, narrow, polished/slickensided surfaces, soft infilling	
STRUCTURE													
	BLOCKY - well interlocked, undisturbed rock mass; large to very block size	m_b/m_i	0.7	0.5	0.3	0.35	0.4	0.1	0.45				
	VERY BLOCKY - interlocked, partially disturbed rock mass; medium block sizes	m_b/m_i	0.3	0.2	0.4	0.45	0.5	0.04	0.5				
	BLOCKY/SEAMY - folded and faulted, many intersecting joints; small blocks	m_b/m_i		0.08		0.5	0.5	0.01	0.55	0.004	0.6		
	CRUSHED - poorly interlocked, highly broken rock mass; very small blocks	m_b/m_i		0.03		0.5	0.55	0.003	0.6	0.001	0.65		

Table A1.4 *The general Hoek-Brown failure criterion (Hoek, Kaiser and Bawden, 1995).*
 Values of the constant m_i for intact rock, by rock group. From Hoek, Kaiser
 and Bawden (1995).

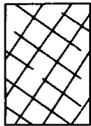
Table 8.3: Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates.

Rock type	Class	Group	Texture			
			Course	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
			← Greywacke (18) →			
	Non-Clastic	Organic	← Chalk 7 →			
			← Coal (8-21) →			
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
	Chemical		Gypstone 16	Anhydrite 13		
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists (10)	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
	Dark		Diorite (28)		Andesite 19	
			Gabbro 27	Dolerite (19)	Basalt (17)	
	Norite 22					
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

*These values are for intact rock specimens tested normal to foliation. The value of m_i will be significantly different if failure occurs along a foliation plane (Hoek, 1983).

Table A1.5 *The generalized Hoek-Brown failure criterion (Hoek, Kaiser and Bawden, 1995). Estimation of constants m_b/m_i , s , a , deformation modulus E and Poisson's ratio ν for the Generalized Hoek-Brown failure criterion. Values are for undisturbed rock. From Hoek, Kaiser and Bawden (1995).*

Table 8.4: Estimation of constants m_b/m_i , s , a , deformation modulus E and the Poisson's ratio ν for the Generalised Hoek-Brown failure criterion based upon rock mass structure and discontinuity surface conditions. Note that the values given in this table are for an *undisturbed* rock mass.

GENERALISED HOEK-BROWN CRITERION		SURFACE CONDITION	VERY GOOD Very rough, unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered or altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings containing angular rock fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a$ <p> σ_1' = major principal effective stress at failure σ_3' = minor principal effective stress at failure σ_c = uniaxial compressive strength of <i>intact</i> pieces of rock m_b, s and a are constants which depend on the composition, structure and surface conditions of the rock mass </p>							
STRUCTURE							
	BLOCKY -very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets	m_b/m_i s a E_m ν GSI	0.60 0.190 0.5 75,000 0.2 85	0.40 0.062 0.5 40,000 0.2 75	0.26 0.015 0.5 20,000 0.25 62	0.16 0.003 0.5 9,000 0.25 48	0.08 0.0004 0.5 3,000 0.25 34
	VERY BLOCKY-interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets	m_b/m_i s a E_m ν GSI	0.40 0.062 0.5 40,000 0.2 75	0.29 0.021 0.5 24,000 0.25 65	0.16 0.003 0.5 9,000 0.25 48	0.11 0.001 0.5 5,000 0.25 38	0.07 0 0.53 2,500 0.3 25
	BLOCKY/SEAMY-folded and faulted with many intersecting discontinuities forming angular blocks	m_b/m_i s a E_m ν GSI	0.24 0.012 0.5 18,000 0.25 60	0.17 0.004 0.5 10,000 0.25 50	0.12 0.001 0.5 6,000 0.25 40	0.08 0 0.5 3,000 0.3 30	0.06 0 0.55 2,000 0.3 20
	CRUSHED-poorly interlocked, heavily broken rock mass with a mixture of angular and rounded blocks	m_b/m_i s a E_m ν GSI	0.17 0.004 0.5 10,000 0.25 50	0.12 0.001 0.5 6,000 0.25 40	0.08 0 0.5 3,000 0.3 30	0.06 0 0.55 2,000 0.3 20	0.04 0 0.60 1,000 0.3 10

Note 1: The in situ deformation modulus E_m is calculated from Equation 4.7 (page 47, Chapter 4). Units of E_m are MPa.