# The Determination of Rock Mass Strength for Engineering Design

**A. G. Meyers**
B.E.(Mining), Ph.D., M.I.E.Aust., M.Aus.I.M.M
Research Fellow, University of South Australia, Australia
**W. S. Kaggwa**
B.E., PhD., M.I.E.Aust.
Senior Lecturer, University of Adelaide, Australia
**S. D. Priest**
B.Sc.Hons., Ph.D., C.Eng. M.I.C.E., F.I.E.Aust., F.Aus.I.M.M.
Head, Mining Engineering, University of South Australia, Australia

**Summary** The range of techniques for estimating the yield strength of discontinuous rock masses include analytical, numerical and empirical methods and laboratory and field testing. No one technique is definitive. This paper summarises the results from a laboratory based and discontinuum modelling study conducted on discontinuous material with the aim of investigating these techniques From this study, a number of modifications to Bieniawski's RMR system are suggested. These modifications enable the system to provide input data to the Hoek-Brown rock mass yield criterion that better reflect those rock mass properties that most influence rock mass behaviour.

## 1. INTRODUCTION

Man made materials generally display easily defined, homogeneous and isotropic properties and well-understood yield mechanisms. Discontinuous rock masses display the antithesis of these characteristics in that they are generally heterogeneous, anisotropic and have unpredictable spatial variability in the engineering properties of both the intact material and the discontinuities. While each of these factors influences the strength of the rock mass, no definitive technique has yet been developed to quantify the influence of even one of these factors on the strength of the rock mass under an imposed stress regime. The range of techniques used to date including analytical, numerical and empirical methods and laboratory and field testing.

### 1.1 Analytical and numerical techniques

Analytical techniques (Amadei, 1988) are restricted to fairly simple domains having a small number of discontinuity sets. Discontinuum type numerical methods (Cundall, 1983) have essentially eliminated these restrictions by incorporating the analytical theory into a computational framework thereby allowing large amounts of rock mass structural and material data to be incorporated into the analysis. The major limitation with most numerical methods is that their implementation in commercially available software is restricted to two-dimensional domains. To date, there has been a lack of validating evidence confirming the effectiveness of two-dimensional models in predicting three-dimensional behaviour.

Casting three-dimensional problems in two dimensions requires that solutions be obtained under plane stress, plane strain or quasi three-dimensional stress conditions. It can be shown that triaxial stress conditions exist at most points within a rock mass; two-dimensional assumptions can, therefore, lead to unrealistic predictions. A second, and more significant difficulty, is that most rock masses contain a complex three-dimensional network of fractures, faults, bedding planes and other mechanical discontinuities that render the predictions of two-dimensional models of questionable validity. For example, double plane wedge failure, which is one of the most commonly observed block failure mechanisms, cannot be modelled in two dimensions. While three-dimensional software is available, the number of blocks that can be modelled is often restricted to an unrealistic number by the amount of hardware memory available. Depending on the CPU speed of the processor, the time required to implement a large two- or three-dimensional analysis can be excessive for day to day or first assessment design purposes.

### 1.2 Laboratory and field testing

Laboratory studies conducted under controlled conditions on small diameter cores of intact rock are generally of little help in predicting the large scale behaviour of a discontinuous rock mass, particularly those rock masses under low stress conditions. On the other hand, tests conducted on large diameter cores of discontinuous rock (Jaeger, 1970, McLamore and Gray, 1967) can provide useful insight into the strength and deformation characteristics of the rock material. It is, however, exceptionally difficult and expensive to obtain undisturbed samples of a rock mass particularly when the rock is either very weak or very strong or when samples must be taken at significant depths. For the cores to be representative, triple tube coring barrels must be used by experienced drillers and extreme care must be taken during core logging and pre-test preparation.

A further complexity arises as a result of the spatial variability in such rock mass characteristics as the spacing, size, persistence and orientation of the discontinuities and the shear strength of these features and the intact material. This variability means that large volumes of material must be tested and large numbers of tests be carried out if statistically significant results are to be obtained. However, the volume of material that can be tested in a laboratory is limited by the size of the available equipment.

As the material and discontinuity characteristics cannot be accurately quantified prior to testing (accurate quantification requiring a specimen to be pulled apart) it is difficult to extrapolate the test results to in situ dimensions. For example, studies have shown that shear stiffness, strength and deformation moduli all decrease with increases in the size of the specimens tested (Cunha, 1990). In addition, without an accurate pre-test characterisation, sensitivity studies aimed at determining how each of the rock mass characteristics influences the overall strength and behaviour of a rock mass are difficult to interpret.

Well planned, in situ testing, carried out pre-excavation, can provide valuable data on the engineering properties of a rock mass, however, the cost of carrying out these tests generally prohibits their use for any but the largest projects. While these tests can impose a set of conditions over a much larger volume of the rock mass than can physically be tested by any other method, it is still impossible to impose these conditions on a scale anywhere near that imposed by even the smallest excavation. A further limitation with in situ testing is that it can be difficult to carry out sensitivity studies on the rock mass as each test will generally alter the properties of the zone over which it was carried out. While this problem can be partially addressed by carrying out a number of tests on different zones within the same rock mass, the prohibitive cost of multiple tests can make this option undesirable. In addition, the heterogeneous nature of most rock masses tends to make it difficult to interpret the results of multiple tests in terms of the performance of the overall rock mass.

The limitations of in situ, pre-excavation testing has meant that engineers have often chosen to monitor the performance of an excavation post excavation rather than physically imposing a set of conditions on it at a pre-excavation stage. By developing a pre-excavation, predictive model using, for example, a numerical model and comparing the predicted performance with the actual post excavation performance, it is possible to refine the predictive constitutive model.

### 1.3 Empirical methods

The use of a rock mass rating systems, such as the RMR system developed by Bieniawski (1989), has gained widespread acceptance for both Civil and Mining Engineering applications. When combined with a strength criterion, the system becomes a practical tool for estimating the yield strength of a discontinuous rock mass. Hoek and Brown's (1988) original rock mass yield criterion is one of the most advanced empirically derived strength criteria developed to date. Limitation in the effectiveness of the criterion for predicting the strength of very poor rock masses led to the development of an updated version of the criterion, referred to as the generalised Hoek Brown criterion, (Hoek et al., 1995). Both the original and the generalised criterion use as input, a RMR number derived from Bieniawski's rating system. While the use of the criterion for predicting the yield characteristics of a rock mass is well accepted, there are several deficiencies in it. These deficiencies relate more to the use of the RMR system used as input than to the yield criterion itself. Several deficiencies in the system discussed by Meyers and Priest (1991) are as follows:

- In order to obtain a rating adjustment for discontinuity orientation, the discontinuities are grouped into one of 3 orientation groups, $0^o \rightarrow 20^o$, $20^o \rightarrow 45^o$ and $45^o \rightarrow 90^o$. These groupings represent an over-simplification of the influence of orientation on rock mass strength.
- The 'orientation rating adjustment' is determined on the basis of the mean orientation of each discontinuity set. No allowance is made for any influence on rock mass strength arising due to variability in the orientation of the individual discontinuities within each set.
- Discontinuity frequency is rated twice in the RMR system, since both the Rock Quality Designation (RQD) and the spacing of the discontinuities are allocated individual ratings. The relationship between RQD and discontinuity frequency has been studied by Priest and Hudson (1976). In the case of underground mines, tunnels and footings, the RMR system attaches a higher priority to the influence of discontinuity frequency on the yield strength of a rock mass than it does to discontinuity orientation. This practice is not consistent with actual rock mass behaviour (Einstein and Hirschfeld, 1973).

- The use of the RQD parameter as an indicator of discontinuity frequency is of questionable merit because RQD is influenced by the shear strength of the intact rock material being drilled, the drilling practice and the expertise of the core logger in differentiating between natural fractures and those caused by blasting or drilling. Moreover, the conventional RQD is relatively insensitive to changes in discontinuity frequency below about $4m^{-1}$. (Priest and Hudson, 1976).
- The behaviour of rock masses under low stress conditions, such as surface outcrops, is influenced more by discontinuity shear strength than by the compressive strength of the intact material. The shear strength of the discontinuities is, however, only considered incidentally in the RMR system through its relation with discontinuity roughness which is assessed subjectively.

The deficiencies in the RMR system carry through to the Hoek-Brown yield criterion procedure and, in many cases, generate a lower bound to rock strength that can lead to conservatism in design (Hoek, 1983; Meyers and Priest, 1991). While conservatism can be inefficient and expensive at the construction phase, of greater concern is that the insensitivity of the procedure to those discontinuity properties that most influence rock mass behaviour can lead to unpredictable and, in some cases, dangerously non-conservative results. The limited studies conducted to date aimed at validating the effectiveness of the criterion have meant that it is difficult to know under what conditions these concerning results could occur or how they can be allowed for at the input stage.

### 1.4 Development of research project

The above discussion highlights several issues regarding the techniques used for predicting the mechanical characteristics of a rock mass requiring further investigation. Three of these issues are:

- the lack of evidence validating the effectiveness of two-dimensional discontinuum methods in predicting three-dimensional behaviour,
- the difficulties in carrying out sensitivity studies in a laboratory on large diameter cores of natural discontinuous rock for the purposes of defining those characteristics that most influence rock mass behaviour, and
- the need for studies to be conducted validating the effectiveness of the Hoek-Brown yield criterion for predicting the yield strength of rock material.

Meyers and Priest (1992a) discussed studies that began in 1988 at the University of Adelaide aimed at investigating these issues. These studies continue to date at the University of South Australia. The early phase of these studies involved laboratory based research into the behaviour of models of rock-like material intersected by randomly orientated discontinuities. The results of this research were used to validate both a two-dimensional discontinuum method of numerical modelling and the Hoek-Brown yield criterion. A technique was suggested for modifying the RMR procedure so that the input it provided to the yield criterion better reflected the characteristics of the rock mass. This technique is currently being validated using the results obtained from a detailed laboratory study on large diameter cores of natural discontinuous rock material. A summary of this research and some of the conclusions reached from it are briefly discussed below. More detailed discussion will be presented in forthcoming papers.

## 2. MODEL STUDIES

For many years researchers have studied the performance of discontinuous models in an attempt to define those factors most influencing rock mass behaviour. These studies have tended to concentrate on models comprised of geometrically similar, parallelepipedal blocks often subjected to uniaxial stress conditions (Brown, 1970; Reik and Zacas, 1978). However, discontinuities in a rock mass are the product of the tectonic history to which the rock mass has been subjected. Each tectonic occurrence creates a fingerprint of discontinuities that appears as an intricate network of discontinuity sets. Research on blocky models has, therefore, tended to be an over simplification of reality. For the current research, this simplification was avoided and the behaviour of specimens comprised of geometrically distinct blocks, subjected to triaxial test conditions, was studied.

For this purpose, four, 150mm diameter x 300mm long, cylindrical specimen geometries were fabricated using an original technique discussed by Meyers and Priest (1992b). Considerable time was devoted in this study to developing equipment that could be used to conduct reliable and well controlled tests. This equipment included a 150mm diameter Hoek cell (Hoek and Franklin, 1968) and a linear displacement pump described by Meyers and Priest (1992c). The main conclusions arising from the laboratory studies are discussed in Meyers and Priest (1991, 1994).

## 3. NUMERICAL MODELLING

The two-dimensional discontinuum block modelling code UDEC was adopted to simulate the laboratory tests with the aim of assessing its ability to duplicate the observed specimen behaviour. If UDEC was found to be effective in modelling the block displacements and yield characteristics for those discontinuity geometries studied and stress environments applied, its use could have been extended to model a range of specimens having geometries more complex than those investigated. Meyers and Priest (1991) drew the following conclusions from this study:

- Although UDEC can provide reasonable predictions of general failure mechanisms, in some cases predictions reflected the calculation sequence rather than the physical processes.
- The code is able to model the elasto-plastic stress versus deformation behaviour of intact and anisotropic specimens.
- The code is unable to predict correctly the yield strength of non-symmetrical three-dimensional domains. However, UDEC may model reasonably the deformation mechanism if the mechanism involves only relative block movement and not fracturing of intact blocks.
- The code cannot generate cracks in previously intact blocks during the calculation cycle so care must be taken to identify the development of shear bands.
- As with all rock mass models, UDEC is limited by the need to provide representative input data on intact material and discontinuity properties.

Due to the concerns in regard to the effectiveness of the two-dimensional code to model effectively the three-dimensional conditions, the decision was made to concentrate on the laboratory rather than the numerical aspects of the study for the next phase of the research. However, before this phase could begin, the findings from the previous phase had to be incorporated into theory.

## 4. DISCONTINUITY WEIGHTING THEORY

The laboratory and numerical studies highlighted the fact that the strength of a rock mass subjected to non-hydrostatic load conditions is highly dependent on:

- the orientation of each of the discontinuities with respect to that of the major principal stress and
- the frictional characteristics of the discontinuities.

This result led to the conclusion that a weighting could be applied to each discontinuity intersecting an excavation by quantifying it according to these criteria. This conclusion led to the development of the discontinuity weighting theory and the development of a procedure for determining a weighting appropriate to a rock mass within which an underground excavation is planned. This theory is discussed in detail by Meyers and Priest (1994) and will be developed further in forthcoming papers. A brief summary of the general principles is given below.

A scanline mapping technique (Priest, 1992) is used to determine input data defining the geometric characteristics of the discontinuities in the vicinity of the excavation. The scanline results are processed using suitable techniques, such as those implemented in the software SCANMASTER as described by Meyers et al. (1993). These techniques determine values for the mean orientations of the discontinuity sets, their mean true spacing and size. Additional data are also required defining the in-situ major principal stress direction, the orientation and dimensions of the proposed excavation and the friction angle of the discontinuities.

It should be noted that the weighting theory does not assume that the discontinuities intersecting the excavation are the same ones that were sampled in the scanline surveys. In fact, it is quite likely that the surveys were conducted on cross sections through the rock mass quite remote from the volume of rock surrounding the excavation. What is assumed, however, is that the statistical properties defining the spatial characteristics of the sampled discontinuities are the same in the region of the excavation as they are in the area sampled. For this assumption to be correct, consideration must be given to how any large scale structural features such as folding and faulting may influence the structure of the rock mass in the vicinity of the excavation.

A statistically based procedure is used to allocate a weighting $N_{e(i)}'$ to each discontinuity sampled. This weighting is an estimate of how many discontinuities having an orientation the same as that sampled in the scanline surveys are expected to intersect the excavation. This estimate is given by:

$$N_{e(i)}' = \frac{N_{e(i)}\, N_{(k)total}}{N_{total}} \qquad (1)$$

where:

- $N_{e(i)}$ is the non-normalised form of $N_{e(i)}'$. $N_{e(i)}$ is determined on the basis of the distance along the scanline at which the discontinuity intersects, the orientation of the discontinuity and the mean spacing of the set in which the discontinuity occurs.
- $N_{(k)total}$ is the total number of discontinuities that will intersect the excavation based on the mean orientation of the discontinuity sets.
- $N_{total}$ is the total number of discontinuities that will intersect the excavation based on the orientation of each discontinuity sampled in the scanline survey.

The propensity for each of the $N_{total}$ discontinuities to mobilise is influenced by its orientation with respect to the direction of the major principal stress.

This propensity is quantified in terms of a parameter, referred to as the orientation weighting coefficient $w_\theta$. This coefficient ranges between 0 and 1 such that a discontinuity having a rating close to 1 has a high potential to mobilise whereas a weighting close to 0 indicates a potentially stable discontinuity. A value of the coefficient is determined for each of the sampled discontinuities using the expression:

$$w_\theta = (cos\ 2\Phi)^n \qquad (2)$$

where,

- $\Phi = (\beta_w - \beta_{crit})$. Note: If $\beta_w - \beta_{crit} < -45^o$ the discontinuity is assigned a value of $w_\theta$=0.
- $\beta_w$ is the acute angle between the direction of major principal stress ($\alpha_\sigma$, $\beta_\sigma$) and the orientation of the normal to each discontinuity ($\alpha_n$, $\beta_n$)
- $\beta_{crit} = \phi_w/2 + 45^o$ (degrees) (3)
- $n = 0.175\ \phi_w - 0.250$ (4)
- $\phi_w$ is the friction angle of the discontinuity

After completing the above analysis, each discontinuity sampled has four parameters defining it. Two parameters $\alpha_n$ and $\beta_n$ defining the orientation of a unit vector normal to the discontinuity, the parameter $N_{e(i)}'$ and the orientation weighting coefficient $w_\theta$ quantifying the potential for the discontinuity to mobilise.

The distribution of orientation weightings for the rock mass in the region of the excavation can be graphically represented in the form of a histogram. An example of such a histogram is shown in Figure 1. The abscissa of the histogram, representing the orientation weightings, is divided into a number of equally spaced class intervals. The ordinate axis gives the respective number of discontinuities having a value for $w_\theta$ within each class interval. An ultimate orientation weighting for the rock mass $w_{\theta(mass)}$ is determined from this histogram. In the case shown, the mean frequency occurs around $w_\theta$ = 0.8 suggesting that the rock mass is highly unstable.

## 5. MODIFIED ROCK MASS RATING SYSTEM

The Hoek-Brown yield criterion was never intended to be used to predict the strength of laboratory sized specimens. However, the parameters that control the strength of an in-situ rock mass are broadly the same as those that control the strength of a discontinuous specimen. On the basis of this fact,

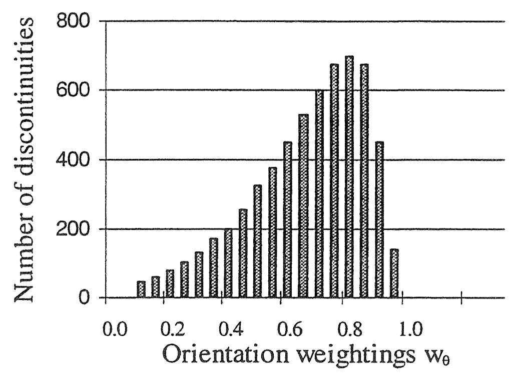


Figure 1. Example of an orientation weighting histogram.

the next stage of the research aimed to compare how well the yield criterion would predict the strength of each specimen.

Meyers and Priest (1991) compared the predicted failure envelopes produced for each specimen with those obtained from the laboratory tests and found that the yield criterion provided a lower bound to rock mass yield strength. Depending on the discontinuity orientation, this lower bound could be quite conservative. To correct the discrepancy between actual and predicted strength characteristics, two major modifications to the RMR system were proposed (Meyers and Priest, 1994). At this stage, these modifications only apply to those cases where the system is used to provide input data to the Hoek-Brown yield criterion for the purposes of estimating the strength of rock masses surrounding underground excavations. These modifications, incorporated into the Modified Rock Mass Rating (MRMR) system, do not modify all aspects of the RMR system but rather seek to address those aspects considered to be most subjective. In summary these modifications involve:

1. Determining the discontinuity orientation rating $RMR_{orient}$ for the rock mass based on the value for the orientation weighting $w_{\theta(mass)}$ using the expression:

   $$RMR_{(orient)} = 16.5\ cos(180\ w_{\theta(mass)}) + 13.5 \qquad (5)$$

   In the MRMR system, this rating ranges from -3 to 30 points. The basic RMR value adjusts the value for $RMR_{orient}$ in the same way that an adjustment is made in Bieniawski's RMR system to allow for the orientation of the discontinuities with respect to the excavation.

2. Determining a discontinuity spacing rating $RMR_{space}$ that quantifies the mean volume of

the blocks of rock surrounding the excavation in terms of that for the excavation.

$$RMR_{(space)} = 20\left(\frac{6\,L_e^{\,3}}{\pi \times N_{(k)total}^{\,3} \times A_e \times S_{\mu(max)}}\right) \quad (6)$$

where,

- $N_{(k)total}$ is the number of discontinuities expected to intersect the excavation.
- $L_e$ and $A_x$ are the length and cross sectional area of the excavation.
- $S_{\mu(max)}$ is the maximum spacing of all the discontinuity sets.

A maximum rating of 20 points is allocated to the rock mass for this parameter. This rating replaces the rating allocated for discontinuity spacing in the RMR system.

3. Ignore the rating for drill core quality, RQD.

The MRMR system has been applied successfully to a case study to confirm its useability. However, as the system was developed on the basis of a series of tests on models of discontinuous rock, it was considered prudent to validate it using specimens of natural discontinuous rock. With this aim, further laboratory studies on such material have now been completed to address this issue. Details of this study and the results obtained from it will be published in the near future.

## 5. CONCLUSION

To date no one technique can provide a definitive estimate of rock mass yield strength. However, on the basis of laboratory test data, the strength can reasonably be predicted using the Hoek-Brown criterion, based on input data provided by the MRMR system. Further studies are currently being conducted to confirm this finding.

## 6. ACKNOWLEDGMENTS

The laboratory work described in this paper was carried out in the Department of Civil Engineering, now the Department of Civil and Environmental Engineering, at the University of Adelaide during the period 1989 to 1993 as part of the first Author's doctoral research studies. The Author is indebted to his supervisors Professor Stephen Priest and Dr. William Kaggwa. In addition, he would like to thank Bruce Lucas, Stan Woithe, Bob Marcussan and Peter Cotton for their dedication and technical assistance.

## 7. REFERENCES

Amadei, B. (1988) Strength of a Regularly Jointed Rock Mass Under Biaxial and Axisymmetric Loading Conditions. *Int. J. of Rock Mech. Min. Sci. & Geomech. Abstr.* Vol.25, pp. 3-13.

Bieniawski, Z.T. (1989) *Engineering Rock Mass Classification: A Complete Manual for Engineers and Geologists in Mining, Civil and Petroleum Engineering*, John Wiley and Sons, 251p.

Brown, E.T. (1970) Strength of Models of Rock with Intermittent Joints. *J. Soil Mech. and Found. Div.* Am. Soc. of Civil Eng. Vol.96, SM6, pp. 1935-1949.

Cundall, P.A. (1983) Development of a Generalised 2-D and 3-D Distinct Element Program for Modelling of Jointed Rock. *U.S. Army Eng. Waterways Final Tech. Rpt.*

Cunha, A.P. (1990) Scale Effects in Rock Mechanics. *In Proc. Scale Effects in Rock Mech. Conf. da Cunha (ed.) Balkema, Rotterdam, pp. 3-27.*

Einstein, H.H. and Hirschfeld, R.C. (1973) Model Studies on Mechanics of Jointed Rock. *J. Soil Mech. and Found. Div.* Am. Soc. of Civil Eng. Vol.99, pp. 229-248.

Hoek, E. (1983) Strength of Jointed Rock Masses. *Gèotechnique*. Vol.33, No. 3, p.210.

Hoek, E. and Brown, E.T. (1988) The Hoek-Brown Failure Criterion - A 1988 Update. In *15th Canadian Rock Mechanics Symp.*, pp. 31-38.

Hoek, E, Kaiser, P.K. and Bawden, W.F. (1995) *Support of Underground Excavations in Hard Rock* Balkema, Rotterdam, 215p.

Hoek, E. and Franklin, J.A. (1968) Simple Triaxial Cell For Field or Laboratory Testing. *Trans. Inst. Min. Metal.* Vol.77, pp. A22-26.

Jaeger, J.C. (1970) Behaviour of Closely Jointed Rock. *Proc. 11th Symp. Rock Mech., Berkeley, Calif.* AIME, New York, pp. 57-68.

McLamore, R. and Gray, K.E. (1967) The Mech. Behaviour of Anisotropic Sedimentary Rocks. *J. Engng for Industry. Trans. Am. Soc. of Mech. Eng., Ser. B* Vol.89, pp. 62-73.

Meyers A.G. and Priest S.D. (1991) *Exp. Studies of the Influence of Fractures on Rock Deformability and strength* Final report for ARC project No. A48715339, 293p.

Meyers A.G. and Priest S.D. (1992a) The Prediction of Rock Mass Strength and Deformability for Eng. Design *Proc. WA Conf. on Mining Geomechanics,* Kalg., pp. 335-348.

Meyers A.G. and Priest S.D. (1992b) A Technique for Moulding Cylindrical Discontinuous Models, *Rock Mechanics*, Vol. 25, No. 3, pp. 207-214.

Meyers A.G. and Priest S.D. (1992c) A Micro-processor Controlled Pump for Triaxial Cell Pressure Control, *Int. J. of Rock Mech. Min. Sci. & Geomech. Abstr.* Vol.29, No.2, pp.187-191.

Meyers A.G. and Priest S.D. (1994) *The Prediction of Rock Mass Yield Strength and Deformability for Engineering Design.* Final report for ARC project No. A489031573, 102p.

Meyers, A.G. Priest, S.D. and Walker, D. (1993) A Technique for Preparing Rock Mass Discontinuity Geometry Files for Slope Stability Monitoring Design. Proc. *WA Conf. on Geotech. Instrum. and Monit.* Kalg. Balkema, London, pp. 365-374.

Priest, S.D. (1992) *Discontinuity Analysis for Rock Engineering.* Chapman and Hall, London 473p.

Priest, S.D. and Hudson, J.A. (1976) Discontinuity Spacing in Rock. *Int. J. of Rock Mech. Min. Sci. & Geomech. Abstr.* Vol.13, pp. 135-148.

Reik, G. and Zacas, M. (1978) Strength and Deformation Characteristics of Jointed Media in True Triaxial Compression *Int. J. of Rock Mech. Min. Sci. & Geomech. Abstr.* Vol.15, pp. 295-303.