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A relationship between joint roughness and joint shear strength

Relation entre la rugosité des fissures et la résistance au cisaillement

Eine Relation zwischen Fugenrauheit und Fugenscherfestigkeit

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SUMMARY.

Tension fractures generated in different strengths of a weak, brittle model material are taken to represent joint surfaces of different dimensions. Direct shear tests performed on these surfaces suggest that, as far as peak shear strength is concerned, no appreciable strength-scale effect exists.

Analysis of the experimental results for a large number of shear tests on model joints reveals that a linear relationship exists between the peak dilation angle and the peak stress ratio. It is also found that a simple relationship exists between the peak dilation angle and the ratio of the normal stress to the compressive strength.

A simple method is developed for statistically analysing the roughness profiles recorded for joints of varying degrees of roughness. This involves the computation of inclination angles for asperities of different base lengths. It is found that these quantities are analogous to the change of peak dilation angles for different normal stresses.

The practical application of this shearing analogy to slope stability problems is summarised, and a typical example enumerated. Photogrammetric recording of the roughness of joints exposed on rock faces, and a statistical analysis of the data, provides an estimate of the peak shear strength for any range of normal stress.

1971

Rock Mech. Min. Sci. Vol. 9, pp. 579–602. Pergamon Press 1972. Printed in Great Britain

A MODEL STUDY OF ROCK-JOINT DEFORMATION

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(*Received* 13 *November* 1971)

Abstract—Existing techniques of rock-joint modelling are reviewed. It is concluded that no methods presently in use are acceptable as either realistic models of mating rock joints or as mass production methods for the development of large, highly jointed models of rock masses.

A method is described for producing mating tension fractures in a weak, brittle model material using a large guillotine device. Parallel sets of model joints can be produced which are continuous, cross-jointed or offset (stepped) depending upon the chronological order of fracturing. The direct shear properties of these three types are compared and evaluated. The model results are used as a basis for predicting the full-scale (1:500) displacements accompanying shear failure of a 96-ft long prototype tension joint.

Recent numerical modelling of jointed rock masses has been based on assumed values of the shear and normal stiffness of the joints. These components are found to dominate the elastic deformation properties of the intact rock. The results of shear and normal stiffness tests on the model joints are used for a careful assessment of these quantities. The shear stiffness (peak shear stress per unit tangential displacement) is found to be both normal stress and size dependent, and this is confirmed by a survey of shear-test data for joints in rock. There appears to be an inverse proportionality between test dimension and shear stiffness, for a given normal stress. The normal stiffness (normal stress per unit closure) is found to be dependent on the preconsolidation or virgin normal stress level.

The problems of simulating the behaviour of jointed rock masses by the finite-element method are reviewed. Two particular drawbacks seem to be the conservation of energy demanded during computation, and the computer storage problems involved in modelling dilatent joints. Both these features are of fundamental importance to rock-mass deformation. A move towards realistic physical modelling is considered essential to an understanding of real processes.

1972

PROGRESSIVE FAILURE OF EXCAVATED ROCK SLOPES.

by Nicholas Barton*

SUMMARY

The stability of a rock slope is largely controlled by the presence of discontinuities in the rock. Their presence means that failure is generally of a translational type, and is therefore amenable to simple methods of analysis. The most unstable situation is chosen; one of the joint sets dipping into the slope with a strike direction parallel to the slope face. This situation is amenable to a two dimensional approach.

A limit equilibrium method is used to analyse a simple plane failure. Three refinements are then incorporated; the division of the unstable rock mass into slices (representing an additional set of vertically dipping joints), the assumption of zero tensile strength across these slices, and analysis of the effect of excavation on the assumed self weight stress distribution acting on the joints exposed by the excavation. The stability or instability of different parts of the slope is characterised by forces acting parallel to the failure surface. The depth of failure can be calculated without recourse to computing methods.

The concept of pre-failure shear displacements and increased weathering of overstressed joints is introduced. This progressive failure mechanism leads to a possible stepped portion of the failure surface. The predicted multi-linear slide scar is characterised by a vertical scarp passing through the crest of the slope, a stepped portion on which the vertical joints open, with sliding on the inclined joints, and shear failure on the inclined joint passing through the toe.

The stepped portion is caused by progressive failure, and residual shear strength parameters are adopted in this region for design purposes. This is considered to be a more realistic solution than a global assumption of residual strength. The method is illustrated by worked examples, in which the progressive failure mode is shown to reduce the failure depth considerably. A further reduction in stability is caused by transient water pressures. The pessimistic assumption of a full tension crack, and steady seepage reducing to zero exit pressure at the toe is used as an illustration.

1971

A MODEL STUDY OF AIR TRANSPORT FROM UNDERGROUND OPENINGS SITUATED BELOW GROUND WATER LEVEL

Etude experimentale pour la circulation de l'air depuis une excavation souterraine située sous le niveau phréatique

Experimentelle Untersuchung über die Luftbewegung aus unterirdischen Hohlräumen unter dem Grundwasserspiegel

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SUMMARY

A parallel plate model (Hele-Shaw analogue) was used to study two phase fluid flow problems in jointed rock. The principal aim of the study was to determine the influence of groundwater on the flow of air from large underground openings. The information obtained was used to interpret the results of some borehole pumping tests that were performed in the field, and to predict the air leakage rates that might occur from large openings excavated in the same location. Drawdown tests were also performed to determine the head of water remaining above a large unlined opening, when the latter was located several diameters beneath the original groundwater level. Flow through uniformly jointed rock and through individual joints has been considered.

RESUME

Un modèle à plaques parallèles (analogue à celui de Hele Shaws) a été utilisé pour étudier les problèmes d'écoulement de fluides diphasiques dans les roches fissurées. Le but principal de l'étude était de déterminer l'influence de l'eau souterraine dur l'écoulement d'air depuis une excavation souterraine importante. Les informations obtenues ont été utilisées pour interpréter les résultats d'essais de pompage dans des sondages et prévoir les débits d'air pouvant intervenir autour de grandes excavations dans la même situation. Des essais de rabattement ont de plus été réalisés pour déterminer la charge de l'eau demeurant au-dessous d'une grande cavité sans revêtement lorsque le niveau se trouve rabattu de plusieurs diamètres de la cavité, au-dessous du niveau statique original. Les écoulements à travers une roche uniformément perméable et aussi à travers des fractures individuelles ont été considérés.

ZUSAMMENFASSUNG

Ein Modell mit parallelen Platten (ähnlich wie das Hele Shaws´sche Modell) wurde verwendet, um die Strömungsvorgänge einer zweiphasigen Flüssigkeit in zerklüftetem Fels zu untersuchen. Der Hauptzweck der Untersuchung war, den Einfluss des Grundwassers über die Luftbewegung aus grossen unterirdischen Hohlräumen abzuschätzen. Die gewonnenen Ergebnissen wurden für die Auswertung von einigen in situ Pumpversuchen und auch für die Abschätzung der Durchflüssmengen von Luft aus Hohlräumen in der gleichen Lage angewendet. Absenkungsversuche wurden auch durchgeführt, um die Wasserspiegelhöhe über einem grossen unverkleideten Hohlraum bei einer Absenkung von mehreren Hohlraumsdurchmesser des Wasserspiegels zu bestimmen. Strömungen durch einen gleichmässig zerklüfteten Fels sowie durch einzelne Klüfte wurden betrachtet.

INTRODUCTION

The containment of fluids in underground openings in rock is common practice in those Scandinavian countries which have the benefit of widely distributed igneous and metamorphic foundation rocks. It is not normally economically attractive to consider complete artificial lining of these openings, so leakage rates and velocities of migration are largely dependent on the inherent rock mass permeability. There is a class of problems where fluid migration need present only a limited problem, if the local ground water level can be maintained so that the hydrostatic pressure exceeds the fluid storage pressure. (Morfeldt (1).) However, a class of problems has recently come to notice in which it is difficult, if not impossible for ground water levels to be maintained. This is because the fluid may be at a much higher pressure than the local hydrostatic pressure, or alternatively, the opening and associated tunnels may be located at such shallow depth below the surface that localized drainage of the ground water occurs.

The model study to be described in this report was directed towards several complicated problems of

1972

Engineering Geology, 7(1973) 287—332
©Elsevier Scientific Publishing Company, Amsterdam — Printed in the Netherlands

Reviews

REVIEW OF A NEW SHEAR-STRENGTH CRITERION FOR ROCK JOINTS

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(Accepted for publication November 14, 1973)

ABSTRACT

Barton, N., 1973. Review of a new shear-strength criterion for rock joints. Eng. Geol., 7: 287—332.

The surface roughness of rock joints depends on their mode of origin, and on the mineralogy of the rock. Amongst the roughest joints will be those that formed in intrusive rocks in a tensile brittle manner, and amongst the smoothest the planar cleavage surface in slates. The range of friction angles exhibited by this spectrum will vary from about 75° or 80° down to 20° or 25°, the maximum values being very dependent on the normal stress, due to the strongly curved nature of the peak strength envelopes for rough unfilled joints.

Direct shear tests performed on model tension fractures have provided a very realistic picture of the behaviour of unfilled joints at the roughest end of the joint spectrum. The peak shear strength of rough—undulating joints such as tension surfaces can now be predicted with acceptable accuracy from a knowledge of only one parameter, namely the effective joint wall compressive strength or JCS value. For an unweathered joint this will be simply the unconfined compression strength of the unweathered rock. However in most cases joint walls will be weathered to some degree. Methods of estimating the strength of the weathered rock are discussed. The predicted values of shear strength compare favourably with experimental results reported in the literature, both for weathered and unweathered rough joints.

The shear strength of unfilled joints of intermediate roughness presents a problem since at present there is insufficient detailed reporting of test results. In an effort to remedy this situation, a simple roughness classification method has been devised which has a sliding scale of roughness. The curvature of the proposed strength envelopes reduces as the roughness coefficient reduces, and also varies with the strength of the weathered joint wall or unweathered rock, whichever is relevant. Values of the Coulomb parameters c and Φ fitted to the curves between the commonly used normal stress range of 5—20 kg/cm^2 appear to agree quite closely with experimental results.

The presence of water is found in practice to reduce the shear strength of rough unfilled joints but hardly to affect the strength of planar surfaces. This surprising experimental result is also predicted by the proposed criterion for peak strength. The shear strength depends on the compressive strength which is itself reduced by the presence of water. The sliding scale of roughness incorporates a reduced contribution from the compressive strength as the joint roughness reduces. Based on the same model, it is possible to draw an interesting analogy between the effects of weathering, saturation, time to failure, and scale, on the shear strength of non-planar joints. Increasing these parameters causes a reduction in the compressive strength of the rock, and hence a reduction in the peak shear strength. Rough—undulating joints are most affected and smooth—nearly planar joints least of all.

1973

Engineering Classification of Rock Masses for the Design of Tunnel Support

By

N. Barton, R. Lien, and J. Lunde

With 8 Figures

(Received August 31, 1974)

Summary — Zusammenfassung — Résumé

Engineering Classification of Rock Masses for the Design of Tunnel Support. An analysis of some 200 tunnel case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to tunnel stability. The numerical value of Q ranges from 0.001 (for exceptionally poor quality squeezing-ground) up to 1000 (for exceptionally good quality rock which is practically unjointed). The rock mass quality Q is a function of six parameters, each of which has a rating of importance, which can be estimated from surface mapping and can be updated during subsequent excavation. The six parameters are as follows; the *RQD* index, the number of joint sets, the roughness of the weakest joints, the degree of alteration or filling along the weakest joints, and two further parameters which account for the rock load and water inflow. In combination these parameters represent the rock block-size, the inter-block shear strength, and the active stress. The proposed classification is illustrated by means of field examples and selected case records.

Detailed analysis of the rock mass quality and corresponding support practice has shown that suitable permanent support can be estimated for the whole spectrum of rock qualities. This estimate is based on the rock mass quality Q, the support pressure, and the dimensions and purpose of the excavation. The support pressure appears to be a function of Q, the joint roughness, and the number of joint sets. The latter two determine the dilatency and the degree of freedom of the rock mass.

Detailed recommendations for support measures include various combinations of shotcrete, bolting, and cast concrete arches together with the appropriate bolt spacings and lengths, and the requisite thickness of shotcrete or concrete. The boundary between self supporting tunnels and those requiring some form of permanent support can be determined from the rock mass quality Q.

Key words: Classification, rock mass, joints, shear strength, tunnels, support pressure, shotcrete, bolts.

1974

A REVIEW OF THE SHEAR STRENGTH OF FILLED DISCONTINUITIES IN ROCK

Oversikt over skjærfastheten hos fylte diskontinuiteter i fjell.

Dr. Nick. Barton, Norges Geotekniske Institutt.

SUMMARY

Rock discontinuities that are filled with plastic materials represent one of the greatest problems in rock engineering. The wide range of properties and variety of occurrences make it extremely difficult to estimate the shear strength in anything but crude terms — for instance "low" ($Ø_r'$ = 12°—20°), or "very low" ($Ø_r'$ = 6°—12°). Even the ability to classify in this manner may be extremely valuable when designing the optimum anchoring or bolting required to stabilize surface cuttings or the walls of large underground openings. The most complicated and critical filled discontinuities may need to be tested in situ, if the cost of failure is sufficiently high.

If direct shear tests are to be performed it is extremely important that the test conditions are as relevant as possible to field conditions. The soil mechanics principles relevant to shearing and unloading problems are briefly reviewed. It would seem that slow drained teste will be the most relevant test method for all cases involving unloading above the critical filled discontinuities.

An increasing degree of complexity is introduced into the problem when the clay fillings are less thick than the roughness amplitude of the wall rock. A limited shear displacement will then result in a marked stiffening when opposed rock asperities make contact.

Both idealized laboratory models and engineering examples of rock wall interaction are reviewed, in an attempt to clarify the relative importance of filling behaviour and rock contact. Shear test results reported in the literature for filled discontinuities are tabulated in an appendix.

INTRODUCTION

For various reasons the rock joints of Norway, both clean and clay-filled, have hardly ever been tested in direct shear, either in the laboratory or in situ. Among the most important reasons for this apparent failure are:

(i) the unusually high strength of most of the rock
(ii) the relative lack of surface wathering, due to recent glacial erosion
(iii) generally widely spaced and discontinuous jointing
(iv) extremely varied and complicated occurences of filled joints

1973

The Shear Strength of Rock and Rock Joints

N. BARTON*

Rock joints exhibit a wide spectrum of shear strength under the low effective normal stress levels operating in most rock engineering problems. This is due to the strong influence of surface roughness and variable rock strength. Conversely, under the high effective normal stress levels of interest to tectonophysicists the shear strength spectrum of joints and artificial faults is narrow, despite the wide variation in the triaxial compression strength of rocks at fracture. In Part I of this review, empirical non-linear laws of friction and fracture are derived which explain this paradoxical behaviour and which can be used to predict or extrapolate shear strength data over the whole brittle range of behaviour.

Under higher confining pressures the behaviour of rock ceases to be brittle as the brittle–ductile transition is reached. Expressions are derived which quantify this condition and explain the variable transition behaviour of rocks as dissimilar as limestone and shale. At still higher confining pressures the Mohr envelopes describing failure of intact rock eventually reach a point of zero gradient on crossing a certain line, defined here as the critical state line. This critical state is associated with a critical effective confining pressure for each rock. It appears that the dilation normally associated with the shearing of non-planar joints and faults may be completely suppressed if the applied stress reaches the level of the critical effective confining pressure.

The empirical laws of friction and fracture were developed during a review of laboratory-scale testing on rock and rock joints. In Part II of this review these laws are applied to the interpretation of full-scale features. The following topics are investigated; the conjugate shear angle of shear joints and faults, the scale effect on frictional strength, the lack of correlation between stress drops measured in laboratory-scale faulting experiments and those back-calculated from major earthquakes, the strength corrosion caused by moisture, and finally the possible effect of fault dilation and water pressure changes at shallow depth in the crust.

INTRODUCTION

As recently as ten years ago Brace & Byerlee [1] suggested that the coefficient of friction relevant to a particular geologic situation could not be predicted to within better than a factor of two. This pessimistic observation is understandable when one considers the great range of stress to which rock and rock joints are subjected in the various engineering disciplines. In many rock engineering problems, the maximum effective normal stress acting across those joints considered critical for stability will lie in the range 0.1–2.0 MN/m^2 (1–20 kg/cm^2). However, tectonophysicists are generally interested in effective stress levels three orders of magnitude larger than this, for example 100–2000 MN/m^2 (1–20 kbars).

One of the most surprising conclusions arrived at as a result of high pressure triaxial tests on intact rock is the *apparent* lack of correlation between the fracture strength of the intact rock and the frictional strength of the resulting fault. Byerlee [2] has even gone so far as to suggest that the frictional strength of faults developed through intact rock may be the same for all rocks, independent of lithology.

At first sight there certainly appear to be reasonable grounds for his suggestion. Figure 1 shows that the peak shear strength of artificial faults (and tension fractures) in a variety of rocks falls within a *relatively* narrow zone when the effective normal stress is of the same order or greater than the unconfined compression strength of the rocks concerned. However, rock mechanics experience under low effective normal stress levels indicates that the shear strength of joints can vary within relatively wide limits as indicated in Fig. 2.

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1976

Recent experiences with the Q-system of tunnel support design

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SUMMARY

The Q-system of rock mass classification and support design is based on a numerical assessment of the rock mass quality using six different parameters. The six parameters consist of the RQD, the number of joint sets, the roughness of the most unfavourable joint or discontinuity, the degree of alteration or filling of the most unfavourable joint or discontinuity, the degree of water inflow, and the stress condition. Another classification system, the Geomechanics Classification (Bieniawski, 1973, 1974) is also based on six parameters. Qualitative differences between the two methods are discussed.

The 200 case records that were analysed when developing the Q-system, included more than 30 cases of permanently unsupported openings. An analysis of the rock mass characteristics involved has shown that certain characteristics are essential if an excavation is to be left permanently unsupported. If the maximum unsupported span for a given Q-value is exceeded, the safe life of the excavation may be shortened. A preliminary attempt is made to correlate stand-up time, rock mass quality Q, and span width.

The Q-system has been applied on several projects in Scandinavia and abroad since its development in 1973/1974. An example of a recent application is given in detail. The preliminary estimates of permanent support for a 19 metres span underground power house were obtained from an analysis of corelogs. In a subsequent site visit the Q-system was applied *in-situ*. The final estimates of permanent support were found to compare well with the preliminary estimates. Core logs, seismic profiles and surface mapping were used as a basis for preliminary design of permanent support for the 9 metres span tailrace tunnel, again using the Q-system. This tunnel is presently under construction so comparison of predicted and actual support is not yet possible.

KEY WORDS

Rock mass, classification, tunnel, powerhouse, support, borecore, case record.

INTRODUCTION

The six parameters chosen to describe the rock mass quality Q are as follows:

RQD = rock quality designation (Deere, 1963)
J_n = joint set number
J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor

These parameters are combined in pairs and are found to be crude measures of:

1. relative block size (RQD/J_n)
2. inter-block shear strength (J_r/J_a) $(\cong \tan \phi)$
3. active stress (J_w/SRF)

The overall quality Q is equal to the product of the three pairs:

$$Q = (RQD/J_n) \cdot (J_r/J_a) \cdot (J_w/SRF) \quad (1)$$

Thus, the following rock mass would be most favourable for tunnel stability: high RQD-value, small number of joint sets, appreciable joint roughness, minimal joint alteration of filling, minimal water inflow, and favourable stress levels.

Individual ratings of the six parameters have been published previously, together with detailed support tables from which estimates of appropriate permanent support can be obtained. In view of the fact that no changes have been found necessary, the support tables are not repeated in this paper, and readers should consult two earlier publications for such details (Barton, Lien and Lunde 1974, 1975). However the classification ratings are given here (see Appendix) so that the following examples of support prediction and classification philosophy may be more easily followed. These classification ratings are also unchanged from the original.

COMPARISON WITH THE GEOMECHANICS CLASSIFICATION SYSTEM

It is not the intention here to make a quantitative comparison between the Q-system and Bienawski's (1974) Geomechanics Classification since this is done in the general review paper in this symposium. However, certain qualitative differences can be mentioned which serve as a useful basis for

1976

The Shear Strength of Rock Joints in Theory and Practice

By

N. Barton and V. Choubey

With 20 Figures

(Received October 4, 1976)

Summary — Zusammenfassung — Résumé

The Shear Strength of Rock Joints in Theory and Practice. The paper describes an empirical law of friction for rock joints which can be used both for extrapolating and predicting shear strength data. The equation is based on three index parameters; the joint roughness coefficient *JRC*, the joint wall compressive strength *JCS*, and the residual friction angle ϕ_r. All these index values can be measured in the laboratory. They can also be measured in the field. Index tests and subsequent shear box tests on more than 100 joint samples have demonstrated that ϕ_r can be estimated to within $\pm 1^0$ for any one of the eight rock types investigated. The mean value of the peak shear strength angle (arctan τ/σ_n) for the same 100 joints was estimated to within $1/2^0$. The exceptionally close prediction of peak strength is made possible by performing self-weight (low stress) sliding tests on blocks with throughgoing joints. The total friction angle (arctan τ/σ_n) at which sliding occurs provides an estimate of the joint roughness coefficient *JRC*. The latter is constant over a range of effective normal stress of at least four orders of magnitude. However, it is found that both *JRC* and *JCS* reduce with increasing joint length. Increasing the length of joint therefore reduces not only the peak shear strength, but also the peak dilation angle and the peak shear stiffness. These important scale effects can be predicted at a fraction of the cost of performing large scale in situ direct shear tests.

Key Words: shear strength, joint, shear test, friction, compressive strength, weathering, roughness, dilation, stiffness, scale effect, prediction.

Die Scherfestigkeit von Kluftflächen in Theorie und Praxis. Zur Ermittlung der Reibungswerte in Kluftflächen wird ein empirisches Gesetz beschrieben, das sowohl das Extrapolieren als auch das Voraussagen von Scherfestigkeitszahlen ermöglicht. Die Gleichung ist auf drei Indexzahlen gegründet: Den Rauhigkeitskoeffizienten der Kluft *JRC* (Joint Roughness Coeff.), die Druckfestigkeit des Felses der Kluftwände *JCS* (Joint Wall Compression Strength) und der residuelle Reibungswinkel der Trennfläche ϕ_r.

1977

UNSUPPORTED UNDERGROUND OPENINGS

Nick Barton, Norges Geotekniske Institutt

SUMMARY

Underground openings can often be left permanently unsupported. The decision of "support" versus "no-support" depends on factors such as the rock mass properties, the span width and the type of excavation. Since unsupported openings range in span from less than 2 metres up to 100 metres, it is clearly the rock mass properties that are of prime importance. The NGI "Q-system" of rock mass classification is used to analyse published case records describing unsupported spans, in an attempt to recognise those rockmass properties which seem to be essential if an opening is to be permanently safe, yet unsupported. Among the essential properties the following were noteworthy; there should not be more than three joint sets, the joints should have some degree of roughness or non-planarity, there should be no alteration or clay filling of the joints, there should be minimal or zero water inflow, and stress levels should be medium. The RQD is usually high, but exceptions (i.e. RQD = 40) are sometimes found.

These published case records are supplemented with rockmass descriptions and Q-classification of the famous Carlsbad limestone caverns of New Mexico, where natural spans ranging from 50 to 100 metres are found. The paper is concluded with a discussion of stand-up times for unsupported openings.

1976

VERY LARGE SPAN OPENINGS AT SHALLOW DEPTH: DEFORMATION MAGNITUDES FROM JOINTED MODELS AND F.E. ANALYSIS

by Nick Barton and Harald Hansteen

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The deformations resulting from excavation of very large openings are compared using two-dimensional F.E. continuum analyses and discontinuous physical models (20,000 discrete blocks). Both the joint orientations and the model horizontal stress levels were varied. Some models were dynamically loaded to simulate earthquakes (0.2-0.7 g). Model deformations were recorded using photogrammetry. The changes in deformation when increasing the simulated spans from 20 m to 50 m were of particular interest. High horizontal stress caused surface heave when joint orientations were favourable for arch stability. Joint orientations also determined whether the pillars between parallel openings were in a state of compression or tension.

INTRODUCTION

The engineering performance of large rock caverns has traditionally been learned from mining and hydro power projects, where the depth below surface is often many times greater than the span of the openings. Deformations measured in the walls and roofs of hydro power caverns generally range from about 5-50 mm, though there is a documented case where a wall moved in 126 mm (1), and another where the arch moved down 147 mm (2).

The chief objectives of the present studies of large near-surface openings were threefold:

(i) to provide deformation data to compare with monitored data from planned engineering projects involving large span near-surface excavations, e.g. underground sports complexes, civil defense shelters, nuclear power stations,

1979

APPLICATION OF Q-SYSTEM IN DESIGN DECISIONS CONCERNING DIMENSIONS AND APPROPRIATE SUPPORT FOR UNDERGROUND INSTALLATIONS

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ABSTRACT

Recent applications of the Q-system of rockmass classification are given. It is shown that four potential storage sites with different rockmass conditions may have different optimal cavern dimensions. Support costs may increase disproportionately if dimensions are chosen that are smaller or larger than the theoretical optimum of 18 to 24 metres span. The Q-system is also used for mapping rockmass conditions during tunnel and cavern construction, to aid in the choice of permanent support. Examples include 25 m^2 and 167 m^2 headrace tunnels, and an underground sewage treatment plant constructed in 1 km of caverns, 16 m in span. Mapping of associated collector and outlet tunnels is also illustrated. The former is being excavated by full-face tunnel boring machines. Finally it is shown how the Q-value can give a preliminary estimate of the *in situ* deformation modulus, and the range of likely deformations.

KEYWORDS

Rockmass classification, geological mapping, tunnels, caverns, support methods, support costs safety, optimum dimensions, unsupported excavations, case histories, deformations, *in situ* deformation moduli.

RESUME

L'article présente des applications récentes du système Q à la classification des massifs rocheux. Il est démontré qu'à quatre sites potentiels d'approvisionnement sous des conditions différentes de massif rocheux, les dimensions optimum des cavernes peuvent différer de façon significative. Le coût des appuis peut devenir hors de proportions si le design fait appel à une envergure plus grande ou plus petite que la dimension optimum théorique de 18 ou 24 mètres. Le système Q est aussi utilisé dans la cartographie des conditions du massif rocheux pendant la construction de tunnels et de cavernes, et peut ainsi aider au choix d'un appui permanent adéquat. Les exemples présentés incluent des tunnels d'amenée de 25 et 167 m^2 et une usine souterraine de traitement des eaux de 16 metrès d'envergure et menée dans un kilomètre de cavernes. L'article illustre aussi la cartographie de tunnels collecteurs et de vidange. Le premier est excavé au moyen de foreuses de tunnel (TBM) avec plein front de taille. Finalement il est démontré que la valeur Q peut donner une estimation préliminaire du module de déformation *in situ* et de l'étendue probable des déformations.

1980

Discussion

Discussion of paper by J. Krahn and N. R. Morgenstern "The ultimate frictional resistance of rock discontinuities". *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **16**, 127–133 (1979).

The authors [1] have observed that "two rock samples of the same mineral composition and tested under the same stress state will not have the same large strain shearing resistance if the structure or roughness along the shearing surfaces is not similar". They therefore suggest the use of the term *ultimate* frictional resistance in place of the word *residual*. This terminology was in fact also used some years ago by Krsmanovic [2], who recognised the limits imposed by direct shear testing. He also found that rough bedding planes in limestone were a long way from their residual strength even after several centimeters of shearing.

In the author's tests [1] samples of natural discontinuities of 15 cm length (A = 225 cm^2) were sheared about 2 cm (13% of their length) before reversing. According to a typical shear load-deformation record, shear resistance gradually reduced to the end of these 4 cm of shearing. All their samples of natural discontinuities were in limestone from Turtle Mountain. Yet because of different roughness their *bedding planes* had ultimate strength of 32°, while their smooth *joints* and *flexural slip surfaces* were down to 14° and 15° respectively.

There may in fact be additional reasons than *initial roughness* for these differences. The state of weathering or joint wall hardness was not described by the authors [1]. Different values of JCS (*joint wall compression strength*) might possibly have been discovered due to the different geological histories of the various discontinuities. Barton & Choubey [3] have in fact found that the *residual friction angle* (ϕ_r) of a joint (the theoretical minimum, with all roughness worn away) is a function of the relative strengths of the joint wall material and the stronger unweathered material in the interior of each block.

$$\phi_r = (\phi_b - 20°) + 20(r/R) \qquad (1)$$

where

ϕ_b = basic friction angle estimated from *tilt* (self weight) tests on dry unweathered sawn surfaces of the particular rock

R = Schmidt hammer rebound on the dry sawn surfaces (unweathered)

r = Schmidt hammer rebound on the wet joint surfaces (weathered)

prototype (p) displacements and normal stresses are shown in figure.

It is clear from the continuing dilation seen in the lower half of the figure that at the end of the tests roughness is still contributing to the shear strength. The authors would correctly refer to this as the *ultimate* shear strength.

The *peak strength* (τ) is described by the following equation ref. [3]:

$$\tau = \sigma_n \tan\,[\mathrm{JRC} \log_{10}(\mathrm{JCS}/\sigma_n) + \phi_r] \qquad (2)$$

where

σ_n = effective normal stress

JRC = joint roughness coefficient (at peak)

JCS = joint wall compression strength (measured with a Schmidt hammer ref. [3]).

For the case of these rough model tension joints JRC = 20.0 and JCS = 0.4 MPa. The latter is the same as the unconfined compression strength of the model material since there is no weathering.

Equation 2 can be rearranged so that the roughness mobilized at any displacement can be back-calculated:

$$\mathrm{JRC(mobilized)} = \frac{\arctan(\tau_m/\sigma_n)^\circ - \phi_r^\circ}{\log_{10}(\mathrm{JCS}/\sigma_n)} \qquad (3)$$

where τ_m = shear strength mobilized at any displacement.

Equation 3 was evaluated at several points along each of the shear force-displacement curves shown in Fig. 1. Data was then normalized to the form JRC(mobilized)/JRC(peak) and δ_h/δ_h(peak), where δ_h(peak) was the displacement required to reach peak strength under the particular test. This dimensionless data is shown in Fig. 2.

It can be shown that:

$$\frac{\mathrm{JRC(mobilized)}}{\mathrm{JRC(peak)}} = \frac{\arctan(\tau_m/\sigma_n)^\circ - \phi_r^\circ}{\phi_p^\circ - \phi_r^\circ} \qquad (4)$$

where $\phi_p = \arctan\,(\tau_{peak}/\sigma_n)$.

When JRC(mob.)/JRC(peak) = 0.5, the shear strength mobilized is equal to $\frac{1}{2}(\phi_p + \phi_r)$. In other words shear strength is midway between peak and residual. This point seems to occur at approximately 10 δ_h(peak) for the case of the rough model tension joints. (Smoother joints, or those under the influence of

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Technical Note

Some Effects of Scale on the Shear Strength of Joints

NICK BARTON*

STAVROS BANDIS†

In a recent article, Tse & Cruden [1] pointed out that fairly small errors in estimating the *joint roughness coefficient* (JRC) when visually comparing joint profiles, could result in serious errors in estimating the peak shear strength (τ) from equation (1), (Barton & Choubey [2]), especially if the ratio JCS/σ_n was large.

$$\tau = \sigma_n \tan\,(\mathrm{JRC} \log_{10}(\mathrm{JCS}/\sigma_n) + \phi_r) \qquad (1)$$

where

σ_n = effective normal stress

JCS = joint wall compression strength

ϕ_r = residual friction angle

They therefore recommended a numerical check of the value of JRC, based on a detailed profiling and analysis utilizing several of the mathematical techniques for describing surface characteristics used in mechanical engineering, to "avoid the subjectivity of estimates of JRC by comparison with typical profiles."

A key point of Barton & Choubey's recommendations [2] was in fact that *tilt* or *push* tests (shear tests under self-weight induced stresses) were a more reliable method of estimating JRC than comparison with typical profiles. Surprisingly Tse & Cruden [1] did not proceed to the important question of scale effect on shear strength.

Scale effect on JRC

In practice it is found that JRC is only a constant for a fixed joint length. Generally, longer profiles (of the same joint) have lower JRC values. Consequently longer samples tend to have lower peak shear strength, as demonstrated conclusively by Pratt *et al.* [3].

Barton & Choubey [2] suggested that the correct size of joint for indexing (shear testing or surface analysis) might as a first approximation be given by the natural block size (specifically the spacing of cross-joints). Rock masses with widely spaced joints have less freedom for block rotation than rock masses with small block sizes. Smaller blocks have greater freedom to follow and 'feel' the smaller scale and steeper asperities of the component joints hence the higher JRC values. This scale effect is illustrated in Fig. 1. (It is appreciated that this freedom for individual rotation may be limited at high stress levels).

In effect the spacing of cross-joints (or block size) is the minimum 'hinge' length in the rock mass, hence its significance as a potential scale effect size limit.

The above scale effect could presumably be simulated by Tse & Cruden's [1] numerical analysis of surface coordinates if larger 'steps' were taken when profiling longer joints. This technique was used by Fecker & Rengers [4] and Barton [5]. In effect, the larger steps jump over the smaller steep asperities, thereby sampling only the longer and more gently inclined asperities which seem to control full scale shear strength, cf. Patton [6]. The shear displacement required to mobilize peak strength seems to be a measure of the distance the

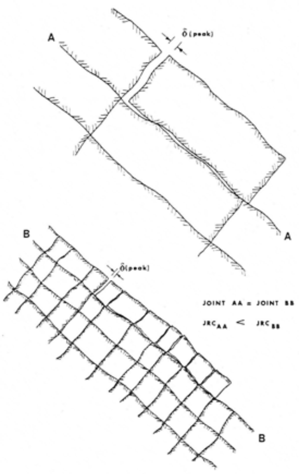

Fig. 1. Scale effect determined by block size, after Barton & Choubey [2].

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Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 18, pp. 1 to 21
0020-7624/81/0201-0001S02.00/0

Experimental Studies of Scale Effects on the Shear Behaviour of Rock Joints

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The effect of scale on the shear behaviour of joints is studied by performing direct shear tests on different sized replicas cast from various natural joint surfaces. The results show significant scale effects on both the shear strength and deformation characteristics. Scale effects are more pronounced in the case of rough, undulating joint types, whereas they are virtually absent for planar joints. The key factor is the involvement of different asperity sizes in controlling the peak behaviour of different lengths of joints. It is shown that as a result both the joint roughness coefficient (JRC) and the joint compression strength (JCS) reduce with increasing scale. The behaviour of multiple jointed masses with different joint spacings is also considered. It is found that despite unchanged roughness, jointed masses consisting of many small blocks have higher peak shear strength than jointed masses with larger joint spacing. These scale effects are related to the changing stiffness of a rock mass as the block size or joint spacing increases or decreases. Economic methods for obtaining scale-free estimates of shear strength are described.

INTRODUCTION

The choice of an appropriate joint test-size during a shear strength investigation is generally based on both economic and technical considerations. The high cost of large scale conventional shear tests often leads to the relatively cheaper alternative of laboratory testing of small joint samples. However, small samples usually represent only a fraction of the natural joint exposures and such tests often yield unrepresentative data. Schneider [1] notes the reluctance of practicing engineers to apply friction values determined on 'laboratory'-size samples, a situation that often leads to more or less arbitrary reductions of friction angles (peak or residual) by 1/3 to 1/2 of their measured value.

The potential influence of joint test-size on measurements of shear strength has often been pointed out [2–5]. However, few systematic studies of the scale effect have been reported, and existing data from small and large scale tests are extremely limited and often inconclusive. One reason is that large *in-situ* shear tests are generally reserved for the most critical situations such as infilled joints, shear zones etc., where scale effects appear to be absent [3, 6, 7]. This is to be expected in those cases with a thickness of infilling larger than the roughness amplitude.

Comparisons of data from unfilled joints present a confusing picture because some tests indicate no scale effect [8], whereas in other cases the scale effect is either 'positive' [9] or 'negative' [10]. 'Negative' scale effects are often the result of dissimilar roughness on the small and large joints. For instance, in the case of Locher and Rieder's tests the laboratory samples were described as smooth, whereas the *in-situ* tested joints had undulating surfaces with amplitudes of ±2 cm. This could explain why the *in-situ* peak friction angle was 5° higher than that measured in the laboratory. Brown *et al.* [11] also found that the peak shear strength of artificially parted cleavage planes in slate increased as the sample areas increased from 60 to 1000 cm^2. Those authors noted that parting of the slate blocks produced surfaces 'stepping' from one cleavage plane to another. As would be expected, this effect became more marked as the sample size increased and produced 'rougher' surfaces with higher strength.

Different sizes of joint samples with similar roughness have shown 'positive' scale effects. A series of field shear tests by Pratt *et al.* [9] on a range of joint sizes in a weathered quartz diorite showed a 40% reduction in peak shear strength as the sample areas increased from 140 to 5000 cm^2. The family of shear stress (τ) displacement (d_h) curves in Fig. 1 summarizes those experimental results. Barton & Choubey [12] measured tilt angles of 59° during self-weight sliding tests on a 45 cm long joint in granite. When the same sample was subdivided into eighteen blocks 10 cm in length, an average angle of 69° was obtained from a combination of tilt and push tests.

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SHEAR STRENGTH INVESTIGATIONS FOR SURFACE MINING

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ABSTRACT

Simple methods for estimating the shear strength of rock joints and waste rock are reviewed. For the case of rock joints, the methods are based on a quantitative characterization of the joint roughness and the joint wall strength. Size-effects are found to reduce the peak strength of large joint samples to values below the ultimate or so-called "residual" values measured in the laboratory. Tilt tests and surface profiling on natural size blocks within the jointed rock mass are recommended for obtaining scale-free properties. The joint parameters obtained can be used to model complete strength-displacement-dilation behavior if this level of input is required. Large scale tilt tests can be performed with advantage on both rock joints and waste rock. The behavior of these two materials is surprisingly similar. Both are influenced by the size-effects on the compression strength of the rock, and both have similar log-linear relationships between effective normal stress and the peak drained friction angles. The resulting high values of friction near the toe or close to a slope face in either material can be misleading.

INTRODUCTION

It is now known with reasonable certainty that tests on small samples of rock produce artificially high values of strength. In the past, arguments have been put forward to explain size-effects by changed stress distributions, changed machine stiffness, etc. Such arguments cannot be invoked to explain scale effects observed on joints. A simple but convincing demonstration is the tilt test. Tilt angles measured during self-weight gravity sliding tests of a large slab of jointed rock are found to be many degrees less than

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16374 JULY 1981 GT7

SHEAR STRENGTH OF ROCKFILL

By Nick Barton[1] and Bjørn Kjærnsli[2]

INTRODUCTION

An important question which arises during stability analysis of rockfill dams is the relative frictional resistance of the rockfill and the underlying rock foundation interface. Attempts to estimate the shear strength of smooth, ice-polished interfaces has led to an examination of the related shear behavior of rock joints. It is found that rockfill, interfaces, and rock joints have several features in common, including dilatant behavior under low effective normal stress, and significant crushing of contact points with reduced dilation at high stress. In each case failure is resisted by strongly stress dependent friction angles. As an example, the peak drained friction angle ϕ' of rockfill near the base of a high dam may be as low as 35°, while close to the toe the same rockfill might exhibit a value of ϕ' as high as 60°. Similar stress dependency is observed with rock joints and interfaces.

In this article it is shown how the value of ϕ' for rockfill can be estimated from knowledge of the following parameters: (1) The uniaxial compressive strength of the rock; (2) the d_{50} particle size; (3) the degree of particle roundedness; and (4) the porosity following compaction. The degree of particle roundedness and the porosity determine the magnitude of the structural component of strength. This component increases shear resistance in much the same way as interlocking asperities on a rough joint surface. The structural component of strength is strongly stress dependent. It is added to the basic angle of friction ϕ_b of flat nondilatant, i.e., sawn, surfaces of the rock to obtain ϕ'.

The methods developed here are of a simple practical nature, and allow a dam designer to obtain a preliminary estimate of the peak drained friction angle of rockfill, whether it consists of angular quarried rock, moraine, or well-rounded fluvial gravels. Simple large scale tilt tests of in-place rockfill are suggested for checking the shear strength in different lifts of a dam, during construction.

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Note.—Discussion open until December 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on December 11, 1980. This paper is part of the Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. GT7, July, 1981. ISSN 0093-6405/81/0007-0873/$01.00.

1981

SITE CHARACTERIZATION OF JOINT PERMEABILITY USING THE HEATED BLOCK TEST

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INTRODUCTION

The isolation of nuclear waste in a mined rock epository poses unique problems in site characteriza-ion. The ultimate barrier to radionuclide migration o the biosphere is the joints and major discontinui-ies that are pervasive at least to several kilometers epth. Modelling the potential effects of these joints n near-field conditions requires that the thermal, echanical and hydraulic properties of joints are oupled. Acquisition of joint data is therefore a ore demanding problem than at any previous time.

Tunneling and mining experience, physical models Barton and Hansteen, 1979) and numerical models Voegele 1978, Wahi et al. 1980) demonstrate the ossibility of significant shear displacement along oints exposed by an excavation. This process is nhanced by anisotropic stress distributions, by tran-ient thermal loading and by dynamic loading from arthquakes. If the relevant joints are rough, with igh wall strength, stability will not necessarily be educed by the shearing process since roughness-nduced dilation will lock the joints in some finite isplaced position. The only serious consequence of nis process is the joint aperture strain. Perme-bility may be enhanced around the repository and haft. According to the present studies a significant hear displacement may be as little as 0.2 mm.

Model studies of flow in a rough joint replica heared at very low stress reported by Maini (1971), ndicated that joint permeability could increase as uch as one order of magnitude in the first 2 mm of hear displacement, and a further one order of magni-ude in the next 4 mm of shear displacement. Although hese effects would be reduced at realistic levels of ormal stress, their influence could have important nfluence on repository sealing requirements.

The effect of temperature on joint permeability has ot been an area of extensive research, although this eficiency is rapidly being adjusted. Tests by Nelson 975) on single fractures in sandstone subjected to a elatively low confining pressure (0.1 MPa) indicated nitial increases in permeability to 60°C, followed by gnificant reductions when increasing the temperature urther to 100°C. In-situ tests conducted in Stripa ranite by Lundström and Stille (1978) using water emperatures of 10°C and 35°C indicated a 50% reduc-on in joint permeability, despite the reduced vis-osity of water at the higher temperature. Unfortun-ely there was no coupling of permeability measure-

spoon et al. 1979a) have suggested that the cubic law relating aperture and flow rate is valid even for rough fractures in intimate contact. Other authors (e.g. Kranz et al. (1979) and Walsh (1981)) have explained the measured flow reductions caused by tortuosity and roughness, by a modification to the law of effective stress.

The possibility of a scale effect on joint permeability has been suggested by Witherspoon et al. (1979b). At present, the data base is too limited and diverse to make definite conclusions. It is often unreasonable to try to compare the permeabilities of rough, fresh artificial fractures (a typical test configuration) with weathered natural joints of different roughness, since the degree of aperture closure under a given stress level will vary in each case. Recent work (Barton, 1981a) suggests that scale dependent joint permeability will probably not be a significant factor under conditions of pure normal closure, but will be observed when shearing occurs. This is due to the scale-dependent dilation that occurs when joints of different length are sheared, as shown in a major test program reported by Bandis (1980).

HEATED BLOCK TEST

The pressing need for large scale coupled thermo-mechanical-hydraulic test data prompted Terra Tek's current 8 m^3 block test, performed under contract with the Office of Nuclear Waste Isolation. The site is located in gneiss, about 150 metres underground in a test adit in the Colorado School of Mines experimental mine in Idaho Springs.

The 2x2x2 metres block is located in the floor of the test adit. Loading is applied on four vertical sides with flatjacks. The base is attached to the surrounding rock mass. The vertical sides of the specimen were formed by line drilling. The extreme hardness of the quartz lenses in the gneiss caused unexpected difficulties with hole alignment, and diamond coring of the slots was required.

The surface of the block is instrumented with some 30 pairs of Whittemore bolts for recording strain and/or displacement across joints, four Irad strain metres, and five surface strain gauge rosettes. Deformation occurring across the block as a whole is registered with horizontal DCDT rod extensometers. Deformations within the block are monitored with MPBX borehole extensometers. Stress levels are monitored

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(NB was sole author, but listed authors from TerraTek did all block-test preparation, and most of the in situ testing).

SOME SIZE DEPENDENT PROPERTIES OF JOINTS AND FAULTS

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Abstract. Marked strength-size effects are observed when joints are subjected to shear. This is due to the mobilization of larger, but less steeply inclined asperities as sample size is increased. The displacement required to mobilize strength is also increased by the changing size of sample. These observed size effects indicate that large scale tests should be performed to obtain realistic data concerning shear behavior, dilation, and associated permeability changes.

Introduction

A simple empirical method of characterizing the shear behavior of rock joints was developed some years ago. It consists of three components: ϕ_b, JRC and JCS. A basic or residual friction angle (ϕ_b or ϕ_r) for flat non-dilatent surfaces in fresh or weathered rock, respectively, forms the limiting value of shear strength. To this is added a roughness component (i). This is normal stress dependent and varies with the magnitude of the joint wall compressive strength (JCS), and with the joint roughness coefficient (JRC). The latter varies from about 0 to 20 for smooth to very rough surfaces respectively. The peak drained angle of friction (ϕ') at any given effective normal stress (σ'_n) is expressed as follows:

$$\phi' = \phi_r + i = \text{JRC} \log(\text{JCS}/\sigma'_n) + \phi_r \qquad (1)$$

Each parameter can be quantified by simple index tests: tilt or self-weight sliding tests for JRC, and Schmidt hammer rebound tests for JCS and ϕ_r. Details are given by Barton and Choubey (1977). When a joint is unweathered or the strength of a fresh fracture or laboratory "fault" is of interest, the value of the unconfined compression strength (σ_c) is used in place of JCS. At extremely high normal stress, confining effects increase the value of JCS to $\sigma_1 - \sigma_3$, the differential stress. This form of equation fits high pressure triaxial strength data for "faults" extremely closely, as shown by Barton (1976).

The anticipated scale dependency of JRC and JCS deduced when analyzing large scale shear tests of joints in quartz diorite (Pratt, et al., 1974) has recently been confirmed in a comprehensive series of tests reported by Bandis, et al. (1981). Figure 1 illustrates diagramatically how the dilation component (d_n) and the strength component (S_a) of asperities reduce with increasing scale. Both these components vary with the values of JRC and JCS.

Repeated tests on cast model replicas of a wide variety of joint surfaces indicate that JRC and JCS reduce most strongly when joints are rough. Planar joint surfaces exhibit only minor scale dependent properties. As a result of this work it is now possible to predict the approximate reduction of ϕ' with increasing scale for a wide variety of surfaces, by substituting scaled values of JRC and JCS in equation 1. Tentative application of scaled values of JRC and JCS to fault-size surfaces indicate that even a rough, jagged failure surface (i.e. a recent fault) may

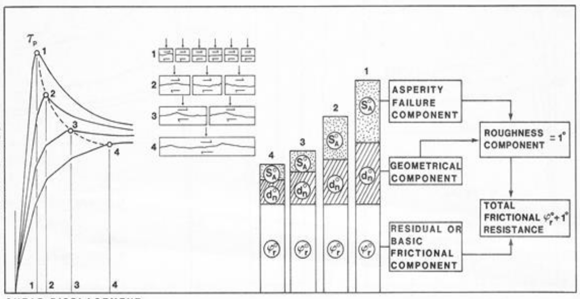

Figure 1. Scale dependent properties of rock joints, after Bandis et al. 1981.

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ONWI-308

Modelling Rock Joint Behavior from In Situ Block Tests: Implications for Nuclear Waste Repository Design

Technical Report

September 1982

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ONWI

1982

EFFECTS OF BLOCK SIZE ON THE SHEAR BEHAVIOR OF JOINTED ROCK

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ABSTRACT

The descriptive term "rock mass" encompasses individual block dimensions ranging from centimeters to many tens of meters. Strength and deformability vary both qualitatively and quantitatively as a result of this size range. A key issue is therefore the appropriate size of the test sample. A large body of test data was reviewed to determine the influence of block size on the displacement required to mobilize peak strength. It is shown that the shear strength and shear stiffness reduce with increased block size due to reduced effective joint roughness, and due to reduced asperity strength. Both are a function of the delayed mobilization of roughness with increasing block size. A method of scaling shear strength and shear displacement from laboratory to in situ block sizes is suggested. It is based on the assumption that size effects disappear when the natural block size is exceeded. This simplification appears to be justified over a significant range of block sizes, but is invalidated when shearing along individual joints is replaced by rotational or kink-band deformation, as seen in more heavily jointed rock masses. Recent laboratory tests on model block assemblies illustrate some important effects of block size on deformability and Poisson's ratio.

INTRODUCTION

The wide range of natural block sizes found in nature has a strong and obvious influence on the morphology of a landscape. The contrast in natural slope angles and slope heights sustained by a ravelling "sugar cube" quartzite and a monolithic body of granite suggests that block size may be a controlling factor when compressive strength and slake durability are high in each case. In a tunnel, the contrast in behavior may produce more than an order of magnitude change in costs

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ROCK MASS CHARACTERIZATION METHODS FOR NUCLEAR WASTE REPOSITORIES IN JOINTED ROCK

Gebirgsklassifikationen für Lagerstätten von radioaktiven Stoffen in geklüftetem Fels

Méthodes de classification des massifs rocheux pour le dépôt des déchets nucléaires dans les roches à diaclases

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SUMMARY:
The planned isolation of nuclear waste in mined rock repositories poses unusual requirements for rock mass characterization. This paper describes recently developed block test methods for characterizing and quantifying the thermal, mechanical and hydraulic properties of rock masses. The heated block test, recently conducted in situ on an $8m^3$ block of jointed gneiss, provides normal stress and temperature-dependent data such as deformation modulus, joint stiffness, joint permeability, thermal expansion, thermal conductivity and dynamic elastic modulus. Simpler tests conducted on singly jointed blocks or on jointed drill core provide joint roughness data. This is incorporated in recently developed constitutive models which describe the coupling of normal displacement, shear displacement, shear strength, dilation and permeability.

ZUSAMMENFASSUNG:
Die geplante Isolierung von radioaktivem Abfall in abgebauten Gesteinsabfällen stellt ungewöhnliche Anforderungen an Felsgesteinkennzeichnung. Dieser Bericht beschreibt kürzlich entwickelte Blocktestmethoden zur Kennzeichnung und quantitativen Bestimmung der Wärmeeigenschaften und der mechanischen und hydraulischen Eigenschaften der Felsmasse. Der Heißblocktest, der kürzlich "in-situ" auf einem $8m^3$ großen Block von geklüftetem Gneis durchgeführt wurde, gibt normale Spannungs- und Temperaturabhängigkeitswerte wie Verzerrungsmodul, Verbindungssteifigkeit, Verbindungsdurchlässigkeit, Wärmeausdehnung, Wärmeleitfähigkeit und Dynamik elastizitätsmodul. Einfachere Messungen, die an einzeln geklüfteten Blcoks oder an geklüfteten Bohrkernen vorgenommen wurden, geben Kluftrauhigkeitswerte an. Diese sind miteingeschlossen in kürzlich entwickelte zusammenfassende Modelle, die die Kupplung normaler Verlagerung, Scherverlagerung, Scherfestigkeit, Dehnung und Durchlässigkeit beschreiben.

RESUME:
L'isolation projetée des déchets nucléaires dans des dépôts de roche extraites présente des exigences peu commune pour la caractérisation de la masse de roche. Cette étude décrit les méthodes, récemment développées, des essais des blocs pour caractériser et pour quantifier les propriétés thermiques, méchaniques et hydrauliques des masses de roche. L'essai du bloc chaud, récemment conduit in situ sur un bloc de gneiss jointé (de $8m^3$) fournit l'indication normale qui dépend de la force et de la température par exemple, le coéfficient de la déformation, la dureté de la jointure, la perméabilité de la jointure, l'expansion thermique, la conductivité thermique et le coéfficient dynamique et élastique. Des essais plus simples qu'on a conduits sur des blocs individuellement jointés ou sur le centre jointé d'un trepan fournissent les indications de la rugosité des jointures. Tout ça est incorporé dans des modèles de base, récemment développés, qui décrivent le couplage du déplacement normal, du déplacement du cisaillement, de la force du cisaillement, de la dilation et de la perméabilité.

1982

24th U.S. Symposium
on Rock Mechanics
June 1983

INSTRUMENTATION AND ANALYSIS OF A DEEP SHAFT IN QUARTZITE

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ABSTRACT

Terra Tek Engineering instrumented and monitored a concrete shaft lining and the surrounding quartzite at levels 2412 ft, 4063 ft, and 5191 ft during sinking of the Silver Shaft at Hecla Mining Company's Lucky Friday Mine in Wallace, Idaho. Rock displacements were monitored with three 50 ft long multiple position borehole extensometers at each level. Stress build-up in the concrete lining was monitored with four pressure cells at each level, and the corresponding strains were monitored with embedded strain gages. Data from each level are compared, and a detailed analysis of rock and concrete behavior at the 5191 ft level is given. The results of the rock displacement and concrete lining stress monitoring have thrown light on several important aspects of deep shaft liner design, and on the behavior of jointed ground at great depth.

INTRODUCTION

Hecla Mining Company is further developing its Lucky Friday Mine in Idaho with the construction of a 6000 ft deep shaft. J. S. Redpath Corporation has sunk the shaft at record rates, achieving 473 ft of advance in one month, early in 1981. The concrete lined 18 ft diameter shaft is the first circular shaft ever sunk in nearly 100 years of mining in the Coeur d'Alene district, where rectangular, vertical or inclined timbered access ways have been the rule.

It is generally assumed that geological structure controls rock displacement near the surface, while high stress levels dominate behavior at depth. However, even at the 5191 ft level, the marked bedding joints in the quartzite were found to dominate behavior, causing distinctly anisotropic and non-elastic deformation of the rock, and correspondingly anisotropic concrete liner stresses. The principal stress direction trends NW-SE, parallel to the steeply dipping bedding joints at this level.

INSTRUMENT INSTALLATION SEQUENCE

Figures 1 and 2 illustrate the installation sequence followed for the 5191 ft level. The three fifty foot long MPB extensometers were installed close to the face, with the instrument heads protected by dog-hole recesses The shaft sinking then continued while continuously monitoring the three MPBX instruments, which had grouted anchors at radial depths of 50, 30, 15, 10, 5 and 3 feet. When the base of the shaft was some 23 feet below the extensometers following five bench blasts, the kerb ring shuttering was lowered ready for the kerb ring pour (see Figure 2). Concrete pressure cells (P.C.) and strain gages (S.G.) were installed in this pour at the locations shown in section in Figure 3.

1983

Proceedings of the International Symposium on Rock Bolting / Abisko / 28 August-2 September 1983

Bolt design based on shear strength

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ABSTRACT: Laboratory tests for determining the shear strength of bolted rock joints indicate that peak values of strength are developed when a grouted bolt is installed at an angle of about 35-50° to the plane of the joint. This range of angles generally corresponds to the peak, post-peak, or pre-residual region of behavior. Solution of practical problems, such as the bolting of unstable slopes or of major wedges in underground caverns, indicates that minimum bolt capacities are required when the bolt is angled perpendicular to the frictional resultant, corresponding to the arctangent of (τ/σ_n) relative to the joint, as above. Methods are described for generating appropriate shear force-displacement curves for rock joints so that a bolt of a given stiffness can be installed at the appropriate angle of mobilized friction. Bolts of lower stiffness require smaller installation angles and correspondingly increased capacity. The use of bolts of lower stiffness, for example partly grouted bolts, may be justified if displacements are irresistable, or if other components of support are of reduced stiffness. Bolt design should always be compatible with the expected deformation.

1 INTRODUCTION

The correct design of rock bolt reinforcement is dependent on compatible strength-deformation properties of both the rock mass and the rock bolts. It is as easy to design a rock bolt system that is too stiff for the rock mass as it is to design a system that allows the rock to strain soften, thereby losing the interactive capacity of the reinforced system. Timing of support installation is of importance both for rock slope reinforcement and for tunnels. If fully grouted bolts are installed before any shear stresses are developed, the high stiffness of the bolts at each "joint-crossing" will cause the bolts to be subjected to the full excavation-induced shear stress, with little if any assistance from the inherent strength of the joints. The difficult-to-achieve ideal is the bolt that reaches yield just tion. The frequent need for careful characterization of the joints is apparent.

2 REVIEW OF LABORATORY TEST DATA

Laboratory tests for determining the shear strength of bolted rock joints indicate that peak values of strength are developed when a grouted bolt is installed at an angle of about 35-50° to the plane of the joint. Test results for 450 mm (17.7 inches) long jointed blocks reported by Bjurström (1974) are reproduced in Figure 1. The most significant result is the existence of an optimum installation angle. Also of interest are the dowel and tensile components of strength and the added shear strength due to a normal stress increment caused by the inherent resistance of the bolt to dilation.

1983

Effects of Rock Mass Deformation on Tunnel Performance in Seismic Regions

Nick Barton

Résumé — On a observé que les mouvements sismiques augmentent l'écoulement de l'eau dans les équipements souterrains. L'augmentation apparente de la perméabilité de l'ensemble est problablement causée par les glissements localisés des joints; glissements qui résulteraient en des changements, petits mais irréversibles, dans l'ouverture de ces joints. De tels changements peuvent être observés physiquement et numériquement dans les excavations dans les joints rocheux. On décrit des méthodes pour mesurer les propriétés des joints, pour rendre l'orientation des tunnels la meilleure et pour prévoir les supports nécessaires. Une attention particulière est portée à la relation entre le glissement des joints l'un sur l'autre et leur élargissement (dilatation) et leur effet sur la conductivité et sur l'agencement optimum pour les boulons flexibles utilisés pour renforcer le tunnel.

1. Introduction

Underground structures have a consistent record of suffering much less damage than surface facilities during earthquakes. Generally only portal areas or fault crossings have suffered severe damage. In the case of portals, the combination of poor ground, stiff linings and amplified near surface shaking, make earthquake resistant design very difficult. In the case of fault crossings, associated block motion may be irresistable by any method of support or rock mass reinforcement.

The reduced intensity of shaking experienced at depth and in more competent rock masses, appear to limit damage to occasional rock drops and to cracking of linings. These events may be the result of out-of-phase high frequency shaking, reactivation of joint slip, or positive or negative stress changes adversely affecting existing high or low stress conditions.

In each of these cases the net result may be partially irreversible strain, due to the hysteretic behaviour of jointed rock masses. As suggested above, the impact on stability may be minimal, but the secondary effect on coupled processes such as water inflow or leakage may be marked. Seemingly minor joint displacements can cause radical changes in conductivity.

The present international interest in geological disposal of high level nuclear waste has focussed particular attention on transport velocities through jointed media. Since migration of radionuclides via ground water flow is the only conceivable mechanism for release to the biosphere, any events that could cause radical changes in flow velocities are of potential concern. Reports describing mine flooding and cracking of linings as a result of earthquakes are indications of a potential problem that may have increased impact on design in the future.

1.1 *Influence of depth*

There are several reasons why earthquakes generally result in less damage underground than at the surface. The predominant surface (Rayleigh) waves decay almost exponentially with depth, and below the surface incident and reflected waves interfere so that the total amplitude is usually reduced. A general tendency for increasing modulus with depth, small excavation dimensions relative to the predominant wavelengths, and the general wave de-amplification with depth all contribute to the reduced damage.

Kanai and Tanaka [1] measured ratios of surface displacement to displacement at 300 m depth as high as 6 in the Hatachi copper mine, and up to 10 when surface data from an alluvium site was included. Similar trends are also indicated when monitoring the effects of underground nuclear explosions. Vortman and Long [2] showed mean peak vectors of acceleration, velocity and displacement that ranged in general from 2.5 to 1.9 times larger at the surface than at 500-m depth.

The relative mismatch of wavelengths and most tunnel dimensions suggests that relative strain between rock blocks can only occur with higher frequency waves, when out-of-phase motion is possible across the structure. Dowding [3] suggested that large accelerations at frequencies in the range 30–60 Hz are probably most capable of causing differential block motion and resulting damage in large excavations.

The location of large caverns at shallow depth may be particularly adverse in terms of seismic design. Stevens [4] refers to the case of a large near-surface stope 45 m below the surface at the Tombstone mine in Arizona, which suffered considerable loosening and rockfalls from the hanging wall during the severe jolting, which was witnessed by mine surveyors.

Although many references are made to more damaging effects of shaking at the surface than at depth in the same mines, there are a limited number of cases in which this trend is reversed, with larger displacements observed at depth than at the surface

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1984

Chapter 48

LARGE SCALE STATIC AND DYNAMIC FRICTION EXPERIMENTS

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ABSTRACT

A series of nineteen shear tests were performed on fractures 1 m^2 in area, generated in blocks of sandstone, granite, tuff, hydrostone and concrete. The tests were conducted under quasi-static and dynamic loading conditions. A vertical stress assisted fracturing technique was developed to create the fractures through the large test blocks. Prior to testing, the fractured surface of each block was characterized using the Barton JRC-JCS concept. The results of characterization were used to generate the peak strength envelope for each fractured surface. Attempts were made to model the stress path based on the classical transformation equations which assumes a theoretical plane, elastic isotropic properties, and therefore no slip. However, this approach gave rise to a stress path passing above the strength envelope which is clearly unacceptable. The results of the experimental investigations indicated that actual stress path is affected by the dilatancy due to fracture roughness, as well as by the side friction imposed by the boundary conditions. By introducing the corrections due to the dilation and boundary conditions into the stress transformation equation, the fully corrected stress paths for predicting the strength of fractured blocks were obtained.

INTRODUCTION

An experimental test program was devised to study the shear strength of rock joints. The large size of the samples (1 m^2) was designed to extend the data base beyond the usual limitations of laboratory test equipment. Attempts were also made to compare the shear strength under pseudo static and dynamic rates of shear. The tests were performed on large fractured blocks of sandstone, tuff,

1984

ANALYSER OG FORSØK FOR BELYSNING AV SETNINGER PÅ EKOFISK

NUMERICAL ANALYSES AND LABORATORY TESTS TO INVESTIGATE THE EKOFISK SUBSIDENCE

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SUMMARY

The Ekofisk subsidence is influencing 150 km^3 of the seabed sediments in the North Sea. Nearly 3 meters of subsidence at a present yearly rate of 40 to 45 cm/year has set in motion several studies of the phenomenon. NGI, under contract with the Norwegian Petroleum Directorate, has utilized advanced non-linear finite element and discrete element methods to investigate various compaction processes in the 300 meter thick chalk reservoir located 3 km beneath the seabed. These detailed calculations were used as a displacement boundary condition for large-scale continuum and discontinuum analyses (with bedding planes and faults) in order to investigate the extent and size of the subsidence. Detailed laboratory tests were performed on the reservoir joints, to measure their shear strength, stiffness and conductivity to hot (80°C) Ekofisk oil. These tests provided the input data for special numerical modelling of the deformation and permeability changes that can be caused by a large reduction in reservoir pore pressure in a jointed, deformable and permeable reservoir rock subjected to one-dimensional strain. An interesting and quite unexpected type of behaviour was discovered during these discontinuum analyses, which can have an important influence on future productivity in the reservoir.

1985

Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 22, No. 3, pp. 121–140, 1985
0148-9062/85 $3.00 + 0.00

Strength, Deformation and Conductivity Coupling of Rock Joints

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Construction of dams, tunnels and slopes in jointed, water-bearing rock causes complex interactions between joint deformation and effective stress. Joint deformation can take the form of normal closure, opening, shear and dilation. The resulting changes of aperture can cause as much as three orders of magnitude change in conductivity at moderate compressive stress levels. Even the heavily stressed joints found in oil and gas reservoirs may also exhibit significant stress-dependent conductivity during depletion, and during waterflood treatments. The magnitudes of the above processes are often strongly dependent on both the character and frequency of jointing.

In this paper the results of many years of research on joint properties are synthesized in a coupled joint behaviour model. Methods of joint characterization are described for obtaining the necessary input data. The model simulates stress- and size-dependent coupling of shear stress, displacement, dilation and conductivity, and of normal stress, closure and conductivity. These processes are the fundamental building blocks of rock mass behaviour. Model simulations are compared with experimental behaviour and numerous examples are given.

INTRODUCTION

The strength and deformability of rock joints have been the subjects of numerous investigations, both for dam sites and for major rock slopes. Extensive reviews of such tests have been given by Link [1], Goodman [2], Cundall *et al.* [3], Bandis [4] and Barton and Bakhtar [5]. It has now been established beyond reasonable doubt that both the shear strength and deformability of rock joints are size-dependent parameters. See for example Pratt *et al.* [6], Barton and Choubey [7] and Bandis *et al.* [8]. The size dependence and general behaviour are governed to a large extent by surface characteristics such as roughness and wall strength, and by block size [9]. At the moderate stress levels of interest in civil engineering and in surface mining, differences in behaviour between rock types may therefore be marked. At very high stress levels, differences between rock types tend to be masked due to the extensive surface damage. See for example Barton [10] and Byerlee [11].

Basic elements of joint strength and deformability are summarized in Figs 1 and 2. In simplified terms, the stress–deformation behaviour of rock joints is convex-shaped with shear loading [8], and concave-shaped with normal loading [12].

In a typical rock mass deformation test (i.e. a plate load test), the predominance of normal joint closure will usually result in concave load–deformation behaviour [13]. On occasions, such as in the NTS block test in Hanford basalt, [14], the shear components acting on hexagonal columnar jointing may be sufficiently strong to linearize the load–deformation behaviour. In effect, the convex and concave behaviours shown in Figs 1 and 2 are of roughly equal magnitude and cancel one another.

PART 1—CHARACTERIZATION

Joint Surface Characterization

Evidence that rock joint properties are dependent on surface characteristics such as roughness and wall strength can be deduced in part from the early work of Coulomb [15]. Direct physical evidence for the influence of surface joint properties were obtained by Jaeger [16], Patton [17], Rengers [18] and Barton [19]. Methods of quantifying roughness and wall strength and utilizing them in shear strength relations were developed by Barton and Choubey [7]. These methods were recently applied by Bandis [4] in his detailed studies of joint deformability and strength. As a result of this work it is now possible to predict shear strength–deformation behaviour and normal stress–closure behaviour with ac-

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1985

ROCK MECHANICS INVESTIGATIONS FOR UNLINED PRESSURE TUNNELS AND AIR CUSHION SURGE CHAMBERS

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ABSTRACT

NGI's investigations for unlined excavations designed for high internal pressures include geological mapping, rock mass classification, minifrac rock stress measurements and joint permeability measurements. In special cases numerical modelling may be performed, using the discrete element code UDEC and the Barton-Bandis joint sub-routine which allows joint conducting apertures to be tracked. Conducting apertures are affected by effective stress levels, shear, dilation and possible gouge production. Each of these effects can be measured on large diameter jointed drill core in a special biaxial deformation-flow apparatus. Practical problems caused by leakage at the transition from the unlined pressure tunnel to the steel lined section are addressed. A water curtain solution to leakages from air cushion surge chambers is described, using an operating example.

INTRODUCTION

A convenient starting point for a discussion of internal pressure effects on unlined rock excavations is the minifrac test. Four possible scenarios can be envisaged in such a test. These are illustrated in Figure 1. The four behaviour modes illustrated may occur in minifrac tests conducted from boreholes in heavily jointed rock; clearly an unfavourable starting point for such tests.

By changing the scale of the diagram, we can envisage an unlined pressure tunnel in a quite massive rock mass with two sets of widely spaced joints. Hydraulic joint jacking (A), hydraulic fracturing (B), hydraulic shearing (C), or combined modes (D) are each possible depending on the magnitude of the following factors:

1987

Deformation phenomena in jointed rock*

N. R. BARTON†

The role of rock joints in rock mass deformation phenomena is described. Individually, joints display concave-shaped stress–closure curves under normal loading and convex-shaped stress–displacement curves under shear, usually accompanied by dilation. The deformation behaviour of rock masses depends on the relative magnitudes of these components of closure, shear and dilation. The deformation of a rock mass may result in dramatic changes in the joint apertures and conductivities. Conversely, changes in joint water pressure cause changes in joint aperture which affect the overall deformation of the rock mass. Examples of compaction in jointed reservoirs and leakage phenomena in pressure tunnels are cited, each of which may be caused by changes in effective stress. The presence of rock joints is seen to affect stress slabbing phenomena in tunnels and is the suspected cause of depth-dependent contrasts of stress in sedimentary rocks. The phenomenon of hydraulic shearing of joints is discussed with particular reference to geothermal reservoir stimulation. Shearing is also the suspected mechanism in cases of mine flooding, following seismic loading. A method of modelling this dilation–conductivity coupling is presented. The Paper concludes by analysing the role of joint dilation in stress transformations and in the behaviour of underground openings. Rock masses have greater resistance to shear than predicted owing to non-coaxial stress and strain components. The shear strength and both the shear and normal stress components are affected by dilation.

L'article décrit le rôle joué par les joints des roches dans les phénomènes de déformation en masse des roches. Individuellement les joints montrent des courbes de contrainte–fermeture concaves sous des charges normales et des courbes de contrainte-déplacement convexes sous le cisaillement, généralement accompagnées de dilatance. Le comportement de déformation des masses rocheuses dépend des valeurs relatives de la fermeture, du cisaillement et de la dilatance. La déformation d'une masse rocheuse peut produire des changements dramatiques dans les ouvertures des joints et dans les conductibilités. Réciproquement, des changements dans la pression de l'eau dans les joints provoquent des changements dans leurs ouvertures qui affectent la déformation totale de la masse rocheuse. L'article mentionne des exemples de compactage dans des réservoirs jointoyés et des phénomènes de fuite dans des tunnels de pression. Chacun de ces exemples ayant pu être causé par des changements dans les contraintes effectives. On observe que la présence de joints dans les roches affecte les phénomènes de formation de dalles par contrainte dans les tunnels et représente la cause présumée des contrastes de contrainte en profondeur dans les roches sédimentaires. On discute le phénomène du cisaillement hydraulique plus particulièrement eu égard à la stimulation géothermique des réservoirs. Le cisaillement est aussi la cause présumée de cas d'inondation de mines à la suite de chargements sismiques. Après avoir présenté une méthode pour modeliser cette combinaison de dilatance et de conductibilité l'article conclut par l'analyse de rôle joué par la dilatation des joints dans les transformations de contrainte et aussi dans le comportement des ouvertures souterraines. Les masses rocheuses ont une résistance au cisaillement supérieure à la valeur prédite, à cause des composantes non-coaxiales de déformation et de contrainte. La dilatance affecte à la fois la résistance au cisaillement et les composantes de cisaillement et de contrainte normale.

KEYWORDS: constitutive relations; deformation; pore pressures; rock mechanics; shear strength; tunnels.

NOTATION

d_n	dilation angle
e	theoretical smooth wall conducting aperture of a joint
E	physical aperture of a joint
e_0	initial conducting aperture of a joint under nominally zero stress
E_0	initial physical aperture of a joint under nominally zero stress
Δe	change in conducting aperture
ΔE	change in physical aperture
JCS	joint wall compression strength
JRC	joint roughness coefficient
k	joint conductivity, $e^2/12$
L_n	in situ block size (equal to the spacing of cross-joints)
M	deformation modulus
γ	density
δ	shear displacement along a joint
δ_{peak}	shear displacement at peak shear strength
σ_h	minor or intermediate horizontal principal stress
σ_H	major horizontal principal stress
σ_n	normal stress
σ_v	vertical principal stress
τ	shear stress
ϕ_b	basic friction angle (unweathered rock surface)
ϕ_p	peak friction angle

* 8th Laurits Bjerrum Memorial Lecture.
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1986

Predicting the Behaviour of Underground Openings in Rock

By
Nick Barton

SYNOPSIS

Rock masses range from intact homogeneous media, through regularly jointed assemblies of blocks, to heavily sheared and crushed clay-bearing fault zones. A brief review of the behaviour of openings created in these media, such as boreholes, tunnels and large mine openings, indicate some consistent trends. The fundamental failure mechanisms and requirements for support are believed to be strongly influenced in all cases by the volume changes accompanying potential failure. Failure, even in unjointed media, eventually occurs on failure surfaces such as extension fractures or shear fractures. A "plastic" zone may not strictly exist since the rock between the failure surfaces can be intact and relatively less highly stressed. Since the volume changes accompanying failure are to a great extent determined by the dilation (or contraction) along these failure surfaces or pre-existing discontinuities, an understanding of the latter is fundamental to the prediction of underground opening behaviour.

Careful measurements of model jointed media are used to demonstrate some of the volume changes that can be expected when rock masses are subjected to increased shear stress. The components of joint deformation such as closure and shear-induced dilation determine the type of response of individual rock masses. Attempts are made to provide a unifying description of rock mass shear strength based on discontinuum behaviour. Both the Q-system and the JRC/JCS index characterization of discontinuities are utilized here. The proposed criterion contrasts with the Hoek and Brown criterion, which primarily describes the strength of intact rock, with adjustments for jointed or crushed media.

Examples of discrete modelling of excavations in jointed media are given, using both physical and numerical models as examples. Particular attention is focused on the discrepancy between the behaviour of jointed media and the attempts to simulate these by means of continuum models. Tensile opening of joints and hysteresis on unloading due to shear are cited as reasons for the discrepancy. Examples of recent discrete element modelling using the UDEC method are described, to illustrate the prediction of the disturbed zone around tunnels.

KEY WORDS

Tunnels, boreholes, failure modes, shear strength, joints, dilation, scale effects, physical models, numerical modelling.

4th Manuel Rocha Memorial Lecture Lisbon, 12th October 1987

1. INTRODUCTION

There are three specialized disciplines of rock engineering that provide us with fascinating glimpses of the way rock behaves around underground openings. The three areas employ different specialists who do not often have the opportunity of communicating their different experiences to each other. Their employees have entirely different goals, yet their common interest is excavation stability in rock.

Table 1. Three categories of openings and their failure modes

Category	Rock characteristics	Failure modes*
1. Deep boreholes (Oil Industry)	Sedimentary rocks. Low intact strength. High Stress.	Shear failures, lamination buckling, «plastic» yielding
2. Deep mines (Mining Industry)	Massive, brittle rocks. High intact strength. High stress	Extension failures, rock bursting, slabbing, buckling
3. Shallow tunnels (Civil, Transport)	Jointed, altered rock, low mass strength, low stress levels	Extension, and shear failures on pre-exiting discontinuities, rotational failures

* Failure mode descriptions deliberately simplified.

2. OBSERVED FAILURE MODES

(i) *Deep Boreholes*

Recent research efforts, funded mainly by international oil companies, have thrown light on the possible failure mechanisms around deep boreholes. The subject is far from closed. However it already appears likely that failure does not initiate at the borehole wall but somewhere inside the wall (Maury, 1987). Carefully instrumented experiments have also shown that the peak tangential stress levels occur well away from the wall (Bandis et al. 1987). Due to the disturbed and partly failed zone, the effective modulus of the rock is lower at the wall of the borehole than within the surrounding material.

1987

Nick Barton[1]

Rock Mass Classification and Tunnel Reinforcement Selection Using the *Q*-System

REFERENCE: Barton, N., **"Rock Mass Classification and Tunnel Reinforcement Selection Using the *Q*-System,"** *Rock Classification Systems for Engineering Purposes, ASTM STP 984,* Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, 1988, pp. 59–88.

ABSTRACT: This paper provides an overview of the *Q*-system and documents the scope of case records used in its development. A description of the rock mass classification method is given using the following six parameters: core recovery (RQD), number of joint sets, roughness and alteration of the least favorable discontinuities, water inflow, and stress-strength relationships. Examples of field mapping are given as an illustration of the practical application of the method in the tunneling environment, where the rock may already be partly covered by a temporary layer of shotcrete. The method is briefly compared with other classification methods, and the advantages of the method are emphasized.

KEY WORDS: rock mass, classification, tunnels, rock support, shotcrete, rock bolts jointing

This paper provides an analysis of the *Q*-system of rockmass characterization and tunnel support selection. The 212 case records utilized in developing the *Q*-system (Barton et al, 1974) are reviewed in detail, so that application to new projects can be related to the extensive range of rock mass qualities, tunnel sizes, and tunnel depths that constitute the *Q*-system data base.

Ultimately, a potential user of a classification method will be persuaded of the value of a particular system by the degree to which he can identify his site in the case records used to develop the given method. The most comprehensive data base of the seven or eight classification systems reviewed is utilized in the *Q*-system. This body of engineering experience ensures that support designs will be realistic rather than theoretical, and more objective than can be the case when few previous experiences are utilized to develop a support recommendation.

Classification Systems Currently in Use

Table 1 is an abbreviated listing of most of the rock mass classification systems currently in use internationally in the field of tunneling. These are:

- *Terzhagi (1946) Rock Load Classification*—This has been used extensively in the United States for some 40 years. It is used primarily to select steel supports for rock tunnels. However, it is unsuitable for modern tunnelling methods in which rock bolts and shotcrete are used.
- *Lauffer (1958) Stand-Up Time Classification*—This introduced the concept of an unsupported span and its equivalent stand-up time, which was a function of rock mass quality. It appears excessively conservative when compared with present-day tunneling methods.

1988

SOME ASPECTS OF ROCK JOINT BEHAVIOUR UNDER DYNAMIC CONDITIONS

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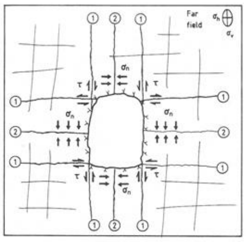
SUMMARY

Data from dynamic shear tests on rock joints are reviewed. Cyclic tests, single high velocity events, and stick-slip shear tests are included in this review. None of these tests provide an entirely satisfactory simulation of damageing dynamic loading, in which shear is accumulated in one direction during successive cycles. Case records of earthquake effects on mines and tunnels in jointed rock are discussed. Accumulation of shear, causing instability and permeability enhancement, may occur when jointing is under combined shear and normal stress. Such would be the case for steeply dipping joint structures in an anisotropic stress field, or for joints that interset tunnel perimeters in non-radial directions. Reinforcement strategies for jointed rock subjected to dynamic loading are suggested. A method of constitutive modelling based on the JRC (mobilized) concept is suggested for modelling cyclic shear with accumulated displacement and roughness damage.

INTRODUCTION

Rock joints beneath a slope or surrounding a tunnel, are acted on by shear and normal stress components. These are caused by the virgin or induced principal stresses and their relation to the orientation of the joints. If we first consider very simple examples (Figure 1) it is easy to imagine the different effects of dynamic loading. Joints that are under the influence of a shear component (τ) will tend to accumulate shear during dynamic loading, while those that are under the influence of only a normal component (σ_n) will tend to cycle (shear) back and forth.

Experimental studies designed to simulate some of the effects that can be experienced under dynamic loading are of three principal types; cyclic (shear reversal) tests, single high velocity shear in one direction, and stick-slip type experiments. The picture that evolves from a review of experimental data is somewhat confusing. Part of the problem is the difficulty of performing realistic tests. When considering the stability of the two structures illustrated in Figure 1, it is tempting to conclude that small amplitude, high frequency cyclic shear tests as often performed, will have little relevance. A shear test that accumulates shear in one direction, with limited shear reversal on each cycle would seem to be of most relevance. "Single shot", high velocity dynamic tests with shearing in only one rapid event also fall short of reality.



Idealized jointing surrounding a tunnel

Figure 1a
Predominance of normal or shear stress determines the joint behaviour under dynamic loading.

1988

The Discontinuum Approach to Compaction and Subsidence Modelling as Applied to Ekofisk

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ABSTRACT

The Ekofisk Centre in the North Sea has undergone unexpected seabed subsidence involving 150 km³ of underlying rock and sediments over an area of 50 km². NGI was engaged by the Norwegian Petroleum Directorate to perform independant studies of the factors involved in the subsidence, and of the implications of the compaction. NGI's studies included laboratory tests of the jointed reservoir chalk, numerical continuum modelling using the CONSAX code and discontinuum modelling using UDEC. In the final studies performed a special joint subroutine was incorporated in UDEC so that the effects of compaction on joint apertures and conductivity could be investigated. The studies showed that the steeply dipping conjugate joints in the 300 m thick reservoir were probably undergoing shear during the approximately one-dimensional compaction. Joint shear and dilation were admissible in this uniaxial strain environment, due to shrinkage and pore collapse of the matrix between the joints caused by the 20 MPa drawdown in pore pressure. The 3 km of overburden shale was also modelled as a discontinuum and demonstrated the possibility of shear along bedding planes and sub-vertical jointing. Discontinuum models showed larger ratios of subsidence to compaction than continuum models due to such shear mechanisms.

1. INTRODUCTION

Those working on the Ekofisk problem are frequently asked the question; why was it not foreseen? A 20 MPa (or more) reduction in pore pressure in a reservoir of large area (50 km²) at no more than 3 km depth must have been expected to cause compaction and surface subsidence?

The questions are well grounded. The answer is at least partly based on an insufficient understanding of a complex material such as chalk at that time.

1988

Cavern design for Hong Kong rocks

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Abstract

The paper describes an integrated Q-system and discrete element design philosophy that can ensure the safe design of very large caverns in Hong Kong's excellent quality granites and welded tuffs. Principal cavern reinforcement methods should consist of fully-grouted rock bolts and fiber reinforced shotcrete. Predicted loading of reinforcement can be checked with numerical sensitivity analyses. The principal activities required to obtain the all-important input data for the empirical and numerical analyses will be described. These include stress measurement by hydraulic fracturing, cross-hole seismic tomography to identify fault zones and joint swarms, characterization of joints in drill core to obtain input for Q-system and discrete element (UDEC-BB) modelling, and follow-up mapping during construction to confirm designs.

Introduction

Slope stability problems in Hong Kong's weathered granites give a misleading picture of the potentially excellent rock qualities available for underground construction. Very large span caverns can be constructed at moderate cost to produce valuable additions to Hong Kong's high priced real estate. The especially favourable economy of large spans should be utilized to the full, to gain greatest benefit from the all-important area/volume ratio that favours minimum supported cavern surface area and maximum cavern volume.

How can one be so sure that large span caverns can be safely constructed and utilized in Hong Kong's granites and volcanics? The initial answer to this important question can be found in NGI's Q-system of rock mass classification and cavern support selection (Barton et al. 1974). Caverns of 20 to 30 m span have been successfully excavated and safely utilized in rock masses of equivalent quality to Hong Kong's granites and welded tuffs. In fact they have been successfully excavated and safely utilized in markedly poorer rock qualities than those available in Hong Kong's underground terrain.

The quality of Hong Kong's rock according to the Q-system

Case record statistics

More than 200 case records were utilized in the original development of the Q-system. Since that time NGI has designed almost 1000 km of tunnels and numerous large caverns based on this method. The level of precedent is therefore high, and it is apparently being added to by successful application in many other countries.

1989

Review of predictive capabilities of JRC-JCS model in engineering practice

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ABSTRACT: The database used in developing the Barton-Bandis joint model is reviewed. It is shown how tilt testing to obtain JRC is extrapolated both in terms of stress and sample size. Field measurement of JRC is demonstrated, and relationships with J_r in the Q-system are developed. Constitutive modelling of shear stress-displacement, dilation and shear reversal are also described.

1. INTRODUCTION

The JRC-JCS or Barton-Bandis joint model started inconspicuously some 20 years ago as a means of describing the peak shear strength of more than 200 artificial tension fractures. These were developed with a guillotine in various weak model materials, which had unconfined compression strengths (σ_c) as low as 0.05 MPa.

Linear plots of peak friction angle (arctan τ/σ_n) versus peak dilation angle (d_n) indicated the following simple expression:

$$\tau = \sigma_n \tan(2d_n + 30°) \quad (1)$$

It was found that the peak dilation angle was proportional to the logarithm of the ratio (σ_c/σ_n):

$$d_n = 10 \log(\sigma_c/\sigma_n) \quad (2)$$

By elimination, the following simple form was obtained

$$\tau = \sigma_n \tan[20 \log(\sigma_c/\sigma_n) + 30°] \quad (3)$$

Thus the first form of the "JRC-JCS" model was actually the "20 - σ_c" model, where the roughness coefficient (JRC) was equal to 20 for these rough tension fractures. The joint wall strength (JCS) was equal to σ_c (the unconfined compression strength). The original form of the equation is therefore perfectly consistent with todays equation:

$$\tau = \sigma_n \tan[\mathrm{JRC} \log(\mathrm{JCS}/\sigma_n) + \phi_r] \quad (4)$$

and represents the three limiting values of the three input parameters i.e.

JRC = 20 (roughest possible joint without actual steps)

JCS = σ_c (least possible weathering grade, i.e. fresh fracture)

ϕ_r = ϕ_b (fresh unweathered fracture with basic friction angles in the range 28½ to 31½°)

In addition, the small size of the samples (60 mm length) meant that both JRC and JCS were truely laboratory scale parameters and would nowadays be given the subscripts JRC_0 and JCS_0 (Barton et al. (1985), to distinguish them from the scale-corrected full scale values JRC_n and JCS_n (see later).

2 PEAK STRENGTH OF ROCK JOINTS AND ITS PREDICTION

Figure 1 illustrates the results of direct shear tests, on 130 rock joints, reported by Barton and Choubey (1977). Eight rock types were represented. The statistics for JRC, JCS and ϕ_r are given in Figure 2. The mean values of these parameters

JRC = 8.9 JCS = 92 MPa ϕ_r = 28°

were used as input parameters to derive the central strength envelope in Figure 1.

A key aspect of this study was the discovery that self-weight tilt testing,

1990

GEOTECHNICAL DESIGN

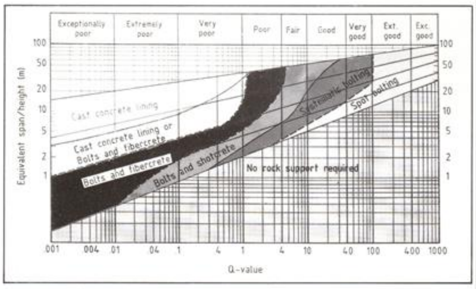

Figure 1. Synthesis of Q-system tunnel support recommendations updated to include fibre-reinforced shotcrete (after Grimstad et al 1986).

The rapid development of Norway's hydroelectric potential during the 1970s and early 80s, and major road tunnels during recent years, has had a significant influence on tunnel design work undertaken by the Norwegian Geotechnical Institute (NGI). Field investigations for some 1,200 km of tunnels, support design for 900 km and construction supervision for 600 km (both drill-and-blast and tunnel-boring machine) have set their mark on both the level of experience and on the design methods developed at NGI.

Norwegian tunnels are beset by diverse problems during drivage, such as occasional major water inflows, stress-induced slabbing and rock-bursting, unstable clay-bearing jointed rock with notable joint persistence, major faulting and zones of severe swelling clay. This great variability is reflected in the huge numerical range of rock qualities (from 0.001 to 1000), described in the NGI Q-system now used world-wide. Figure 1 shows a recent (1986) update and a new feature - 'S(fr)'.

Q-SYSTEM B AND S(FR) REINFORCEMENT

Rock bolts and shotcrete as tunnel support (the B + S method) have been used in very many countries for several decades, but few would dispute the pioneering work performed in Scandinavia in the developments made with these products. In particular, robotically-applied, wet-process, fibre-reinforced shotcrete (S(fr)) has caused a revolution in support of difficult ground and has completely superceded the use of mesh-reinforced shotcrete (S(mr)) in Norway. As a result of this, a Q-system chart (Figure 1) developed by Grimstad et al (1986)[1] already incorporated this product 5 years ago, following some six to eight years of excellent experience with S(fr) both in Norway and Sweden.

'B + S(fr)', signifying systematic bolting and fibre-reinforced shotcrete, is a flexible combination seldom matched by NATM support methods which often involve mesh-reinforced shotcrete, but can result in high labour costs and cause a "shadow" effect under spraying. Moreover, the initially unreinforced shotcrete gives poor protection to mesh-fixing personnel.

In poor ground, or in a major excavation such as the 60m span Olympic ice-hockey cavern to be described later, it is usual to check the performance of the B + S(fr) support by convergence measurements or by MPBX extensometer installation. B + S(fr) has been used for at least a decade and gives superior advance rates and personnel safety. It is also the major component of final rock support in large caverns and tunnels through difficult ground.

ROCK MASS VARIABILITY

To those familiar with the Q-system method of rock-mass classification[2] the following six numbers (selected from hundreds of thousands of alternative combinations) communicate a significant amount of information on the quality (or otherwise) of the rock mass:

$$Q = \frac{80}{6} \times \frac{2}{3} \times \frac{0.66}{1} = 6 \quad (1)$$

$$\text{where } Q = \left(\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right)$$

(RQD = rock-quality designation, J_n = joint-set number, J_r = joint-roughness number, J_a = joint-alteration number, J_w = joint water-reduction factor, SRF = stress reduction factor.) These numbers represent a valid description of the rock mass at a given location in a tunnel, and are associated with a specific need for tunnel reinforcement, for example B (1.5m c/c) + S(fr) 5 cm for a 15-m span road tunnel.

When surface mapping, or logging drill core, or when recording large amounts of geotechnical data in an advancing tunnel, it has been found convenient to record Q-system data in histogram form such as in Figure 2. This gives a good indication of rock-mass variability, and early data can be combined with subsequent data, and manipulated in PC-based spreadsheet format. In the case of the ice-hockey cavern cited earlier, sets of histograms were produced from preliminary mapping in existing, nearby excavations, and subsequently combined with the results of Q-logging of 250-m of drill core[3]. This data base provided cavern support designers with preliminary indications of rock reinforcement needs. The system has since been used for mapping the distribution of Q-values in the arch of the huge cavern and confirming the prognoses obtained from geophysical studies. These studies are described later.

PREDICTING ADVANCE RATES

Experience in using the Q-system within NGI's group of engineering geologists is very extensive due to 600 km of rock-reinforcement supervision and design, including more than 150 km of hard-rock TBM tunnels. An interesting synthesis of experience with two drill-and-blast road tunnels of 10-m and 14-m spans driven in the early and late eighties respectively, is shown in Figure 3. The diagram shows the tunnel drivage rate in m/week/advancing face as a function of the Q-value and the corresponding rock reinforcement or temporary support method. The extreme range of conditions encountered, from swelling siltstones (Q = 0.002) to massive basalts (Q = 80), also involved advance through at least a dozen rock types in the two tunnels combined.

The more rapid advance with B + S(fr) compared to cast concrete is clearly seen in Figure 3. The driving rate represents progress with both excavation and temporary support of the full tunnel cross-sectional areas of 70 and 90 m^2. Some 60% to 100% of the final support quantities are incorporated in the temporary

1991

Scale effects or sampling bias?

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ABSTRACT: A wide range of scale effects and potential scale effects in rock engineering are reviewed. These include uniaxial compression strength, joint roughness and shear strength, conductivity-shear coupling, shear stiffness, failure modes, and stress-strain behaviour. Sampling bias and sampling disturbance effects may be responsible for incorrect conclusions concerning some of the apparent scale effects.

1 INTRODUCTION

Numerous potential scale effects are evident in rock mechanics. Many are real effects, but many are undoubtedly caused by the difficulties in obtaining representative samples. Large samples are more easily damaged and may therefore demonstrate lower strength or stiffness since the larger sampling size tends to include more "flaws", a fundamental scale effect would of course be expected; however, it may be exaggerated out of proportion by the sampling preparation, extraction or testing process. In this paper a fairly wide ranging look will be taken at many of the areas where scale effects are expected or suspected. The author's personal experiences lend support to many of the interesting observations made by authors to this workshop on Scale Effects in Rock Masses.

2 THE DILEMMA OF STRESS EFFECTS

It may be wise to start this review by pointing out one scale effect problem which may never be resolved, before going on to more tangible problems which have been explained or show potential for being explained.

Compilation of direct shear test data for rock joints tested under low stress levels, show very large variations in shear strength, while compilations of high stress triaxial data for faulted rock specimens show relatively small variations in shear strength. An equally wide range of rock types may have been tested in each case.

Figure 1 illusrates these different ranges of shear strength. It also illustrates the approximate ratio of test sizes: small-finger-size cylinders may represent apparatus test limits when measuring the triaxial shear strength of "faulted rock" specimens at normal stress levels in the kilobar range of stresses. This reduced specimen size is not showing a reversed scale effect. It is the enormous stress that is removing the effects of variable rock strength and discontinuity roughness, otherwise seen in tests on discontinuities in rock.

The stippled envelopes shown in Figure 1 indicate the potential scale effect for rock joints at low (engineering) stress levels. The scale effect at kilobar stress levels can only be inferred from geotectonics; but it is presumeably much less marked than the scale effect we as rock engineers must live with in engineering design.

3 SCALE EFFECT ON UNIAXIAL COMPRESSION STRENGTH

This fundamental index of rock strength has been the subject of numerous scale effect investigations over the last 30 to 40 years. A useful compilation of data is that given by Lama and Gonano (1976),

1990

NORWEGIAN METHOD OF TUNNELLING

For a country with only 4 million inhabitants, Norway has quite an unusual level of tunnelling activity. Tunnel construction in the civil sector has been especially high in the last 15 years, with 8 of these years seeing more than 4 million m^3 of tunnel and cavern excavation. Several years have seen more than 3 million m^3 of water tunnel construction for hydropower per year (mostly in the late 1970s), and many years with more than 1 million m^3 of road tunnels (2 million m^3 in 1990). In 1990, 32 km of hard rock TBM tunnels were driven, mostly at Statkraft's Svartisen hydroelectric project. One of the largest Norwegian tunnelling contractors, Selmer A/S has constructed more than 20 km of tunnels per year during five of the last seven years, with a total of 31 km in 1991. A/S Veidekke, who in a joint venture with Selmer A/S were responsible for the construction of the 62m span Olympic ice hockey cavern, also have a very impressive record of tunnelling and underground construction. The Veidekke Group excavated a total of 95 km of tunnels in 1987 and 1988 and have averaged more than 30 km per year since 1987. It has been estimated that 4,500 km of tunnels have been constructed in Norway since 1970.

Norwegian tunnelling is also blessed with several very experienced Consultants whose role in pre-investigations, design and tender document preparation are significant in many ways. Norconsult, Norpower, Berdal Strømme, Grøner, Noteby, Fortifikasjon, Geoteam, SINTEF and NGI are notable examples of organisations with extensive tunnelling and underground construction experience, built around numerous hydropower, petroleum storage and road tunnel projects both in Norway and abroad.

A major Norwegian tunnelling development over the past 12 to 15 years has been high capacity wet process shotcreting equipment which allows steel fibre reinforcement (typically 30 mm x 0.5 mm) to be applied by robot at the tunnel face 15 to 20 m ahead of the rig, in volumes of 15 to 25 m^3 per hour. At present, some 50 to 60,000m^3 of fibre reinforced shotcrete are sprayed each year in Norway; one company, Entreprenørservice A/S, being responsible for 30,000m^3 in 1991.

Another major player in the shotcreting field, besides Selmer and Veidekke, is Robocon International who did pioneering work in the development and application of fibre reinforced shotcrete in the early 80s, and are currently

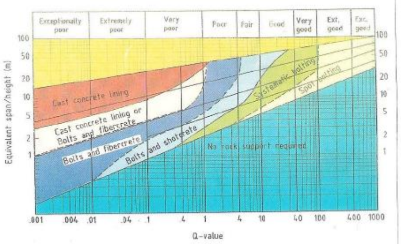

Figure 1. Simplified diagram for design of rock support based on the Q-system (Grimstad et al., 1986)[2].

Selmer, Veidekke and Statkraft, have between them constructed one half of Norway's 200 underground hydroelectric power stations, or more than one quarter of the world's total of approximately 400. In a current hydroelectric project in Northern Norway (Statkraft's Svartisen Project), more than 40 km of hard rock TBM tunnels have been constructed in record time in hard gneisses, diorite, quartzite, marbles and schists with compressive strengths of 120 to 300 MPa. High cutter loads of 32 tons and high cutter head power (3150 HP in 4.3 m and 5.0 m diameter Robbins machines) have given average weekly advance rates of 119 to 200 m in 100 hour weeks for the five machines. Best results of 61.2 m in a shift, 90.2 m in a day, 415 m in a week and 1176 m in a month were achieved in the 36 km driven during a 2 year period from 1989 to 1992. At the Meråker hydroelectric project the joint venture Veidekke/Eeg Henriksen has recently set a new world record of 426 m in one week, using a Robbins HP TBM of 3.5 m diameter.

This article describes key aspects of Norwegian tunnelling technology to assist potential users of these methods in deciding between the so-called New Austrian Tunnel Method (NATM) and the Norwegian Method of Tunnelling (NMT). A comparison of methods and typical areas of application is given. In assembling key aspects of NMT, the authors have materials, and contractual aspects to give readers a glimpse of the level of technology available. A case record of NMT used in difficult tunnelling conditions is given at the end of the article.

NMT AND NATM - WHAT ARE THE DIFFERENCES?

Despite the comment by an experienced NATM pioneer that "it is not usually necessary to provide support in hard rocks", Norwegian tunnels require more than 50,000m^3 of fibre reinforced shotcrete and more than 100,000 rock bolts each year. Two major tunnelling nations, Norway and Austria, have in fact long traditions in using shotcrete and rock bolts for tunnel support, yet there are significant differences in philosophy and areas of application for NATM and NMT. To start this brief review, it may be pertinent to first state what appear to be the major differences between NATM and NMT.

NATM appears most suitable for soft ground which can be machine or hand excavated, where jointing and overbreak are not dominant, where a smooth profile can often be formed and where a complete load bearing ring can (and often should) be established. Monitoring appears to play a significant part in deciding on the timing and extent of secondary support. (Using the surrounding ground as the main loading component is not an exclusive NATM philosophy. It is essential practice and is often inevitable!)

NMT appears most suitable for harder ground,

1992 (Part I)

NORWEGIAN METHOD OF TUNNELLING

Continuation from June issue

In June *WT* we introduced the idea of The Norwegian Tunnelling (NMT) as a hard rock rival to NATM. In this second instalment, Dr Nick Barton and his colleagues further explore its application against the background of Norway's contract system.

NORWEGIAN CONTRACTS

An inexperienced tunnel Owner who describes the desired final product with its concrete lining and leaves all risks to the Contractor in a Turnkey or Lump Sum, Fixed Price contract invites high costs, disputes and legal actions. Another extreme, also inviting high costs which this time maximises the Owner's risk and minimises the Contractor's risk is the Cost Reimbursement type of contract. Figure 10 illustrates where Norwegian practice lies (Kleivan, 1988)[13]

The contract system used in Norway which has a 20 year track record of low costs and few disputes is based on tender documents that reflect the unit prices for the equipment, methods and materials most likely to be needed for tunnelling through the investigated rock. An integral part of this system is a tender document that thoroughly describes the geological and geotechnical investigations, giving the Contractor a fair idea of likely rock conditions, rock support needs and details of all investigations performed. The Owner utilises engineering geologists from his own or from his Consultant's organisation who are experienced in tunnelling work, for this important task.

The tender documents therefore represent the best judgement of the Owner and his Consultants on most likely conditions. They list the different types and amounts of support work that are to be included in the tender sum, and request alternative unit prices for instance for driving a pilot heading, for probe-drilling, and for various pre-grouting strategies in case these are needed for parts of the tunnel. The required support work is divided into two main categories: that to be executed at the tunnel face, and that to be executed behind the tunnel face that does not delay the progress of the tunnel. Unit prices are also given for all delays and idle time for workers and for equipment, and of course for running costs for the workers' camp and for administration.

When rock problems are encountered, the Owner, Contractor and Consultant together choose the most suitable and practical methods for coping with the problems. The Contractor who has correctly priced different types of support will be able to give his best advice

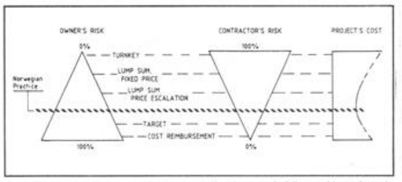

Figure 10. Risk sharing according to type of contract and assumed influence on project cost (Kleivan, 1988)[13]

without concern for "tactical motives". The Owner pays for the technically correct solution, no more and no less. (Aas, 1986)[14]

The idea of the Norwegian contract system is to help create a cooperative attitude among parties involved when difficult and unexpected conditions are encountered, yet still maintain the advantage of a full competition at the tender stage since all eventualities have been priced. The need for the Owner to employ consultants and engineering geologists who have significant experience is fundamental to the proven success of the Norwegian tunnel contract system.

Great emphasis is laid on avoiding unnecessary damage to the rock mass due to careless blasting by the Contractor. In the tender documents the Owner asks for alternative unit prices concerning restricted

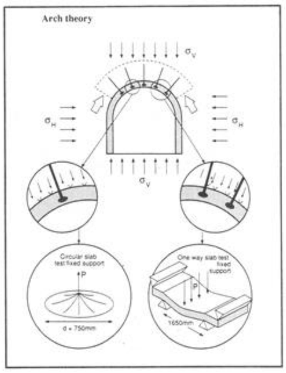

Figure 11. Analogy between a bolted shotcrete lining and the large scale load simulation rigs: the one-way slab test and the circular slab test. (Torsteinsen and Kompen, 1986)[17]

1992 (Part II)

Geotechnical Core Characterisation for the UK Radioactive Waste Repository Design

Geotechnische Kernmaterial Beschreibung für den Entwurf von Lagern für radioaktiven Abfall in Grossbritannien

Description de carottes géotechniques pour le dimensionnement d'aires de stockage enfouis et permanent pour déchets radioactifs

by
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The NGI methods of characterising joints (using JRC, JCS and ϕ_r) and characterising rock masses (using the Q-system) are being utilised extensively in a current geotechnical consultancy project for UK Nirex Ltd. Present geotechnical characterisation activities include the logging of six kilometres of 100mm drill core from cored drill holes of up to 1,960m depth.
Preliminary rock reinforcement designs (systematic bolting and unreinforced or fibre-reinforced shotcrete) are derived from the Q-system statistics, which are logged in parallel with JRC, JCS and ϕ_r. The UDEC-BB modelling provides a check on the performance of the proposed excavations with Q-system reinforcement, giving predicted bolt loads and rock deformations, together with joint shearing and hydraulic apertures to better define the disturbed zones.

NGI's Methoden der Trennflächenbeschreibung (unter Gebrauch der Q-Methode) sind wesentliche Bestandteile des gegenwärtigen geotechnischen Consultingprojektes für UK Nirex Ltd. Die geotechnischen charakterisierenden Aktivitäten beinhalten die Beschreibung von 6 km, 100 mm Kernmaterial aus einer Tiefe bis zu 1960 m.
Das vorläufige Sicherungskonzept (systematisches Ankern und unverstärkter oder fiberverstärkter Spritzbeton) beruht auf einer statistischen Q-system Analyse. Diese wird parallel mit der Registrierung von JRC, JCS und ϕ_r durchgeführt. Die UDEC-BB Simulationen erlauben eine überprüfung des Verhaltens der geplanten und Q-System gesicherten Kavernen. Berechnungen der Ankerlasten, Felsdeformationen, und Scherdeformationen entlang der Trennflächen und der hydraulischen Kluftöffnungen erlauben eine verbesserte Beschreibung der Auflockerungszone.

Les méthodes NGI pour caractériser les joints (utilisant JRC, JCS et ϕ_r) et les massifs rocheux (utilisant le système Q) sont utilisées à grande échelle dans un project de consultation géotechnique pour UK Nirex Ltd. Les activités de description en cours incluent l'enregistrement de 6 kilomètres de carottes de 100 mm extraites de trous de forages d'une profondeur jusqu'à 1950 m.

Les dimensionnements d'armement du rocher (ancrages systématiques et béton projeté non-armé ou armé de fibres) sont dérivés des statistiques du système Q, enregistrées en parallèle avec les paramètres JRC, JCS, et ϕ_r. Les modèles analytiques UDEC-BB permettent de vérifier le comportement de l'armement basé sur le système Q, et donnent forces d'ancrages, déformations du rocher, cisaillement du joint, et ouvertures hydrauliques afin de mieux définir les zônes remaniées.

INTRODUCTION

The NGI methods of characterising joints (using JRC, JCS and ϕ_r) and characterising rock masses (using the Q-system) are being utilised extensively in a current geotechnical consultancy project for UK Nirex Ltd. This organisation is responsible for the safe disposal of low and intermediate level radioactive waste in the UK. Present planning and site investigation is now focused at Sellafield in NW England where extensive deep drilling, downhole testing, geological and geophysical investigations are being progressed.

The NGI/WS Atkins/Taywood Engineering work as Geotechnical Consultants to UK Nirex Ltd has included field mapping and core logging, using newly developed geotechnical logging charts which combine Q-system parameter histograms with more detailed joint and rock mass descriptions suitable for use in the distinct element code UDEC-BB. Extensive numerical analyses of access tunnel and cavern excavation response are being carried out to investigate rock reinforcement requirements and the extent of the disturbed zones.

GEOTECHNICAL LOGGING CHART

As a first step in the rock mechanics design process, data of relevance to cavern and tunnel design studies are collected from field mapping and from current logging of some 6 km of oriented drill core. The chart used in the field mapping is illustrated in Fig. 1.

Key to geotechnical logging charts

Q-value (Barton, Lien and Lunde 1974)

The Q-value is a measure of the stability of excavations in a rock mass. The Q-value is

1992

COMPARISON OF PREDICTION AND PERFORMANCE FOR A 62m SPAN SPORTS HALL IN JOINTED GNEISS

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ABSTRACT

The feasibility of excavating caverns of very large span for underground location of a nuclear power station in Norway was investigated in the early 1970s. In the end, the 1994 Winter Olympic Games has provided the necessary impetus for utilising very large engineered rock caverns. The 62m span Olympic Ice Hockey cavern has been constructed in Gjøvik, Norway. It is located in jointed gneiss of average RQD = 70% and has a rock cover of only 25 to 50m, thus posing challenging design problems. The investigations prior to construction included two types of stress measurements, cross-hole seismic tomography, special core logging, Q-system classification and numerical modelling with UDEC-BB. Predicted maximum deformations were 4 to 8mm; surprisingly small due to the high horizontal stresses recorded. Extensometer (MPBX) installations from the surface prior to construction, precision surface levelling and MPBX installed from inside the cavern give a combined measure of maximum deformations in the range 7 to 8mm with the 62m span fully excavated, and three adjacent caverns for the Postal Services also completed.

INTRODUCTION

During the 1970s, NGI performed a series of siting studies and some *in situ* testing, to investigate the feasibility of underground siting of nuclear power plants. Special attention was focused on the need for a reactor containment cavern with a hemispherical domed arch of at least 50m diameter. The Norwegian State Power Board (Statkraft) and subsequently also the Swedish State Power Board (Vattenfall) funded parallel theoretical studies of large span caverns.

The Norwegian Geotechnical Institute (NGI) first performed physical models of large spans in jointed rock, studying the effect of medium and high horizontal stress levels and the effects of various jointing patterns. Comparisons were also made with continuum FEM studies. Today, fifteen years later, we would have used discrete element methods such as UDEC. A brief review of some of the findings of these earlier studies will be used as an introduction to the real life problems subsequently encountered at Gjøvik, where a 62m span cavern has been successfully engineered for the 1994 Winter Olympic Games which will be centred at Lillehammer.

Prior to specific siting of large caverns, whether for nuclear reactor vessels or for Olympic Ice Hockey, estimates have to be made of stress levels and rock properties. Concerning stress levels, we elected to investigate low, medium and high stress as follows:

$$\sigma_h/\sigma_v = 1/3$$
$$\sigma_h/\sigma_v = 1.0$$
$$100/z + 0.3 \leq \sigma_h/\sigma_v \leq 1500/z + 0.5$$

The latter (where z is depth in metres) is based on measured data reviewed by BROWN & HOEK (1978). The actual level chosen in the third case was k = 20 at 25m depth and k = 6 at 100m depth, i.e., a trapezoidal distribution of stress, within the above range of observations.

Figure 1 illustrates the FEM results obtained with an assumed rock cover of only 25m, E = 14 GPa, υ = 0.1 and ***plane stress*** conditions (equivalent to the two-dimensional physical models). For stage 2 of the excavation (roughly equivalent to the Olympic Ice Hockey cavern dimensions) maximum vertical deformations for the three stress cases were 2.7mm, 1.5mm and (-) 10.8mm (i.e., heave). When the rock cover was increased to 50m, the isotropic case (k = 1.0) showed a maximum downward deformation of 2.9mm, i.e., the tendency for heave is of course reduced.

The physical models, which are described in detail by BARTON & HANSTEEN (1979) consisted of 20,000 blocks of discretely fractured model material, with two regular joint patterns of constant dip. These two-dimensional models were loaded by gravity and by vertical boundaries that resemble today's numerical roller boundaries. Figure 2 illustrates two of the physical model results under stress levels equivalent to those described above. It should be noted that the tension fracturing technique gives the two sets of joints (one continuous, the other discontinuous) unusually high values of JRC (joint roughness coefficient). Stability is therefore ensured in the arch of the models, but not in the walls.

The above numerical and physical models demonstrated the possible favourable nature of near-surface siting for large caverns at least from the point of view of possible high horizontal stress levels and reduced deformation. A compromise would need to be

1992

Geotechnical Predictions of the Excavation Disturbed Zone at Stripa

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Abstract

The 3m wide by 2m high by 50m long validation drift at Stripa surprised the site characterisation and validation project investigators, by the limited amount of water inflow compared to borehole measurements and predictions. Rock mechanics characterisation and laboratory and field tests performed by NGI are reviewed in an attempt to explain the reduced inflows. Discrete element UDEC-BB modelling was used to predict the effects of excavation induced disturbance in two-dimensional models. Full coupling of hydro-mechanical effects was incorporated in some models and shown to be important compared to mechanical modelling.

Prédictions géotechniques de la zone perturbée autour d'une excavation à Stripa

Résumé

La galerie d'accés, de trois mètres de large, deux mètres de haut et cinquante mètres de long, à Stripa, utilisée pour les validations, a surpris les enquêteurs chargés de la charactérisation du site et du projet de validation, par le faible débit d'eau comparé aux prédictions et mesures de sondage. La charactérisation de mécanique des roches et les tests in situ et au laboratoire effectués par NGI sont repassés en revue afin d'expliquer les débits réduits. Une modélisation par la méthode des éléments discrets UDEC-BB a été effectuée pour prédire les effets de perturbation induite par l'excavation dans des modèles à deux dimensions. Le couplage des effets hydro-mécaniques a été incorporé dans certains modèles et se révèle important comparé aux cas où la modélisation est purement mécanique.

1992

Physical and discrete element models of excavation and failure in jointed rock

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Abstract:
Physical models of single joints, of rock masses and of model excavations in rock can sometimes provide important insights into potential behaviour and failure modes in real rock masses. They can also provide verification or validation of computer codes. However, where failure or large strains are concerned, the computer models that are based on isotropic continuum behaviour will usually fall short of reality. Discrete element models with realistic constitutive laws for the joints may, on the other hand, provide good simulations of the physical behaviour seen in physical models, and therefore appear likely to be able to simulate or predict real behaviour. An example of a virtual validation of the UDEC-BB discrete element code with results from a well instrumented large excavation are given to illustrate this point. A unifying theme that runs through the article is the importance of shear induced dilation and associated joint roughness. This prime parameter helps rock masses to accommodate the "key" blocks and "plastic" zones that we sometimes all to eagerly predict when ignoring the rock block and rock mass interlock effect. The exact opposite is experienced with the low-J_r and high-J_a discontinuity that causes rock support needs to escalate due to non-dilatant or even contractile behaviour, when such a feature tries to resist but actually causes failure. The interlock of the surrounding rock joints may be seriously compromised by such features, and ravelling may result.

1. PHYSICAL MODELS OF ROCK JOINTS

Physical models of rock joints, rock masses and excavations in rock have much to offer in rock mechanics. As a starting point, some of the things we have learned from studies of model rock joints will be considered.

Direct shear tests of tension fractures that were developed in a range of weak model materials are shown in Fig. 1. What appeared at the time to be alarmingly high peak friction angles (ϕ_p) proved later to be a fundamental feature of non-planar rock joints. It appears that if a shear test is conducted at low enough normal stress, ϕ_p may tend to be as high as 90°, as shown in the inset to Fig. 1.

As explained by Barton and Bandis (1990), the rough tension fractures depicted in Fig. 1 represent valid "end members" of the family called rock joints. In the terminology JRC, JCS and ϕ_r developed by Barton and Choubey (1977) to extrapolate tilt test results, the tension fracture has the highest possible JRC, JCS and ϕ_r values.

2. CONSTITUTIVE MODELS FOR THE SHEAR STRENGTH OF ROCK JOINTS

For the model tension fractures discussed above, linear plots of peak friction angle (arctan τ/σ_n) vs peak dilation angle (d_n) indicated the following simple expression:

$$\tau = \sigma_n \tan(2d_n + 30°) \qquad (1)$$

It was found that the peak dilation angle was proportional to the logarithm of the ratio (σ_c/σ_n) (compression strength/normal stress):

$$d_n = 10 \log\left(\frac{\sigma_c}{\sigma_n}\right) \qquad (2)$$

By elimination, the following simple form was obtained:

$$\tau = \sigma_n \tan\left[20 \log\left(\frac{\sigma_c}{\sigma_n}\right) + 30°\right] \qquad (3)$$

Thus the first form of the "JRC-JCS" model was actually the "20 - σ_c" model, where the roughness

1992

SUPPORT

Rock mass conditions dictate choice between NMT and NATM

The Norwegian Method of Tunnelling is most appropriate for drill+blast tunnels in jointed rock which tends to overbreak. ***Nick Barton*** *and* ***Eystein Grimstad****, Norwegian Geotechnical Institute, discuss the different applications of NMT and NATM, usually employed in driving soft ground tunnels.*

The NATM support design philosophy has been employed on numerous occasions for soft ground tunnelling[1]. In general, it has been used with great success. The soundness of an active design approach, sometimes called design-as-you-go (more correctly design-as-you-monitor), has been demonstrated by major cost savings compared to conventional, inflexible design approaches. However, it would be unfair to the NATM concept, and also incorrect, to refer to all tunnels that incorporate shotcrete and rockbolting in their method of construction as being 'driven by NATM', which appears to be occurring in some quarters.

NATM clearly cannot be the best or cheapest method for tunnels in extensively jointed, harder rock masses that are drill+blasted as opposed to machine excavated. Extensive overbreak (i.e. negative radii) frequently causes mesh reinforced shotcrete (S(mr)) and lattice girders to be impractical, time consuming and possibly unsafe. Such methods may also cause unnecessarily large concrete consumptions. For this reason, Norwegian tunnellers were only too ready to stop using mesh reinforcement and steel ribs within a few years of their developing wet process, steel fibre reinforced shotcrete (S(fr)) in place of the earlier S(mr) method. Commercial application of wet process S(fr) in Norway by 1978 caused S(mr) to fall out of use by about 1984[2].

Fibre reinforced shotcrete

Use of this revolutionary permanent reinforcement and final support method for jointed ground with overbreak since 1978 has increased from 60 000 to 70 000m³/year in Norway, close to the highest use in the world at present, despite Norway's small population. Robotic application 10 to 20m above, to the side of or in front of the operator, production rates of 10 to 25m³/h, low dust levels (rebound 5 to 10 per cent), secured rock bolting conditions in unstable ground, and no problems with uneven profiles and overbreak, have caused a revolution in driving rates and tunnelling costs.

Cast concrete lined sections for permanent support of fault zones and clay-bearing rock are virtually disappearing from use due to their cost and time constraints as compared to S(fr). Rib (rebar) reinforced shotcrete (RRS) with S(fr), unreinforced shotcrete (S) and bolting (B) are now used as permanent support in such zones at approximately half the cost of cast concrete. Similar advantages can be expected when S(fr) and RRS are used as permanent support in tunnels or caverns in soft jointed rocks and in over-consolidated fissured clays, such as London Clay.

The no-nonsense B+S(fr) Norwegian Method of Tunnelling (NMT) allows drill+blast driving rates of up to 40 and 70m a week in 75 hours and 110 hours a week tunnelling. These figures are achieved even when significant amounts of shotcreting and bolting are performed. Maximum rates of about 60 and 100m a week are achieved when there is only minor rock reinforcement.

Surprisingly, the use of shotcrete as the final lining of major tunnels in Scandinavia has gone relatively unnoticed. A recent survey of major tunnels with S(fr) as final support overlooked both Norway and Sweden in this respect, due no doubt to the commonplace use of these methods in Scandinavia which goes largely unreported[3].

There are in fact some 460km of main road tunnels in Norway which have stretches totalling 160km with S or S(fr) as approved final support, some of them subsea tunnels. An important point to remember is that the Norwegian Public Roads Administration is just as interested in maintenance free tunnels as its international counterparts.

No significant fibre corrosion

A common misconception is that S(fr) is unsuitable for long life, maintenance free tunnels, due to possible fibre corrosion. This is proving to be an unfounded worry, even in salt-water environments, provided that sensible precautions are taken. The key to success is good quality concretes.

High grade concretes with plasticisers, super-plasticisers, silica fume, slump killers and hydration control have extremely low water contents, permeabilities and porosities. Since the fibre is non-continuous, it does not suffer galvanic cell type corrosion as may occur with mesh reinforcement. Even the medium grade concretes such as C35 that were common with S(fr) application ten years ago do not show fibre corrosion in ten year old subsea tunnels. Convincing information on the environmental effects in such tunnels was

1) Areas of usual application:
Jointed rock giving overbreak; harder end of uniaxial strength scale (σ_c = 3 to 300MPa)
Clay bearing zones, stress slabbing
Q = 0.001 to 10 or more
2) Usual methods of excavation:
Drill-blast, hard rock TBM, machine excavation in clay zones
3) Temporary rock reinforcement and permanent tunnel support may be any of following:
CCA, S(fr)+RRS+B, B+S(fr), B+S, B, S(fr), S, sb, (NONE) (see key below and Fig 1)
• Temporary reinforcement forms part of permanent support
• Mesh reinforced shotcrete not used
• Dry process shotcrete not used
• Steel sets or lattice girders not used; RRS and S(fr) are used in clay zones and weak, squeezing rock masses
• Contractor chooses temporary support
• Owner/consultant chooses permanent support
• Final concrete linings are less frequently used, i.e. B+S(fr) is usually the final support
4) Rock mass characterisation for:
• Predicting rock mass quality
• Predicting support needs
• Updating of both during tunnelling (monitoring in critical cases only)
5) The NMT gives low costs:
• Rapid advance rates in drill+blast tunnels
• Improved safety
• Improved environment
CCA = cast concrete arches, S(fr) = steel fibre reinforced shotcrete, RRS = reinforced ribs of shotcrete, B = systematic bolting, S = shotcrete, sb = spot bolts, NONE = no support needed

Table 1. Essential features of NMT (after Barton et al, 1992b).

Reprinted from **TUNNELS & TUNNELLING**, OCTOBER 1994

1994

UPDATING OF THE Q-SYSTEM FOR NMT

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ABSTRACT

Since the early 1980s, wet mix, steel fibre reinforced sprayed concrete (S(fr)) together with rock bolts have been the main components of permanent rock support in underground openings in Norway. The concrete technology and the experience with this concept of rock support has improved considerably in this decade. Based on studies of 1,050 case records, an empirical connection has been established between the thickness of sprayed concrete and bolt spacing on the one hand and the rock mass quality, Q, on the other hand. In extremely poor rock mass quality, a concept using rebar steel reinforced sprayed concrete ribs in addition to S(fr) and rock bolts has been developed which has actually been replacing cast concrete lining during the last few years. The thickness, width and spacing between the ribs depend on the rock mass quality, Q. Rock support by means of S(fr) has also been widely used in order to prevent spalling and slabbing under high rock stresses. Use of the Q-system together with S(fr) and rock bolting as final tunnel support constitute the most important components of NMT, the Norwegian Method of Tunnelling. The article provides a detailed discussion of some improvements that have been made to the stress term SRF in the Q-system. Onset of stress slabbing in massive rock and squeezing in soft fractured rocks are more closely defined. Finally, the ability of early S(fr) support to minimise the SRF (loosening) term is noted, in marked contrast to the adverse effect of using steel sets which tend to increase the SRF value of the rock mass. Ground reaction concepts in Q-NMT support design are discussed.

INTRODUCTION

Sprayed concrete is a product which has mainly been developed by practical application. It is one of several tunnel support techniques, and is often combined with other types of support such as rock bolts, steel straps, wire mesh, steel arches and reinforced sprayed ribs. In recent years, use of additives and an increased knowledge of concrete have made it possible to vary the properties of sprayed concrete in desired directions, in response to the planned application (Opsahl, 1982).

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(NB sole author, hard work with case records by Grimstad)

LETTERS

Updating the NATM

Sir, Tunnels & Tunnelling should be congratulated upon impeccable timing, and Sir Alan Muir Wood on impeccable foresight for the Sept '94 article on potential problems with NATM in London Clay. The need to excavate and support more quickly in order to gain full benefit from negative pore pressure development was eloquently presented. It leads one seriously to suspect that: dry process shotcrete (with its large rebound, dust and invert quality control penalties); mesh reinforcement (with its long delays at intersections and shadow effects); and hand held equipment (with its volumetric, reach and human limitations) collectively doom NATM to a less than optimal performance in a weak, fissured medium beneath important structures at Heathrow and the Jubilee Line Extension. One should learn from one disaster and avoid a second by appropriate changes of methodology.

Classic negative pore pressure development is presumably most effective in an over-consolidated continuum and least effective in a fissured medium like much of the London Clay. Although each affected Clay 'block' is temporarily hardened by the unloading and/or initial shear strain and dilation accompanying tunnelling, the transfer of negative pore pressures to deeper layers is presumably less effective across fissures than through a continuous medium. Later re-establishment of positive pore pressures might nevertheless be assisted by minute flows along the fissures, where softening will preferentially occur.

The well known scale effects on shear strength and deformability documented by Marsland of BRS in the early '70s are fissure-induced and are also seen in (jointed) rock mechanics. Perhaps a rock mechanics engineer would be forgiven for suggesting that fissures should sometimes be included discretely in design as is regularly done in rock mechanics, either through empirical methods or using distinct element codes such as Cundall's UDEC and 3DEC. There is no need (nor do we have the ability) to model all blocks!

Exploring just the empirical approach, which is easier, one finds that the Q-system of rock classification can be readily applied in a logical and quite sophisticated manner to fissured London Clay, to take into account both time and construction methods, and of course geotechnical variability, using histogram field mapping sheets. The Q-system and NMT (Norwegian Method of Tunnelling) principles have been used in worse conditions than London Clay in many countries, with great success. Reference to *T&T*, Oct '94, p41 will help interpretation of what follows.

The London Clay must clearly be treated as an 'incompetent rock' following Deere, and be given RQD = 0 (i.e. use ten as the minimum value in the Q calculation). The number of fissure sets and more persistent joints and 'backs' will determine the J_n value (number of sets). The first pair of parameters (RQD/J_n) which describe relative block size will therefore be equal to approximately 10/(6 to 15) and will vary considerably from site to site.

The second pair of parameters (J_r/J_a) describing inter-block friction should have the final character of 'thick clay fillings' (no rock to rock contact) and should therefore be given the values of 1.0/(6 to 8). This is conservative; no initial dilation on the non-planar fissure surface has been allowed. In reality, J_r/J_a might vary from 2/4 to 1/8 with time if strain softening is allowed, and need more support as a consequence. The third and final pair of Q-system parameters (J_w/SRF) describing water ingress (or pressure) and the effective stress to strength ratio will clearly be affected in London Clay by whether the excavation is performed fast and supported fast (using robot applied fibre reinforced shotcrete) or excavated slowly and supported slowly using the type of hand held equipment frequently photographed in *T&T* from NATM construction sites (to the continued amazement of Norwegian tunnelling colleagues). Depending on tunnelling method, depth and site characteristics, J_w/SRF may have values in the range (0.66 to 1.0)/(2 to 5). (An allowance for squeezing is made in the worst case.) One ends up with a potential Q-value range of probably about 0.01 to 0.1, i.e.

$$Q = \frac{10}{6\text{-}15} \times \frac{1}{6\text{-}8} \times \frac{0.66\text{-}1.0}{2\text{-}5} = 0.01 \text{ to } 0.1$$

This range of quality plots in the 'extremely poor' rock class in the Q-diagram (Fig 1, *T&T*, Oct '94, p40). For excavation spans of more than 10m, this would imply substantial rib (reinforcing bar) reinforced (or lattice girder reinforced) steel fibre reinforced shotcrete of about 15 to 25cm thickness as final support in the best quality end of the above range of Q. Cast concrete final lining following rib reinforced S(fr) primary support would be needed at the other end of the quality range, again for spans of more than 10m. Primary S(fr) can be built up following NATM monitoring principles if desired; but the thickness can be designed using the Q-system.

Robot-applied steel fibre reinforced wet process shotcrete with production rates of 5 to 25m³/h (depending on the size of the rig) should be adopted in place of outdated mesh reinforced shotcrete both at Heathrow and Jubilee Line, allowing more rapid and mechanised mining and immediate support of the ground within the negative pore pressure phase emphasised by Sir Alan Muir Wood. With reduced strain softening, subsidence and final loads would thereby be reduced. Robot S(fr) technology is ten years and one cycle ahead of S(mr). Some would say that this lost cycle (for mesh fixing) is the difference between success and failure in difficult tunnelling. Commercial application of S(fr) 16 years ago in Norway has certainly kept prices low throughout this period in the 100km/year of tunnelling. Of particular relevance to Heathrow problems as reported in the UK press is that inverts are not covered by low quality rebound material. When using wet process S(fr), there is virtually no rebound!

Clearly, Norwegian contractors have been too quiet, and English owners all too easily influenced by NATM successes — of which there are of course many. It is perhaps time to combine the best aspects of English, Austrian and Norwegian tunnelling technology and advance tunnels in London faster and more cheaply, confident of having used the best available technology and design principles, thereby minimising the chance of another black October.

Yours faithfully,
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Clarification of NATM

Sir, NATM is not a hard rock method! The NATM was developed in Austria around 1954 for construction of soft ground tunnels. The first application for underground railway projects was at Frankfurt/Main in 1968 with the construction of a trial tunnel. The benefits of the technique in urban areas resulted in 70 per cent of the tunnelling carried out in Germany using the NATM, mostly in clay, silty clay and similar soft ground conditions.

In the intervening years since its initial introduction, many large cities around the world, e.g. Washington DC, Dallas, São Paulo, Brasilia, Seoul, Taipei, Istanbul, Athens, Rome, Thailand, among others, have accepted the method.

Regardless of the tunnelling method, collapses do from time to time occur and for a variety of reasons. When apparently new methods are applied using the concepts of older methods, the behaviour of the ground support is difficult to assess. Hence, the skill of the tunnelling practitioner needs to be re-adapted. In order to further these changes, the advice of known specialists needs to be sought, not only at the design stage, but especially in the control of the

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The Q-System following Twenty Years of Application in NMT Support Selection

By Nick Barton, Ph. D. and Eystein Grimstad M. Sc.

1994 marks a twenty-year milestone since the publication of the Q-system* (Barton et al., 1974). During this time the method has been used for the design of more than 1,000 km of tunnels in Norway alone. It has obviously matured in this time, been improved and updated, and is today being used more and more frequently as a quantitative measure of tunnelling conditions and support needs in an increasing number of countries around the world.

At the time of the Q-system development in the early 1970s, mesh reinforced shotcrete and bolting were increasingly being used as a replacement for steel sets and concrete, as a means of cutting costs, improving safety and completing tunnels more efficiently. S(mr) + B was becoming accepted in several countries even outside Scandinavia as a valid permanent support method. Of course, when no longer relying on the cure-all (but expensive) final cast concrete liner, there was a need to be sure of the adequacy of this seemingly „light" support method. The need to describe rock mass conditions in an appropriate manner (in case concrete *was* needed) was the reason that the Q-system could be developed, thanks to NGI's and other people's excellent case records.

In our experience, tunnel projects where reliance was placed on steel sets for temporary support and cast concrete for final support (virtually independent of rock conditions) were poorly described in an engineering geological sense. This state of affairs is probably still true today, and tunnelling costs are still very high in many countries, partly for these reasons.

Corrosion Worries Over

A tunnelling revolution has occurred in the last 15 to 20 years with the development of wet process shotcrete and the ability to spray stainless steel fibre reinforcement S(fr) in dense, low permeability concrete of C35 to C45 MPa in situ quality. Since steel fibres are non-continuous, they do not suffer anodic/cathodic corrosion like steel mesh or steel reinforced concretes or shotcretes.

In the area of bolting, another revolution has occurred with the development of epoxy-coated and PVC-sleeved triple corrosion protection rock bolts. These can be end anchored and tensioned as temporary support, and later (after shotcreting), can be fully grouted in one simple operation both along the inside and outside of the PVC liner.

No longer can critics claim that final support consisting of S(fr) + B (steel fibre reinforced shotcrete and systematic bolting) has limited life. Of course, as in 1974, many Owners and Consultants are still nervous of the longevity and assumed maintenance costs of „light" permanent support methods such as S(mr) or S(fr) and bolting. However, if they follow the recommendations and methods outlined in the remainder of this paper, they will acquire many kilometres of tunnels at a fraction of the present cost!

The Norwegian Public Roads Administration, in their 450 km of main road tunnels have stretches totalling some 160 km where final support consists only of shotcrete (S) or fibre reinforced shotcrete S(fr) and bolting (Grimstad et al., 1993). Critics and conservatives may assume that this is due to the predominantly harder jointed rocks in Norway. However, S(fr) + B is not used unless rock conditions are poor or very poor (i. e., Q-values from about 4 down to 0.01). Such conditions usually involve heavy jointing, clay bearing joints and marked overbreak.

Concrete lining is only used where exceptional conditions prevail. However, it is steadily losing ground to RRS (rib reinforced shotcrete) supplemented by S(fr) + B (Grimstad and Barton, 1993). This is a flexible (easy to apply) method of building steel reinforced shotcrete ribs that are in immediate and complete contact with the whole tunnel profile. Their thickness and spacing can be varied as dictated by the ground and by convergence measurements.

Q-System Classification

Following an extensive period of trial and error in 1973, a final total of six Q-system parameters and ratings were developed as shown in equation 1 and in Table 1. According to the Q-system, the rock mass quality may be expressed by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad [1]$$

The numerical value of Q ranges from 0.001 (exceptionally poor) to 1000 (exceptionally good) quality rock. The six parameters can be estimated from surface mapping and from core logging, and can later be verified or corrected during excavation. The parameters represent:

RQD = degree of jointing
J_n = number of joint sets
} $\frac{RQD}{J_n}$ is a measure of block size

J_r = joint roughness
J_a = joint alteration or filling
} $\frac{J_r}{J_a}$ is a measure of inter-block friction angle

J_w = joint water leakage or pressure
SRF = rock stress conditions
} $\frac{J_w}{SRF}$ is a measure of the active stresses

The large range of Q-values (six orders of magnitude) is a very important feature of the Q-system and reflects rock quality variation probably more readily than the linear

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* See Glossary of Terms at the end of this article.

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Predicted and Measured Performance of the 62 m Span Norwegian Olympic Ice Hockey Cavern at Gjøvik

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The feasibility of excavating caverns of very large span for underground siting of nuclear power stations in Norway was investigated in the early 1970s. In the end, the 1994 Winter Olympic Games provided the necessary impetus for utilizing a very large engineered rock cavern and proving its general feasibility. The 62 m span Olympic Ice Hockey Cavern was constructed in Gjøvik by Veidekke-Selmer JV in 1991. It is located in a jointed gneiss of average RQD = 67%. The Q-values range from 1 to 30, with a weighted mean of about 9, i.e. fair quality rock. The cavern has a rock cover of only 25–50 m, thus posing challenging design problems. The investigations prior to construction included two types of rock stress measurements, cross-hole seismic tomography, geotechnical core logging, Q-system classification and numerical modelling with UDEC-BB. Predicted maximum deformations were 4–8 mm; these were surprisingly small due to the high horizontal stresses recorded. Extensometer (MPBX) installations from the surface prior to construction, precision surface levelling and MPBX installed from inside the cavern gave a combined measure of maximum deformations in the range 7–8 mm with the 62 m span fully excavated, and three adjacent caverns for the Postal Services also completed. Permanent rock reinforcement based on the Norwegian method of tunnelling (NMT), consisted of 10 cm wet process steel fibre reinforced shotcrete, and systematic bolting and cable bolting in alternating 2.5 and 5.0 m c/c patterns. Both the cables and bolting were untensioned and fully grouted.

INTRODUCTION

During the 1970s, NGI performed a series of siting studies and some *in situ* testing, to investigate the feasibility of underground siting of nuclear power plants. Special attention was focused on the need for a reactor containment cavern with a hemispherical domed arch of at least 50 m diameter. The Norwegian State Power Board (Statkraft) and subsequently also the Swedish State Power Board (Vattenfall) and Sweden's BeFo organization funded parallel theoretical studies of large-span caverns at NGI.

Physical models of large spans in jointed rock were used to study the effect of medium and high horizontal stress levels and the effects of various joint orientations. Comparisons were also made with continuum FEM studies. Today, fifteen years later, we would probably have used discrete element methods such as UDEC, although the number of discrete blocks in the physical models (20,000) exceeds all but the most extreme discrete element models.

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A Q-SYSTEM CASE RECORD OF CAVERN DESIGN IN FAULTED ROCK

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Summary

A 23m span by 46m high pumped storage power house to be located in interbedded siltstones and sandstones with up to twelve inclined bedding plane faults intersecting the 160m long excavation does not represent ideal geology for large cavern construction. However, appropriate solutions were engineered by the cavern designers and their consultants which may have application elsewhere. In this paper the role of empirical Q-system based design is highlighted, and it is shown how seismic design considerations were incorporated in the integrated empirical design. A discussion of the cavern performance and of reinforcement strategy dilemmas is given.

Introduction

The Mingtan Pumped Hydro project at Sun Moon Lake in Central Taiwan was the first large cavern project in Taiwan in which shotcrete (fiber reinforced) and systematic bolting (and cables) were accepted as final support. Previously, somewhat conservative thick concrete linings were used, based on Japanese designs of this earlier period. Cavern owners Taipower, and their consultants Sinotech engaged Golder Associates and Dr. Evert Hoek for assessment of alternative designs, and later also the author for adjustments of S(fr) + B design using the Q-system.

The geology at Mingtan was somewhat unique for large cavern construction, due to the presence of some twelve bedding-plane-parallel faults in the 35° dipping sandstones and siltstones of the Waichecheng Series. Figure 1 shows the general layout. One or two of the faults had a meter or so thickness of clay filling and rock fragments. Nine of the major faults were "seam-treated" using a special Sinotech technique in the area of the future powerhouse arch, by replacing the clay with grout and concrete using high pressure water jetting (Liu, et al. 1988). Access was from longitudinal galleries driven on either side of the future powerhouse arch. Some 4m of rock (clay) above the (future) cavern roof was treated in this way. This is shown in Figure 2 from Moy and Hoek (1989).

At the time of the author's first visit in 1987 this seam treatment was successfully completed, and extensive pre-reinforcement of the future cavern arch had been achieved with untensioned but grouted 2 × 15 mm tendons at 2m centres, splayed downwards from the floor of the drainage gallery. This system of prereinforcement is also shown in Fig. 2.

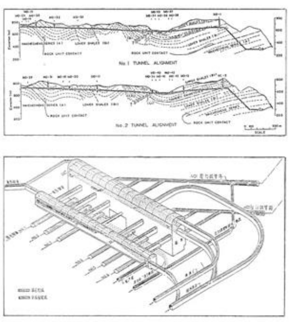

Figure 1 - The Mingtan pumped storage project in Taiwan. Powerhouse axes were finally perpendicular to the strike of the bedded sandstones, siltstones and bedding plane faults. (Liu, Cheng and Chang, 1988/Taiwan Power Company and Sinotech Inc.).

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Radioactive waste repository design using Q and UDEC-BB

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Abstract

The NGI methods of characterising joints (using JRC, JCS and ϕ_r) and characterising rock masses (using the Q-system) are being utilised in a current geotechnical consultancy project for UK Nirex Ltd. Present geotechnical characterisation activities include the logging of six kilometres of 100mm drill core.

Preliminary rock reinforcement designs (systematic bolting and unreinforced or fibre-reinforced shotcrete) are derived from the Q-system statistics, which are logged in parallel with JRC, JCS and ϕ_r. Discrete element UDEC-BB modelling provides a check on the performance of the proposed excavations with Q-system reinforcement, giving predicted bolt loads and rock deformations, together with joint shearing and hydraulic apertures to better define the disturbed zones.

INTRODUCTION

The NGI methods of characterising joints (using JRC, JCS and ϕ_r) and characterising rock masses (using the Q-system) are being utilised in a current geotechnical consultancy project for UK Nirex Ltd. This organisation is responsible for the safe disposal of solid low and intermediate level radioactive waste in the UK. Present planning and site investigation is now focused at Sellafield in NW England where extensive deep drilling, downhole testing, geological and geophysical investigations are progressing. According to present plans, approximately 2 million m^3 of low and intermediate level radioactive waste may eventually be disposed of at Sellafield, utilising large rock caverns at depths in the region of 800m. Present plans are for caverns of 25m span and heights of 15m (low level waste) or 35m (intermediate level waste) (refer to Ireland, 1992) [9].

The NGI/WS Atkins/Taywood Engineering work as Geotechnical Consultants to UK Nirex Ltd has included field mapping and core logging, using newly developed geotechnical logging charts which combine Q-system parameter histograms with more detailed joint and rock mass descriptions suitable for use in the distinct element code UDEC-BB (Cundall, 1980 [6], Makurat et al., 1990 [11]). Extensive numerical analyses of access tunnel and cavern excavation response are being carried out to investigate rock reinforcement requirements and the extent of the disturbed zones. The latter is graphically represented by UDEC-BB plots of stress, deformation, joint shearing and joint aperture distributions and magnitudes.

GEOTECHNICAL LOGGING CHART

As a first step in the rock mechanics design process, data of relevance to cavern and tunnel design studies are collected from field mapping and from current logging of some 6 km of oriented drill core. The methods used by BGS for core orientation are described by Horseman et al. (1992). [8]

The NGI/WSA team has utilised newly developed geotechnical logging charts for describing jointing in the drill cores. The chart used in the field mapping is illustrated in Fig. 1 for a hypothetical data set. A brief explanation of each parameter logged is given below.

Since the stability of excavation in hard rock masses depends largely on jointing, much of the data concerns such features as joint geometry and the surface characteristics of the joint planes. Included in the charts are also some data concerning permeability, rock compressive strength and rock stresses, as obtained by other Nirex contractors.

Data for the six Q-system parameters are given on the left hand side of the geotechnical chart. In the histograms drawn for each Q-parameter, values plotting on the right hand side of the chart are favourable for good stability, while values plotting more on the left hand side imply poorer stability.

To the right of the six Q-parameters in the geotechnical chart, there are other parameters which express rock mass character in related ways. These additional data are necessary in order to give a more complete description of the rock mass and rock joints, especially for the performance of subsequent numerical modelling.

The upper third of the chart (including RQD and J_n) describes geometrical factors of the ***rock mass*** as a whole. The middle third of the chart (including J_r and J_a) describes ***joint character***. The lower third of the chart (including J_w and

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The influence of joint properties in modelling jointed rock masses

Influence des propriétés des diaclases sur la modélisation des masses rocheuses diaclasées

Der Einfluß von Spaltbrucheigenschaften auf die Modellbildung von geklüfteten Felsmassen

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ABSTRACT: Prediction of likely response to excavation, and production of final designs for the rock reinforcement, require realistic descriptions of the components of rock mass behaviour. This article explores some of the methods that have proved reasonably successful in describing and modelling rock joints and rock masses, despite the complexities involved. Index testing of rock joints and rock mass characterisation, including geophysical methods, are the essential activities in preparation for two- and three-dimensional distinct element modelling. Recent improvements are described.

RESUME: La prévision de la réponse vraisemblable d'un massif rocheux lors de la réalisation d'une excavation, ainsi que le dimensionnement des renforcements nécessaires, nécessitent une description réaliste du comportement des composants de ce massif. Cet article explore quelques unes des méthodes qui se sont montrées raisonnablement satisfaisantes pour la description et la modélisation des massifs rocheux et de leurs joints, en dépit de la complexité que cela suppose. Les essais sur joints et la caractérisation du massif (y compris par les méthodes géophysiques) sont les éléments essentiels préalables a une modélisation en deux ou trois dimensions par éléments discontinus. Des développements récents sont décrits.

ZUSAMMENFASSUNG: Die Vorhersage der wahrscheinlichen Gebirgsreaktion auf das Auffahren von Untertageräumen und das Design von Felsverstärkungen verlangt die wirklichkeitsnahe Beschreibung der einzelnen Komponenten des Felsverhaltens. Dieser Artikel beschreibt einige Methoden, welche trotz ihrer Komplexität, erfolgreich zur Kluft- und Felsmodellierung und Beschreibung angewandt werden. Das Indextesten von Klüften und die Gebirgsklassifizierung, geophysikalische Methoden eingeschlossen, sind wesentliche Bestandteile in der Vorbereitungsphase von zwei und dreidimensionalen bestimmten Elemente Simulierungen. Neuere Entwicklungen werden beschrieben.

1 INTRODUCTION

This article explores some of the methods which appear to be having some success in realistic modelling and design for jointed rock masses. Key techniques are joint index testing, rock mass characterisation, seismic measurements and distinct element modelling. At NGI, these methods can be represented by the following basic symbols: JRC, JCS, ϕ_r Q, V_p, UDEC and 3DEC. The first three are the index parameters for the joint sets of concern (Barton and Bandis, 1990). The Q-values give estimates for rock mass moduli and rock reinforcement, following Grimstad and Barton, 1993. The two- and three-dimensional distinct element models UDEC and 3DEC conceived by Cundall and refined by Itasca Inc. provide the final essential link to reality.

Spatial variability within the rock mass which is reflected to some extent by the statistics for JRC, JCS, ϕ_r and Q, is further described by the seismic measurements which provide a means of extrapolation between mapping locations (i.e., exposures or drill core). In its optimal form (cross-hole seismic tomography), it gives detailed information that can be approximately correlated to Q-values and to deformation modulus, using recent developments.

2 SHEAR BEHAVIOUR OF ROCK JOINTS

Direct shear tests of rough-walled tension fractures developed in weak model materials, that were performed many years ago when the author was a student, indicated the importance of both the surface roughness and the uniaxial strength (σ_c) of the rock. The empirical relation for peak shear strength given in Equation 1 was essentially the forerunner of the subsequent JRC-JCS or Barton-Bandis model, where the joint roughness coefficient (JRC) was equal to 20 for these rough tension fractures. The joint wall strength (JCS) was equal to σ_c (the unconfined compression strength).

$$\tau = \sigma_n \tan\left[20 \log\left(\frac{\sigma_c}{\sigma_n}\right) + 30°\right] \quad (1)$$

The original form of Equation 1 is therefore perfectly consistent with today's equation:

$$\tau = \sigma_n \tan\left[\mathrm{JRC} \log\left(\frac{\mathrm{JCS}}{\sigma_n}\right) + \phi_r\right] \quad (2)$$

Equation 1 represents the three limiting values of the three input parameters, i.e.,

JRC = 20 (roughest possible joint without actual steps)

JCS = σ_c (least possible weathering grade, i.e., fresh fracture)

ϕ_r = ϕ_b (fresh unweathered fracture with basic friction angles in the range 28½ to 31½°).

Bandis et al., 1981, 1983 and Barton et al., 1985, have subsequently shown how these three index parameters JRC, JCS and ϕ_r can be used for modelling both the shear-dilation and normal closure behaviour of rock joints with estimation of physical and hydraulic joint aperture, and with due account of scale effects and shear reversals, etc.

Figure 1 illustrates the first version of the constitutive model for shear and dilation behaviour, which was subsequently coded by Itasca for use in UDEC-BB (Christianson, 1985, personal communication) and improved by NGI and Itasca (Gutierrez, 1995; Christianson, 1995, personal communication) for use in an improved version UDEC-BB.

Although different degrees of joint weathering and mineral

1995

Investigation, Design and Support of Major Road Tunnels in Jointed Rock using NMT Principles

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Summary When fibre reinforced shotcrete, S(fr), and rock bolts form the key components of permanent rock reinforcement and tunnel support and are not followed by concrete lining, then the investigation, design and tunnel support phases have each to be relied upon to a greater extent than is the case with typical NATM tunnelling. The Norwegian Method of Tunnelling (NMT) which can be used for a very wide range of jointed and faulted rock, places reliance on rock mass classification, on empirical design of permanent support, on numerical verification of special cases and on the knowledge or assumption that a flexible approach to rock support variation will be possible within the contract. "Design as you drive" or "*in situ* selection of support" presupposes anticipation and designs for the full range of rock conditions, and unit prices for all the tunnelling and support methods likely to be used. Tunnelling and support costs in the range of US$4,000 to US$8,000 per metre are normal in Norway for two-to-three lane highway tunnels using these NMT principles. The article demonstrates the use of the Q-system and correlations with seismic investigation methods for anticipation of the likely range of tunnelling conditions. A look at the Sydney basin sandstones is used to demonstrate this method. Numerical verification of empirical support designs is demonstrated with UDEC-BB and UDEC-S(fr). Finally, some details of NMT permanent support components are illustrated including corrosion protected rock bolts, almost rebound-free fibre reinforced shotcrete and economic frost and water insulation methods.

1 INTRODUCTION

In the context of road and rail tunnels, NMT (Barton et al., 1992) is a collection of practices that produce dry, drained, permanently supported and "lined" (fully cladded) tunnels for approximately US$4,000 to US$8,000 per metre. These low-cost, high-tech Norwegian tunnels may range in cross-section from about 45 m^2 to 110 m^2. The following list gives the essential components of NMT.

1.1 Design

- Preliminary design is based on field mapping, drill core logging and seismic interpretation.
- Rock mass quality is usually described by the Q-value (Barton et al., 1974; Barton and Grimstad, 1994).
- Final support is selected during tunnel construction based on tunnel logging and use of the Q-system support recommendations.
- Numerical verification of the various permanent support classes may be performed with the distinct element (jointed) two-dimensional UDEC-BB or three-dimensional 3DEC computer codes.
- A basic NMT designed tunnel is drained, with insulated, pre-cast concrete panels for water and frost control when needed. These can be assembled at approximately 1 km per month.

1.2 Contractual

- The Owner pays for technically correct support.
- The Contractor is compensated *via* the unit prices quoted in the tender document.
- The Owner bears more risk than the Contractor, thereby reducing prices.
- Needed support is based on the agreed Q-value, and may vary frequently.

1.3 Excavation and Support

- Excavation, usually by drill and blast, is tailored to the rock conditions.
- The temporary support such as sb, B or B+S(fr)[1] is approved as part of the permanent support.

[1] sb = spot bolts; B = systematic, fully grouted bolting; S(fr) = wet-mix, steel fibre reinforced shotcrete

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Rock Mass Characterisation and Seismic Measurements to Assist in the Design and Execution of TBM Projects

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Abstract

The choice of TBM excavation *contra* drill-and-blast excavation of major tunnels is a decision that can have major economic and schedule implications. Unfortunately, both economy and schedule on occasions go in the opposite direction to those expected, and greater costs and more time are incurred than would be likely with the drill and blast alternative. This paper is an attempt to reduce the risk in TBM tunnel driving, so that costly TBM machines are less likely to be stopped for long periods. The proposed method is based on systematic collection and utilisation of data from the rock ahead of the TBM. The principles involved are based on correlations between seismic P-wave velocity and rock quality. Rock quality is described by a Q-value that has been normalised by the uniaxial rock compression strength. An estimate of the rock matrix porosity and the approximate stress level or depth is also used in the analysis of tunnel support needs. Convergence monitoring is used to verify that the prediction of support class and execution of support is in accordance with expected behaviour.

Introduction

Large TBM machines represent a very big financial investment and they are a commitment to a method of tunnelling that includes standard solutions to what may be a very wide range of ground problems. It probably has to be admitted that the range of human and technical ingenuity in tackling tough ground problems is limited by the TBM "tunnel production" method. Problems incurred can, on occasion, be correspondingly exaggerated due to the reduced flexibility when a large tunnel is filled with perhaps 200 metres of heavy machinery and associated equipment. More room for human ingenuity and a wider range of solutions are available in the drill-and-blast method, including multi-drift excavation and more thorough pre-injection or pre-treatment methods.

Learning Curve or Geological Delays?

On some TBM projects, there are extended early periods of low productivity due to mistakes in expected ground conditions (Barton and Warren, 1996). These are usually followed by impressively inclined learning curves once redesign of TBM details and crew experience are optimised. A classic example of this is the Channel Tunnel project between England and France, where English contractors eventually broke world records for soft ground (chalk marl) tunnelling after making some design changes that were needed because of blocky, overbreaking, water-bearing ground in the early kilometres.

1996

ESTIMATING ROCK MASS DEFORMATION MODULUS FOR EXCAVATION DISTURBED ZONE STUDIES

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ABSTRACT

The full scale deformation modulus that needs to be used in numerical models of tunnels, caverns and geological disposal facilities (or related tunnel research sites) is of fundamental importance to the stresses, displacements and magnitude of the excavation disturbed zone or EDZ that is predicted. An alternative to direct measurement which has merit in scoping exercises and may be accurate enough for detailed design is outlined in the paper. The method is derived from rock mass characterisation methods and was initially based on correlation between Q and RMR to give access to additional case records. Key parameters in the new method include the seismic P-wave velocity obtained from seismic refraction surveys, or from cross-hole seismic tomography, the Q-value, the depth of the site and the physical properties of the matrix as described by its uniaxial compression strength and porosity. The method has been checked at hard rock sites with sparse or frequent jointing, and in weaker, porous rocks which have no relevance to waste disposal but which provide a large range of conditions for verification.

INTRODUCTION

Predicting the behaviour of excavations in rock masses is complicated by the huge number of interlocked pieces of rock that react with one another *via* non-linear stiffness and strength components. In one major school of rock mechanics the obviously discontinuous rock mass is simplified as if it were a continuum. Finite element, finite difference, or boundary element analyses are utilised with elastic or elasto-plastic constitutive models. There is an obvious need for a good estimate of the rock mass modulus which takes into account "all" the features of the rock mass that are otherwise ignored. Nevertheless, details of behaviour are sure to be missed in such analyses and it is often necessary to change the modulus close to the excavations to get better fit to observed rock displacements, *e.g.*, Barton and Bakhtar (1983).

Another school of rock mechanics which is expanding due to the needs for more detailed understanding of "real" behaviour, follows the argument that major sets of joints and discontinuities can be represented discretely in two- and three-dimensional distinct element models such as UDEC or 3DEC, *e.g.*, Cundall and Hart (1993). However, since the number of discrete blocks that can be modelled is still rather limited—usually no more than a few thousand blocks—it is inevitable that the detailed joint structure that is seen in a scale of a few metres is only represented in terms of a deformation modulus and Poison's ratio. Only the major joints, *i.e.*, those which are expected to affect performance most, are modelled discretely. An

1996

Rock Mass Classification of Chalk Marl in the UK Channel Tunnels

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ABSTRACT

Although Chalk Marl is nearly at the weakest end of the strength spectrum for rock, its bedded and jointed nature make it quite amenable to classification by rock mass quality descriptors such as the NGI Q-system. Steeply dipping jointing and subhorizontal bedding was mapped and photographed in the partly flooded Beaumont (Abbots Cliff) and Terlingham Tunnels prior to analysis of core logs and core box photographs from the PB series of marine core drillings. Mean Q-values were 3.4, 10.6 and 12.6 respectively. The Grey Chalk seen in the cliff exposures at Shakespeare indicated Q-values in the range 4 to 33. Jointing appears to have been similar in the slightly weaker underlying Chalk Marl, where permeabilities of about 1 to 20 Lugeons in an otherwise very impermeable matrix also indicated the presence of extensive jointing. The jointed and bedded nature of Chalk Marl as experienced in the Beaumont, Terlingham and Channel Tunnels resulted in a lot of distinctly discontinuum as opposed to continuum behaviour. Overbreak was marked where joint sets, bedding joints and an unfavourable tunnel direction combined to give the necessary degrees of freedom for block release. The inevitability of block release problems was increased by the relatively smooth and planar character of the joints and by the destabilising effect of high pore pressures in the case of the sections of the Channel Tunnel having low cover and higher permeability. Trans Manche Link (TML)'s own rock mass Q-characterisation in the Marine Service Tunnel for km 20-30 was based on 250 face logs and 1,120 side wall logs. Average Q-values were 9.9 for km 20 to 24 where most difficulties with overbreak were experienced, and 33.4 for km 24-30. Lower values were obtained when only face logs were analysed due to the absence of swarf. In the low cover zone between km 20.5 to 21.3, TML's mean Q-value was only 5.6. The above range of mean values is similar to that obtained independently from pre-construction sources. According to Q-system case records, tunnels of 8.4m span (Marine Running Tunnel, MRT) and 5.3m span (Marine Service Tunnel, MST) need Q-values of 40 and 10 respectively for no support to be required. The 17 to 18m of unsupported tunnel lengths behind the MST and MRT tunnel boring machine tunnel faces made overbreak a very likely phenomenon when Q-values were in the range 1 to 10.

1996

USING NMT PRINCIPLES IN PREDICTING PERFORMANCE OF A POWERHOUSE IN THE HIMALAYAS, INDIA

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SYNOPSIS

This paper describes some of the NMT (Norwegian Method of Tunnelling) priciples applied to an underground powerhouse in the Himalayas. The discontinuum code UDEC-BB (Barton - Bandis joint constitutive model) has been used for two-dimensional modelling of the Nathpa Jhakri powerhouse in low strength anisotropic rocks in the Himalayas region in India. The powerhouse has a 20 m span, 49 m high walls and is 216 m in length. The main rocks in the area are metamorphic rocks such as gneisses, schists, gneissose schists and basic intrusives (amphibolites). The low strength metamorphosed rocks are quartz mica schists, biotite schists, and muscovite schists. The input data required for UDEC-BB have been derived from Q-system logging, from index testing density, porosity, specific gravity, uniaxial compressive tests and rock joint characterization of drill core (JRC, JCS, ϕ_r) and from in situ stress measurements, using hydrofracturing and overcoring techniques and from sonic wave measurements. The stabilizing effect of the fiber reinforced shotcrete S(fr) in underground constuctions is now possible to model in UDEC-BB. With the new version 2.02 of UDEC - BB (December '95) it is possible to model both the effect of applying S (fr) and the rock reinforcement bolts that are installed afterwards. In this paper we present results from numerical modelling for modelled S (fr) thickness varying between 15 cm for the arch and 10 cm for the walls (model 1) and 25 cm for the arch and 20 cm for the walls (model 2). In addition to this, we also modell the Q-system derived rock bolt pattern of 32 mm in diameter with alternating length 6 and 12 m bolt at 3 m spacing. By the time of writing this article (December '95) the excavation of the powerhouse was almost completed. The results of this most recent modelling work are discussed. A maximum deformation value of approximately 45 mm on the cavern walls is predicted after the final excavation of the powerhouse. The maximum recorded deformation on powerhouse arch by the time of writing this article was 24 mm. There are strong indications that the deformation on the cavern walls may be around 40 mm.

1 INTRODUCTION

In recent years construction of tunnels through low strength anisotropic rocks such as phyllites, shales and schists in the Himalayan Regions has generated new thoughts in anticipating and assessing the problems in such rocks. The problems faced in tunnelling through these rocks include squeezing ground if the rock contains a considerable amount of clay minerals and loosening of the rock mass in the case of layered and jointed rock masses. Loosening results in the separation of the rock mass from the main body which produces a dead load.
The behaviour of low strength anisotropic rocks cannot easily be assessed through common engineering experience due to the variation of mineral assemblage, fabric and geo-mechanical properties. Hence, a detailed engineering geological assessment of such rocks is warranted.

In this paper the three types of schists namely, quartz mica schist, biotite schist and muscovite schist encountered in the head race tunnel and in the underground powerhouse at the project site have been analyzed and have also been modelled numerically with weak zones in the jointed rock mass (Figure 1). The index studies carried out include petrographic and petrofabric analysis through electron microscopy, thin sections and X-ray diffraction. The geo-mechanical properties have been evaluated with emphasis on their behaviour in underground structures.

Two approaches have been adopted to study the geo-mechanical properties of these rock masses. The first approach includes laboratory tests to find the index properties of the rock which include the density, specific gravity, porosity, sonic wave velocity and the uniaxial compressive strength of the rock samples. The second approach estimates the strength and deformability of a jointed rock mass through the recently updated Q-System of rock mass classification (Grimstad & Barton, 1993). In-situ rock stress measurements for finding the principal stress directions have also been carried out. The numerical modelling techniques (UDEC) i.e., the distinct element code (Cundall, 1980) which incorporates the strength and deformability properties of the joints and intact rock separetely, has been used to predict the

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Comparison of Predicted and Measured Performance of a Large Cavern in the Himalayas

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Detailed investigative and performance monitoring studies have been carried out at the site of an underground powerhouse cavern in the Himalayan Region of India. The updated empirical (Q-system) and numerical (UDEC-BB) approaches, applied for predicting the behaviour of the rock mass prior to the construction of the underground cavern (20 × 49 × 216 m), have been compared with the instrumentation data from multi point borehole extensometers (MPBX). Upon completion of the first numerical excavation step (20 m span arch), a relatively high stress–strength ratio and a maximum deformation of approx. 18 mm was predicted in the roof of the cavern. MPBX readings in the arch have indicated maximum deformations in the range 19–24 mm with the 20 m span fully excavated. The results of numerically excavating the cavern to its full height (49 m), have indicated maximum deformations in the range 43–45 mm in the walls of the cavern. Upon completion of the ongoing benching operations, the measured performance from the walls of the cavern will be available for comparison with the existing numerical results. Permanent rock support in the cavern consists of systematic bolting of alternating lengths and mesh reinforced shotcrete S(mr). However, rock support design recommendations based on the Norwegian Method of Tunnelling (NMT), which employs wet process fibre reinforced shotcrete S(fr) instead of S(mr), have been numerically tested and verified.

INTRODUCTION

Rock mass classifications, which form the backbone of the empirical approach, have proven to be useful in providing guidelines for assessing the behaviour of rock masses and in choosing support requirements. Over 1050 cases have been analysed in the updating of the Q-system [1].

Ever since its development, the Q-system of Barton *et al.* [2], has attracted the attention of tunnel engineers, field geologists and researchers in its application to hard, jointed and faulted rocks. Construction engineers and geologists have preferred the empirical approach over the analytical and numerical approach, mainly because of its simplicity. While classification of rock masses will never be a substitute for experience in tunnelling, there is no doubt that an approach using one of the established classification schemes, together with a suitable numerical modelling technique, can help in forecasting and in better understanding the behaviour of the ground.

With the advent of a new statistical method of logging the Q-system parameters and the more detailed joint and rock mass descriptors (JRC, JCS and ϕ_r), the empirical and numerical approaches have recently been suitably integrated [3,4]. The mapped geotechnical data together with the Q-system parameters can be conveniently incorporated into numerical models for predicting the behaviour of the rock mass and for validating the empirically derived reinforcement. The interaction between the potentially relevant variables for engineering design, such as rock type, discontinuities, stress,

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1996

A comparison of the Barton-Bandis joint constitutive model with the Mohr-Coulomb model using UDEC

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ABSTRACT: A numerical study is performed to investigate the influence of a joint constitutive model on the stress-strain behaviour of a rock mass. Distinct element simulations are carried out on 3 different block size models of a rock mass using the Barton-Bandis (BB) and the Mohr-Coulomb (MC) joint constitutive models. The results show that the peak shear strength of a rock mass depends on the constitutive law used. The BB model, which allows the modelling of the dilation accompanying shear, predicts results similar to those from reported physical model tests on jointed slabs of a rock model material. A closely jointed rock mass in which block rotations occur exhibits a lower stiffness but a higher strength than a rock mass with widely spaced joints. The MC model, in which the dilation angle is constant, is relatively insensitive to the effects of different block sizes on the stress-strain behaviour of a rock mass.

INTRODUCTION

Numerical models serve as useful tools in simulating the response of discontinuous media subjected to loading. The Discrete Element Method, UDEC [Universal Distinct Element Code, (Cundall (1980), Cundall and Hart (1993)] is a powerful discontinuum modelling approach for simulating the behaviour of jointed rock masses subjected to quasi-static or dynamic loading conditions. In this method, the deformations and volumetric changes of the intact rock material (blocks) as well as the shear and normal displacements along the joints are included.

Due to the high degree of non-linearity of the systems being modelled, explicit (as opposed to implicit) numerical solution techniques are favoured for codes like UDEC (Universal Distinct Element Code). In this technique no matrices are formed as the procedure marches forward in small steps ensuring final equilibrium at each material integration point in the model.

The mechanical behaviour of a jointed rock mass is strongly affected by the behaviour of discontinuities. Therefore, an inevitable component of many numerical techniques is the constitutive model of discontinuities. During the excavation of an underground opening, the jointed rock may slip or separate along the discontinuities and the movement of the rock blocks may occur through translational or rotational shear. A clear understanding of the mechanical behaviour of rock joints is important for analysing and predicting underground structures in jointed rock masses. Several joint constitutive models have been developed in the past two decades for providing a realistic simulation of the mechanical behaviour of rock discontinuities. (*e.g.,* Barton (1982) and Barton and Bandis (1990), Cundall and Hart (1984), Saeb and Amadei (1992), and Souley and Homand (1995). However, it is still customary among many numerical modellers to use the non-realistic linear-elastic Mohr-Coulomb joint constitutive model. This may be attributed to its computational efficiency in numerical codes and the assumed availability of the Mohr-Coulomb parameters for the cohesion intercept and friction angle in the literature.

This paper compares the results from numerical modelling of the stress-strain behaviour of a rock mass using the non-linear Barton-Bandis (BB) joint constitutive model with those from the Mohr-Coulomb (MC) model. The numerical modelling of block size effects and the influence of joint

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JOINT APERTURE AND ROUGHNESS IN THE PREDICTION OF FLOW AND GROUTABILITY OF ROCK MASSES

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ABSTRACT

Changes in the geometry of rock joints following changes in the state of normal and shear stresses affect rock to rock contacts, roughness, aperture and tortuosity of flow channels. The paper discusses the role performed by the effective physical aperture **E** (or ΔE) and its relation with the theoretical aperture **e** (or Δe) used in the parallel plate analogy for flow in rock joints. The influence of joint wall roughness is discussed in terms of the joint roughness coefficient **JRC** (Barton and Choubey, 1977) and the relative roughness concept (Lomize, 1951). The behaviour of the ratio **E/e** and the simultaneous influence of roughness and aperture on flow through rock joints is analysed in terms of the hydraulic conductivity of a joint for varied **JRC** and relative roughness, using empirical equations derived from laboratory work. Grouting prediction and behaviour of the ratio **E/e** after grouting is also discussed.

KEYWORDS

Rock joints, aperture, roughness, shearing, flow, coupled processes, JRC, hydraulic conductivity, relative roughness, groutability.

1 INTRODUCTION

Changes in the hydraulic conductivity of rock joints induced by changes in normal or shear stresses are important for the evaluation of the hydromechanical behaviour of rock masses and grouting prediction.

Basic phenomena related to these problems have been studied by many authors, since the 1970's (Jouanna, 1972; Louis, 1974; Gale, 1975; Iwai, 1976; Whitherspoon at. al., 1979, among others). These researchers aimed most of the time to evaluate the effect of normal stress on the hydraulic conductivity of the rock joints.

Coupled methods related to changes in normal or shear stresses however, increased substantially only in the last 15 years probably due to the needs of the nuclear waste and petroleum industries, and due to the advent of personal computers and consequent advances in numerical modelling of rock masses and rock joints. There was therefore a need for extensive experimental testing to obtain relevant input data and behavioural laws.

Following this trend, some of the major factors controlling flow through rock joints were extensively studied in the laboratory and simulated by modelling (Gale, 1982; Bandis et al., 1983; Barton, 1985; Barton et al., 1985; Makurat and Barton, 1985; Raven and Gale, 1985, Hakami and Barton, 1990; Esaki et al., 1995, Makurat and Gutierrez, 1995, among others).

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The Disturbed Zone Around Tunnels in Jointed Rock Masses

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1. INTRODUCTION

The disturbed zone around an excavation is a region where the original state of the *in situ* rock mass, such as stress, strain, rock stability, water flow, etc. has been affected. The definition of the disturbed zone depends on the nature or the purpose of the excavation. For instance, the disturbed zone of a road tunnel normally means the region where rock blocks have undergone notable displacement or the tangential stress shows a major increase. The displacement and stresses are the factors controlling the tunnel stability. For nuclear waste disposal however, the disturbed zone around a deposition tunnel is more frequently considered the area where joint movement (open or sliding) occurs. The joint movement in this case is of more concern than the tunnel's local stability because it changes the water flow, and hence increases the possibility for radioactive material migration.

In both cases, joints in the rock mass play a key role in the development of the disturbed zone. Joints can create loose blocks near the tunnel profile and cause local instability [1]; joints weaken the rock mass and enlarge the displacement zone caused by excavations [2,3]; and joints change the water flow system in the vicinity of the excavation due to the channelling effect [4].

According to the frequency of jointing, a rock mass may be described as "intact" (without joint), "sparsely jointed" (with a few joints), "jointed" (with several intersecting joint sets) and "heavily jointed" (with closely spaced and intersecting joint sets). These descriptive terms are approximate and depend on the joint spacing relative to the dimension of the excavation.

In this study, we have investigated the effect of joint spacing on the size and shape of the disturbed zone around a tunnel. A 2-D distinct element code, UDEC, is used to model the tunnel excavation in a simply jointed rock mass. An analytical method was also used to verify the numerical results. Rock masses ranging from intact rock to heavily jointed rock (joint spacing less than 1/16 of tunnel diameter) are studied. The influence of boundary condition and *in situ* stress condition on the disturbed zone is also studied.

2. UDEC MODELS

The models have the dimension of 56 × 56 m, which facilitates the excavation of a tunnel with a diameter of 20 m in the centre of the model. Two sets of persistent joints, both dipping 45° but being perpendicular with each other, cut the model into blocks with regular shapes. Joint spacing varies from 7.2 to 1.2 m in different models and the number of blocks in the models ranges from 250 to 10,000. Four models are used in studying the effect of joint spacing. They are (Fig. 1):

Model	Joint spacing (m)	Number of blocks
No. 1	7.2 m	250
No. 2	3.6 m	1000
No. 3	1.8 m	4000
No. 4	1.2 m	10000

In this study, the joints are assumed to be Mohr–Coulomb joints, i.e. elasto-perfectly plastic joints. The blocks are treated as elastic blocks. The properties of the rock blocks and joints are listed in Table 1.

For all the above four models, a stress condition of $\sigma_v = 20$ MPa and $\sigma_h = 5$ MPa is assumed. These values represents the gravity-induced stresses at a depth of about 700 m. For model No. 3, an additional stress state ($\sigma_v = 20$ MPa and $\sigma_h = 10$ MPa) is also applied in order to study the influence of stress state on the disturbed zone.

Models No. 1–4 are assigned roller boundary condition for all the boundaries except the top one on which stresses are applied instead. Two additional calculations are carried out with model No. 3 to study the sensitivity of boundary conditions. The two additional boundary conditions used are: stress boundaries and mixed stress–displacement boundaries. The first one represents the boundary condition usually used in laboratory tests, where the loading stresses are ensured while the block movement is not limited. The second one is to apply a stress boundary for the first few hundred cycles in UDEC (initial loading without equilibrium) and then change to roller boundaries. This is a technique

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Fiber reinforced shotcrete simulation using the discrete element method

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ABSTRACT: Fiber reinforced shotcrete has been widely used as part of permanent tunnel support during the last 15 years especially in connection with the application of the Norwegian Method of Tunnelling (NMT). The interaction of the fiber reinforced shotcrete and the rock bolt reinforcement can now be numerically modelled with the Distinct element method (DEM). The discontinuous code UDEC (Universal Distinct Element Code) is used to investigate the overall stability of an excavation, to predict the expected stresses and deformations caused by the excavation and to investigate the optimal excavation sequence to be followed. The jointed rock geometry of Hyundai's shallow test tunnel in jointed biotite gneiss has been considered for demonstrating the fiber reinforced shotcrete, S(fr), subroutine. The results have shown that by using S(fr) and subsequently rock bolts as primary support in the tunnel, the load attained by some of the rock bolts is reduced by approximately half compared to the case were only rock bolts were used.

1 INTRODUCTION

The Norwegian Geotechnical Institute (NGI) of Oslo has been involved in a joint effort with Itasca Consulting Group for establishing an algorithm for improved simulation of the behaviour of fiber reinforced shotcrete S(fr) in multiple layers in underground structures. A special S(fr) subroutine that was developed by Itasca and financed by NGI has been incorporated in UDEC (the two dimensional Universal Distinct Element Code). In NGI's modelling work the UDEC-BB version is generally used. This is a special version of UDEC that includes the Barton - Bandis joint constitutive model (Barton and Bandis 1990).

A project that NGI and Hyundai Institute of Construction Technology (HICT) were involved in 1996 in Seoul has been chosen as an example to demonstrate the use of S(fr) in UDEC-BB. Modelling work was performed simultaneously in NGI and Hyundai and in situ measurements have been taken to be compared with the numerical results. The work involved a tunnel in Hyundai's test station (span 5.4, height 6.4 m) in biotite gneiss.

2 THEORETICAL BACKGROUND FOR THE FIBER REINFORCED SHOTCRETE

The structural elements in UDEC can be used to model the effect of fiber reinforced shotcrete on any rock surface. The area of application of the shotcrete is specified and UDEC automatically creates the elements necessary to represent a uniformly applied layer. The material behaviour model associated with the structural element formulation in UDEC simulates the inelastic behaviour representative of many common surface-lining materials. This includes non-reinforced and reinforced cementitious materials, such as concrete and fiber-reinforced shotcrete, that can exhibit either brittle or ductile behaviour as well as materials such as steel, that behave in a ductile manner. The behaviour of the material model used for S(fr) can be shown on a moment-thrust interaction diagram, see Figure 1. Moment-thrust diagrams are commonly used in the design of concrete columns. These diagrams illustrate the maximum force that can be applied to a typical section for various eccentricities (e). The ultimate failure envelopes for non-reinforced and reinforced cementitious materials are similar. However, reinforced materials have a residual capacity that remains after failure at the ultimate load. Non-reinforced cementitious materials have no residual capacity.

1997

Quantitative Description of Rock Masses for the Design of NMT Reinforcement

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ABSTRACT

The different merits of TBM, and drill-and-blast tunnelling are compared, together with the support design philosophies of NATM (analyse-monitor) and NMT (analytical-empirical). Details of the NMT method are given, including the investigation, design, execution and contractual aspects. Improved methods have been developed for interpreting seismic data, where the velocity - Q-value relationship is modified by depth and rock strength and porosity. Extensive recent data on tunnel convergence and Q-values for tunnels of different size indicate a simple relationship between span, Q-value and convergence, which can be used to assist in confirmation of support class when tunnel logging. The method can also be used in back-analyses to estimate stress ratios. A simple relationship between RMR and Q allows stand-up time to be estimated, which can be useful in assessing TMB problems.

INTRODUCTION

Slow development, evolution and occasional revolution could be used to describe the developments made in the last 100 years of tunnelling. It may be reasonable to claim that the invention and development of the TBM, the road header, the hydraulic drill, rock bolts and shotcrete have each revolutionised the practice of tunnelling. Within each class there have been important evolutions, such as earth pressure balance (EPB) machines, rock bolts with plastic sheaths (CT) and shotcrete with fiber reinforcement S(fr), to name just a few.

Methods of tunnel design have also developed slowly, but there has been evolution and occasional revolution here also. The use of empirical design methods has evolved following slow developments, and the use of displacement monitoring likewise. Possibly we would be correct in describing discontinuum modelling as a revolution in relation to earlier continuum modelling.

In parallel with tunnelling methods (e.g. TBM, roadheader or drill-and-blast) and tunnel design (e.g. empirical or analytical or instrumental) there seem to have developed some fairly distinct schools of tunnelling which utilise different principles. Each get the job done but different speeds of construction (m/week) and different costs ($/m) are an inevitable consequence.

1998

8th International IAEG Congress/8ème Congrès International de AIGI © 1998 Balkema, Rotterdam, ISBN 90 5410 990 4

Excavating in weak rocks with the Norwegian Method of Tunnelling (NMT)

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ABSTRACT: The updated Q-system for rock mass classification and support selection and the use of modern materials such as wet process fiber reinforced shotcrete, S(fr), anticorossive bolts, and reinforced ribs of shotcrete, RRS, are essential elements of the Norwegian Method of Tunnelling (NMT). The Norwegian Geotechnical Institute of Oslo has been involved in a joint venture with Contractor C.J.Sarantopoulos S.A and Consulting Group Axon Ltd for the study and construction of the first tunnel project in Patras, Greece, using principles from the Norwegian Method of Tunnelling (NMT). This project comprises the construction of a bypass highway, with twin tunnels, scheduled to open by year 2001, in weak marl formation with sandy interbeddings. The twin tunnels with an approximate length of 650 m have a designed pillar thickness of 16 m. The Q - system was used for the classification of the rock mass which could be characterised as extremely poor to very poor with Q - values ranging between 0.01 and 0.3.

RESUME: Le système Q de classification des masses rocheuses et de sélection de la méthode de soutènement, ainsi que l'utilisation de matériaux modernes tels que le béton projeté renforcé avec des fibres d' acier, S(fr), le boulonnage anticorrosion, le cintres en gunite renforcés, RRS, sont des éléments essentiels de la méthode norvégienne de construction des tunnels (NMT). L'Institut de Géotechnique Norvégien d'Oslo a été impliqué dans un projet multilatéral avec le maître d'ouvrage C.J. Sarantopoulos S.A. et le bureau d'études Axon Ltd pour l'étude et la construction d' un premier projet de tunnels à Patras, Grèce, basé sur les principes de la méthode NMT. Le projet comprend la construction d'une voie d'autoroute avec deux tunnels jumelés, dont l'ouverture est prévue pour 2001, dans des formations marneuses peu résistantes avec des bancs sablonneux. Les deux tunnels, d'une longueur approximative de 650 mètres, ont une épaisseur de pilier de 16 mètres. Le système Q a été utilisé pour la classification de la masse marneuse, qui peut être caractérisé comme extrêmement faible à très faible, les valeurs Q variant entre 0.01 et 0.3.

1 INTRODUCTION

Due to uncertainties in connection with the ground conditions revealed in core logs, a pilot tunnel 40 m in length, in a nearby location of the twin tunnels with nearly quadratic cross section with dimensions 2 × 2 m, was first excavated. Several in situ tests such as plate loading tests for determination of the E modulus, deformation measurements for the elastic response of the rock mass and bolt pull out tests for the determination of shear strength of the material were performed. All these in situ tests provided

Several extensometers were installed in three different locations in the test tunnel. The roof extensometers of Section C at 35.0 m from the entrance reached values of about 13 and 11 mm at 1.5m and 2.5m from the arch crown respectively, before they were stabilised. Surprisingly good results were derived from the bolt pull-out tests on site. The five tested fully grouted bolts of effective grouting length of only 1.25m were able to take loads ranging between 7.9 and 17.2 tnf. Failure on the pull-out tests occurred between grouting and rock. The maximum shear stress during the bolt pull-

1998

TBM performance estimation in rock using Q_{TBM}

Nick Barton, Technical Adviser, NGI, Norway, Visiting Professor at the University of São Paulo, Brazil, has developed a new method for predicting penetration rate (PR) and advance rate (AR) for TBM tunnelling. This method is based on an expanded Q-system of rock mass classification and average cutter force in relation to the appropriate rock mass strength. Orientation of fabric or joint structure is accounted for, together with the compressive or point load (tensile) strength of the rock. The abrasive or non-abrasive nature of the rock is incorporated via the University of Trondheim cutter life index (CLI). Rock stress level is also considered. The new parameter Q_{TBM} can be estimated during feasibility studies, and can also be back calculated from TBM performance during tunnelling.

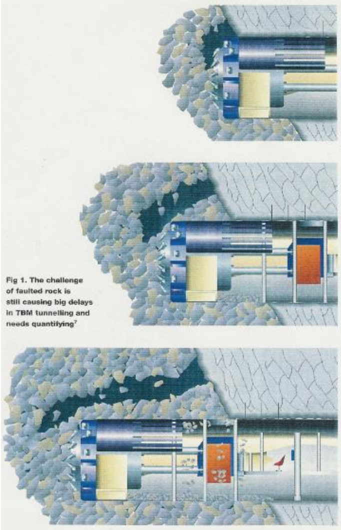

Fig 1. The challenge of faulted rock is still causing big delays in TBM tunnelling and needs quantifying?

TBM tunnelling may give extremes of 15km/year and 15m/year, sometimes even less. The expectation of fast tunnelling places great responsibility on those evaluating the geology and hydrogeolgy along a planned tunnel route. When rock conditions are reasonably good, a TBM may be two to four times faster than drill+blast. The problems lie in the extremes of rock mass quality, which can be both too bad, as in Fig 1, and too good (no joints), where alternatives to TBM methods may be faster.

There has been a long-standing challenge to develop a link between rock mass characterisation and essential machine characteristics such as cutter load and cutter wear, so that surprising rates of advance (or slowness) become the expected rates. Even from a 1967 TBM tunnel Robbins[7] could report 7.5km of advance in shale during four record breaking months. Yet, earlier in the same project, 270m of unexpected glacial debris had taken nearly seven months. Advance rates (AR) of 2.5m/h that can decline to 0.05m/h in the same project need to be explained by a quantitative rock mass classification.

A penetration rate (PR) pushing 10m/h for short periods is so different from an advance rate through a major regional fault zone as slow as 0.005 m/h that a large range of quality seems to be required. The new parameter Q_{TBM} can range over 12 orders of magnitude but each end of the scale is exceptionally unfavourable for progress and project economy.

1999

ROCK MASS CHARACTERIZATION FROM SEISMIC MEASUREMENTS

by Nick Barton
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Technical Adviser, NGI, Oslo, Norway

1. INTRODUCTION

"Nature has left us an incomplete and often well-concealed record of her activities, and no "as constructed" drawings"! These introductory remarks from Stapledon and Rissler (1983) who were General Reporters at the ISRM Congress in Melbourne can be utilized as one of the justifications for performing geophysical surveys. Before drilling begins at a site we must produce preliminary plans of investigation that will produce useful guidelines for the next more detailed stage of investigation. If a model is already available for converting seismic velocities into preliminary rock engineering data (rock quality, deformability, rock support needs, etc.) we can focus the next phase of drilling and associated testing more clearly on a set of objectives. The objectives will generally be to optimise the safety and economy of that which is to be constructed. Low velocity and potentially high permeability zones will be the natural focus of attention, though in a TBM tunnelling project we may also be concerned by too much high velocity rock, due to the slow progress made in hard, sparsely jointed rock.

In this connection, a velocity of 2.5 km/sec for massive chalk marl of high porosity will have entirely different consequences to that of a regional fault of the same velocity crossing a Japanese high speed rail tunnel, and delaying progress by months, while world record speeds of boring are achieved in the chalk marl, perhaps even 1.5 km/month. The natural velocity of the unjointed rock under in situ conditions (Sjøgren et al. 1979), and the contrast seen in low velocity zones is the main index of difficulty, since an order of magnitude reduction in Q-value (rock quality) may accompany each 1.0 km/sec reduction in seismic velocity.

2. SHALLOW REFRACTION SEISMIC

Shallow refraction seismic measurements for measuring first arrival, compressional P-wave velocities close to the surface can give a remarkable picture of near surface conditions due to some fortuitous interactions of physical phenomena. Firstly, weathering and the usual lack of significant stress near the surface has allowed joint systems, shear zones and faults to be exaggerated in both their extent and severity. Secondly stress levels are low enough to allow joints and discontinuities to be seismically visible due to their measurable apertures. So-called acoustic closure occurs at greater depths than those usually penetrated by conventional hammer seismic, unless rock strengths are rather low.

The example of joints in chalk marl at the Chinnor Tunnel in the UK closing at about 15 meters depth (Hudson et al. 1980) to give a stable 1.6 km/sec field velocity (Figure 1) can be contrasted to the saturated joints in gneiss at the Gjøvik cavern in Norway, which gave a continuous rize in velocity from 3.5 to 5.5 km/sec in the first 50 meters depth due to increased stress, yet had almost unchanged rock quality (Barton et al. 1994).

1999

General report concerning some 20th century lessons and 21st century challenges in applied rock mechanics, safety and control of the environment

Rapport général concernant quelques leçons du 20ième siècle et défis du 21ième siècle en mécanique des roches appliquée et sûreté et contrôle de l'environnement

Bericht über angewandte Felsmechanik, Sicherheit und Kontrolle der Umwelt-Erfahrungen aus dem 20. Jahrhundert und Herausforderungen für das nächste Jahrhundert

NICK BARTON, Nick Barton & Associates, Oslo, Norway & Visiting Professor, USP, São Paulo, Brazil

ABSTRACT: Application of rock mechanics in civil and mining engineering is reviewed, based on perceived weaknesses and strengths, and based on the wide range of topics presented at the Paris Congress of ISRM. Arguements are put forward for making improvements in some basic areas such of stress transformation in dilatent materials, and in constitutive modelling of rock masses, both of which may be missing some basic concepts of behaviour. The wide reaching effects of dilation and anisotropic properties and boundary conditions are emphasised. Rock mass classification and empirical design is also reviewed. Such methods are the inevitable consequence both of the complexity of rock masses and of the world-wide volume of construction activities in jointed rock. Useful and simple links between classification and input data for design and verification are emphasised, using an extended Q-system and a recent development called Q_{TBM}. Continuum and discontinuum modelling are compared. It is concluded that the modelling of the components; rock, rock joints, and discontinuities is far more logical and technically relevant than present "black-box" continuum models. Excavations larger than boreholes usually mobilize joints or fabric in their response which is often anisotropic, and neglect of this represents a serious and uneccessary error. Coupled behaviour adds to the misconceptions that may be spawned by inappropriate continuum modelling

RÉSUMÉ: L'application de la mécanique des roches au génie civil et minier est passée en revue, selon les faiblesses et points forts ressentis, et la gamme étendue des sujets présentés au congrès de la SIMR à Paris. Quelques arguments sont avancés pour améliorer certains points dans des domaines de base, comme la transformation des contraintes dans les matériaux dilatants, et les lois de comportement des massifs rocheux, tous deux des domaines dans lesquels des concepts de base du comportement pourraient être manquants. Les effets innombrables de dilatation, propriétés anisotropes et conditions aux limites sont soulignés. Les systèmes de classification du massif rocheux et de la conception empirique sont aussi passés en revue. Ces méthodes sont la conséquence inévitable à la fois de la complexité des massifs rocheux, et du volume mondial de construction en milieu fissuré. Des relations simples et utiles entre méthode de classification et données d'entrée pour la conception et vérification sont soulignées, en utilisant le système Q et un récent développement appelé Q_{TBM} Des résultats de modélisation continue et discontinue sont comparés. Il est conclu que la modélisation des composants, roche, joints et discontinuités est, de beaucoup, plus logique et appropriée que les modèles existants continus type "boîte-noire". Les excavations plus larges que des trous de forage sollicitent lors de leur réponse, souvent anisotrope, les joints ou fabrique de la roche, et le fait de négliger cet aspect représente une erreur sérieuse et inutile. Les comportements couplés ajoutent aux erreurs de jugement, qu'une modélisation continue inappropriée pourrait décupler.

Zusammenfassung: Die folgende Ausführungen geben einen Überblick über Anwendung der Felsmechanik in den Bereichen Bau- und Grubeningenieurwesen, auf der Grundlage bekannter Schwächen und Stärken und im Hinblick auf die Fülle von Themen, die auf dem ISRM Kongress in Paris vorgestellt wurden. Verbesserungsvorschläge wurden in einigen Grundbereichen gemacht, wie z.B. Spannungsumlagerungen in dilatierenden Materialien und Modellierung des Spannungsdehnungsverhaltens von Fels, welche beide grundliegende Mängel in der Handhabung aufweisen können. Die weitreichenden Auswirkungen von Dilatanz, anisotropen Eigenschaften und Grenzbedingungen werden hervorgehoben. Ausserdem werden die Klassifikation von Fels und empirische Designmethoden besprochen. Solche Methoden sind eine notwendige Folge der Komplexität des Fels und des weltweiten Umfangs der Bauaktivitäten in geklüftetem Fels. Die nützliche und einfache Verbindung zwischen Klassifizierung und Eingangsdaten wird an Hand des erweiterten Q-Systems und der Neuentwicklung Q_{TBM} hervorgehoben. Kontinuierliche und diskontinuierliche Simulierungen werden verglichen. Der Autor kommt zu der Schlussfolgerung, dass die kombinierte Simulierung der Komponenten, Fels, Klüfte und Trennflächen weitaus mehr logisch und technisch relevant ist, als herkömmliche "black box" kontinuierliche Modelle. Ausschachtungen, grösser als Bohrlöcher, verursachen normalerweise eine Reaktion der Klüfte und der Matrix. Die Ausserachtlassung dieser Reaktionen stellt einen ernsthaften und unnotwendigen Fehler dar. Gekoppelte Prozesse und auf irrtümlichen Annahmen beruhende kontinuierliche Simulierungen, tragen weiterhin zum magelhaften Verständnis der oben beschriebenen Vorgänge bei.

1999

Rock joint and rock mass characterization at Sellafield

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Nick Davies
Mount Isa, Australia

Alan Hooper
United Kingdom Nirex Limited, UK

ABSTRACT: The NGI methods of characterizing rock joints and rock masses were utilized extensively in geological investigations undertaken by United Kingdom Nirex Limited (Nirex) at Sellafield to determine whether it was suitable as the location for a deep geological repository for radioactive wastes. In addition to the standard rock mechanical laboratory testing of joints, coupled shear flow testing (CSFT) was also performed on natural rock joints for obtaining the magnitude of joint conducting apertures. The objectives of the CSFT tests were to produce site specific data, albeit on small scale samples, so that the effects of normal and shear stress changes, closure and shearing, could be evaluated and compared with the patterns of behaviour predicted by numerical modelling of the disturbed zone which would be caused by excavation of disposal caverns.

Preliminary rock reinforcement designs for the conceptual disposal caverns were derived from the Q-system statistics. Numerical modelling using UDEC-BB was carried out for predicting the behaviour. The purpose of numerical modelling was to investigate the potential stability of various sizes of rock caverns and in particular the rock reinforcement (predicting bolt loads and rock deformations), the extent of the disturbed zone (joint shearing and hydraulic aperture) with respect to cavern orientation, the effect of pillar widths, and the effect of cavern excavation sequence.

1 INTRODUCTION

The NGI methods of characterizing rock joints (using JRC, JCS and ϕ_r) and characterizing rock masses (using the Q-system of Barton et al, 1974) formed the basis for NGI's participation in the site characterization programme at Sellafield to determine whether the site was suitable as the location for a deep waste repository for the disposal of the UK's intermediate-level and certain low-level solid radioactive wastes (Nirex, 1997). A special geotechnical logging chart (see Fig. 1)was developed for recording and presenting key engineering geological parameters including the data required for rock mass classification purposes (Q-system). This PC based chart has allowed the data logged from different areas around the project site to be combined enabling input data files to be set up for numerical modelling of sections of the underground excavations. Advanced rock mechanics testing of joints which include coupled shear flow conductivity tests (CSFT) were performed on natural joints from the sedimentary and volcanic rocks. The CSFT testing apparatus, which was designed by NGI, helped derive the experimental data needed to quantify the effect of joint deformation on conductivity (Makurat et al, 1990). Rock reinforcement designs were evaluated using the Norwegian Method of Tunnelling (NMT) concepts (Barton et al, 1992).

2 JOINT CHARACTERIZATION AT SELLAFIELD

2.1 *Joint shear strength parameters*

Index tests to determine JRC (tilt tests, pull tests and profiling), JCS (Schmidt hammer tests), ϕ_r (tilt tests, pull tests and Schmidt hammer) were carried out on joints recovered in the 96 mm diameter drill core. The NGI methods of tilt testing and Schmidt hammer testing are described in detail by (Barton and Choubey, 1977 and by Barton and Bandis, 1990).

The original form of the non-linear «JRC - JCS» criterion for predicting the shear strength of rock joints (Barton and Choubey, 1977) is written as:

$$\tau = \sigma_n \tan\left[JRC \log\left(\frac{JCS}{\sigma_n}\right) + \Phi_r \right] \qquad (1)$$

1999

Rock mass classification for choosing between TBM and drill-and-blast or a hybrid solution

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ABSTRACT: The speeds of TBM tunnelling and drill-and-blast tunnelling are compared, using the new Q_{TBM} model for TBM performance estimation, and the conventional Q-value for drill-and-blast prognoses. By using these two methods it can be estimated whether a hybrid solution might be the most economic and timely. For instance one would drill-and-blast the most problematic ground, if early access was feasible, while waiting for TBM delivery. A hybrid solution was used at the 18 km long Qinling Tunnel in China, and is also planned in Brazil, where abrasive, massive rocks occur at both ends of the tunnel. Logging methods that can conveniently be used to describe the ground, including the use of seismic, are described in this paper, together with some of the details of the Q_{TBM} method, including a worked example.

1 INTRODUCTION

The pressing need for fast tunnelling solutions for infrastructure development has naturally focussed attention on TBM tunnelling. In hydropower development an even more obvious need for TBM tunnelling is apparent, due to the potentially favourable smooth profile obtained if the rock mass has favourable properties.

Western countries noted with interest the recent introduction of two large TBM into China for a planned 27-month completion of the 18.5 km long Qinling rail tunnel. The hard granites and very hard gneiss reportedly gave best penetration rates of about 4 m/hr but slowed to only 0.3 m/hr in the hardest gneiss. Besides the reportedly massive rock, an overburden as high as 1600 m, and averaging 1000 m, probably played its part in slowing the machines. Utilisation was less than 30% in a 24-hour day on average, and cutter wear was significant (Wallis, 2000).

A political decision to drill-and-blast the central section of the tunnel to bring forward completion deadlines, while the two TBM completed 5.3 and 5.6 km from the N and S portals conveniently focuses attention on our subject "Choosing between TBM and drill-and-blast". A hybrid solution combining the benefits of both methods of tunnelling should always be carefully assessed beforehand, and compared to the single solutions of one (or two) TBM, or drill-and-blasting alone. How best to make this assessment?

2001

Nick BARTON y Eystein GRIMSTAD. NORWEGIAN GEOTECHNICAL INSTITUTE.

EL SISTEMA Q PARA LA SELECCIÓN DEL SOSTENIMIENTO EN EL MÉTODO NORUEGO DE EXCAVACIÓN DE TÚNELES

1. INTRODUCCIÓN

Con una superficie de 323.000 km^2 y una población de tan sólo 4 millones de habitantes (lo que supone la densidad de población más baja de Europa), Noruega posee un nivel de construcción de túneles absolutamente inusual. Durante los últimos ocho años, se vienen excavando en dicho país más de 4 millones de m^3 de roca anuales de túneles y cavernas. Algunas de estas obras constituyen hitos importantes en el campo de las obras subterráneas, como la Caverna Olímpica de Gjøvik, de 62 m de luz, Fig. 1, los Proyectos Hidroeléctricos de Statkraft's Svartisen y Ulla Førre, el túnel submarino de Byfjord (de 5,8 km de longitud) o los túneles gemelos de Oslo.

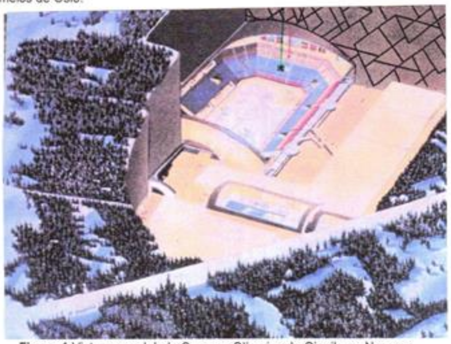

Figura 1. Vista general de la Caverna Olímpica de Gjøvik, en Noruega.

2000

Letter to the Editor

Rock Mass Characterisation and Classification

The recent collection of articles in FELSBAU 4/2001 were of considerable interest, treating as they did the controversial question of rock mass classification. As pointed out by at least one of the authors, the more and more frequent demand for numerical "verification" and the unfortunate attempts to only apply continuum analyses means that too many people are inputting an obviously anisotropic rock mass, yet using an isotropic guesstimate of properties from RMR, GSI, RMi, and Q.

As author of the Q-system, which was originally intended only to provide permanent support solutions for tunnels and caverns in jointed rock, I would join other authors in suggesting careful consideration when using the inevitably approximate estimates of properties that one can supposedly obtain from these classification methods.

Speaking only of the Q value, I would emphasise that a 1980 estimate of rock mass modulus for relatively competent rock (E m = 25 log Q) is a reasonable mean value, provided that the (probably anisotropic) most continuous jointing is discretely represented in a UDEC or 3DEC model. The modulus estimate, even an isotropic one, takes approximate care of all the smaller blocks that one cannot possibly model discretely.

Since 1995 a more accurate equation that takes more account of the obviously important effect of low (or high) uniaxial compression strengths has been available (3). This can easily predict a rock mass modulus of less than 1 GPa, thereby falling beneath the Seraphim and Pereira modification of Bieniawski, as appropriate to very young or weathered rocks. The need to discretely model principal jointing may still apply.

Alber's article (1) on anisotropy contained an obviously incorrect estimate of the Q range (mostly 13 to 17.5) which did not correspond correctly to the RMR range (mostly 60 to 65). Despite this error, the assumption of zero radial convergence using the Q estimate is clearly an impossibility – convergence is needed to give a stable arch, even if no support is needed!

A rough rule of thumb (which can be much improved) is that deformation in mm is approximately equal to span in meters divided by the Q-value. If the Q-value is improved by pre-grouting (or by pre-reinforcement!) the improved Q-value, if estimable, will be the relevant number here. Alber's assumption of squeezing – "all but the Q-classification predict squeezing tunnelling conditions" – suggests the need for more thought from the author. Is 2 cm convergence really considered a squeezing problem? Hardly.

Finally, a comment about the Editors editorial (2). Major shortcomings of classification systems like Q or RMR will easily be "documented" if the person concerned has this as his or her objective. It is also easy to try to criticize RQD if one really feels the need to discuss the dramatic consequences of 9 cm or 11 cm joint spacing!

Nevertheless RQD as a single rock mass parameter, taking Deere's instruction about competent or incompetent rock seriously, is a remarkably good starting point for classification. I suspect that there are thousands of careful users of RQD, RMR and Q who find these methods of use to their profession, despite the efforts of Schubert and Riedmüller. Are there some non-Austrian, and non-Graz users who would like to join the discussion?

Nick Barton, Oslo

References

1. Alber, M.: Incorporation of Discontinuity Related Anisotropy into Rock Mass Classification. In: *Felsbau* 19 (2001), No. 4, pp. 55-60.
2. Riedmüller, G. ; Schubert, W.: Editorial. In: *Felsbau* 19 (2001), No. 4, p. 4.
3. Barton, N.: *The influence of joint properties in modelling jointed rock masses.* Keynote lecture, Proc. of ISRM Cong. Tokyo, Vol. 3, pp. 1023-1032. Rotterdam: Balkema, 1995.

2001

An improved model for hydromechanical coupling during shearing of rock joints

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Accepted 13 December 2000

Abstract

This paper presents some experimental results from hydromechanical shear tests and an improved version of the original model suggested by Barton (Office of Nuclear Waste Isolation, Columbus, Ohio, ONWI-308, 1982. 96pp.) for the hydromechanical coupling of rock joints. The original model was developed for coupling between mechanical and hydraulic aperture change during normal loading and unloading. The method was also suggested for the coupling of shear dilation and hydraulic aperture changes. The improved model has the same appearance as the original and is based on hydromechanical shear experiments on granite rock joints. It includes both the mechanical and hydraulic aperture and the mobilised joint roughness coefficient (JRC_{mob}).

1. Introduction

As a consequence of engineering works in a rock mass, deformation of both the joints and intact rock will usually occur as a result of the stress changes. Examples of such works are repositories for radioactive waste, dam foundations, excavation of tunnels and caverns, geothermal energy plants, oil and gas production, etc. Due to the stiffer rock matrix, most deformation occurs in the joints, in the form of normal and shear displacement. If the joints are rough, deformations will also change the joint aperture and fluid flow.

Traditionally, fluid flow through rock joints has been described by the cubic law, which follows the assumption that the joints consist of two smooth, parallel plates. Real rock joints, however, have rough walls and variable aperture, as well as asperity areas where the two opposing surfaces of the joint walls are in contact with each other.

According to this, apertures can generally be defined as *mechanical* (geometrically measured such as with epoxy injection) or *hydraulic* (measured by analysis of the fluid flow).

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1.1. Mechanical aperture (E)

The *mechanical* joint aperture (E) is defined as the average point-to-point distance between two rock joint surfaces (see Fig. 1), perpendicular to a selected plane. If the joint surfaces are assumed to be parallel in the $x-y$ plane, then the aperture can be measured in the z direction. Often, a single, average value is used to define the aperture, but it is also possible to describe it stochastically. The aperture distribution of a joint is only valid at a certain state of rock stress and pore pressure. If the effective stress and/or the lateral position between the surfaces changes, as during shearing, the aperture distribution will also be changed.

Usually, the mechanical aperture is determined from a two-dimentional (2-D) joint section, which is a part compound of the real 3-D surface.

1.2. Hydraulic aperture

The *hydraulic* aperture (e) can be determined both from laboratory fluid-flow experiments [1–4], and bore-hole pump tests in the field [5,6].

Fluid flow through rock joints is often represented (assumed) as laminar flow between two parallel plates. The equivalent, smooth wall hydraulic aperture (e) can

2001

The Editor,
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11th December 2001

Dear Sirs,

Deformation moduli and rock mass characterization

A recent article in your journal by Palmstrom and Singh (16, 2001, 115-131) drew attention to the difficulties of interpreting the results of plate jacking and plate loading tests. Although one may have reservations, the article is a useful contribution to the literature. The authors compared some of the earlier and more recent classification methods e.g. RMR, Q, and RMi (the latter developed by Palmstrom) and the degree to which they correlated with the measured results from the reviewed loading tests. A potential weakness of course is the correctness of the rock mass characterization at each test site, but collection by mostly one organization may have minimised this source of error.

Two of the older correlations between deformation modulus (which we can refer to as M) and RMR and Q, date from Bieniawski, 1978 and Barton et al. 1980. These were specifically for rockmasses at the higher end of the quality scale, namely RMR>50 and Q>1. Naturally their development was limited to the data base that was used at that time. The error introduced if attempts are made to use such correlations outside the intended range of the data base are clear, and hardly need to be emphasised. The authors' Table 3, showing the effects of varying uniaxial strength from 4 MPa to 200 MPa was a clear example of the inadequacy of the 20 year-old Q-relation as a *general formulation* for all strengths of rock. This is because uniaxial strength did not appear explicitly in the 1980 formulation, as it was only designed to estimate moduli for rock masses with Q>1. As a basis for distinct element modelling of medium strength rock masses, it has proved very successful.

The Norwegian first author is aware of the published improvements and generalizations made between the Q-value, deformation modulus, and seismic velocity. He was a guest, and seminar participant at NGI in the same period as their development. Unfortunately, these widely published developments, including two ISRM congresses and an international symposium in India, were not included in the Palmstrøm and Singh

2001

A numerical study of cryogenic storage in underground excavations with emphasis on the rock joint response

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Accepted 11 September 2001

1. Introduction

One of the most important problems related to underground storage of cryogenics is to prevent leakage of liquid and gas along joints that might be caused by tensile stresses due to shrinkage of the rock mass around the caverns [1]. This view is supported by conclusions from continuum modelling of cryogenic storage in rock that repeatedly have shown the development of very large tensile stresses, that are identified to be the main driving force for cracking. Cavern depths of prohibitively large magnitude are needed to counteract these tensile stresses through the development of high initial tangential stresses following excavation.

The use of numerical methods to simulate the mechanical behaviour of rock masses requires knowledge of the scale dependent stress-displacement behaviour of the rock joints. In general, behaviour close to an excavation in jointed rock will tend to be quite different from that predicted by isotropic, continuum models [2]. In this region, one can expect the greatest gradients of stress and deformation, which clearly calls for an appropriate non-linear description of joint behaviour. Since the behaviour of individual joints was expected to significantly affect the size of the region of cracking, a discontinuum code was used for the modelling. The Universal Distinct Element Code UDEC-BB [3] was selected, which has the ability to calculate non-linear joint opening/closing/displacement response to stress changes, according to the Barton–Bandis constitutive model [4–6].

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The present study considers greatly simplified geometries; the objective was to investigate the separate effect of different physical conditions such as jointing, stress anisotropy and depth. Consequently the rock mass was strongly idealised and some of the material parameters were kept constant under different temperatures and stress levels. Apart from one simulation, the cooling was applied instantaneously which was believed to be the worst case. Simplifications were made in order to gain understanding, rather than to make absolute predictions.

One of the most difficult questions is the behaviour of joints with cryogenic fluid. If the storage is unlined and frozen down to $-162°C$, when the rock is cooled and the joints start to open, say more than a couple of millimetres, a part of the gas flows into the joints and continues cooling inside the rock wall. This will open the joints successively, and heavily increase the cooled area and the extent of the cooling front. However, modelling such processes of connected cooling-fluid behaviour were beyond the scope of this study. An important goal was to investigate the effect of non-linear joint behaviour on the magnitude of joint opening and tensile stress development caused by the general rock shrinkage during cooling. In this way it may be correct to say that the numerical study has been done from the rock point of view, rather than from the fluid point of view.

Eight realisations have been modelled, with single caverns of 25 m diameter at 100 or 500 m depth in massive rock, with only two or three idealised, continuous joints intersecting the caverns. In three of the realisations, at 100 m depth, the stress ratio was varied systematically using ratios of $\sigma_H/\sigma_v = 0.5$, 1.0 and 2.0. Another two of the realisations were run with an extra oblique joint crossing the excavation. In an

2001

The Editor,
Prof. Marc Panet,
The ISRM News Journal,
Paris

jkbart@usinternet.com

11th December 2001

Dear Sirs,

Water and stress are fundamental to rock mass *characterization* and *classification*.

A recent report from a rock mass classification workshop held in Australia in 2000, which was reported by Palmstrøm, Milne and Peck in your ISRM News Journal (August 2001, 6,3 : 40-41) may leave the impression that the subject of rock mass classification has been definitively debated, and that an important conclusion has been reached for the profession to follow. Regrettably, this desirable goal has clearly not been achieved in this forum, as we will try to demonstrate. Of concern, perhaps only to the undersigned, is the fact that the developers of two of the principal classification methods under discussion, were not able to give possible alternative points of view. Perhaps this *will be permitted* in the pages of your ISRM News Journal.

The key dilemma is whether the 'internal' and 'external' so-called boundary conditions of water and stress should be permitted in a rock mass characterization scheme. The latter might be RMR, Q or the more recent RMi of Palmstrøm. The problem is particularly relevant here, since RMi does *not* include water and stress, and this new but complex method *was* represented by its developer in Melbourne.

Firstly, one can address rock mass *classification* for preliminary empirical design of rock mass reinforcement and tunnel support. Here it seems to be generally agreed that water and stress parameters are 'allowable', and in fact are necessary components of the classification. Their possible wide-reaching effects in excavation design are clear, and many users understand the necessity of, for instance, Jw and SRF in the Q-system. The close correlation between the six-parameter Q-value and the necessary quantities of B + S(fr), were shown by Grimstad and Barton, 1993, in a major Q-system support-recommendation update, which now includes some 1260 case records.

Secondly, there is the problem of rock mass *characterization* of relevant rock units at appropriate depths for a future rock engineering project. Prior to excavation, the 'virgin' rock mass qualities and correlated design parameters like deformation modulus, may be required as input data for modelling. The correct interpretation of seismic velocities obtained from shallow refraction seismic, or from deeper reaching VSP, or from focussed

2001

GUEST EDITORIAL

Destructive criticism - constructive thoughts.

The violent events that we have recently witnessed, as always, are due to different points of view of what is right and what is wrong. The 'right' to a certain point of view, and a destructive action, will nevertheless require a vigorous response. Let us draw some lighter-hearted parallels, closer to our profession.

Those who criticise a supposedly benign entity like RQD, can have a field day demonstrating what happens if joint spacing is a uniform 9cm in one stretch of core, and 11cm in the next. The same argument can be applied to RMR and Q. But we have a right to respond vigorously in defence of Deere's RQD, if we nevertheless believe that the 'core stick >10cm' technique, lies in a useful range for differentiating most of our rock engineering problems.

Readers of ISRM News Journal will recently have seen the opinion that joint spacing, number of joint sets and RQD do a poor job of quantifying block size. An alternative method (unlike RQD) is capable of quantifying block sizes of 1000, 10,000 or even 100,000m^3, as if this was the region where we needed solutions. We have a right to resist such anarchy, and defend the traditional point of view that joint spacing, and number of joint sets (already sufficient) and RQD, actually give a very good quantification of block size, especially in the area where our problems lie.

Your benign and colourful sub-continent is being violently pushed towards the north, throwing up constant challenges to hydropower developers in the lower Himalayas. The signs of a violent past are everywhere evident, but you resist these forces with admirable vigour, and accept the need for flexibility – like the bulldozer-drivers on constant stand-by, ready to use a new sector of the creeping hillside for today's road. Visitors fortunate enough to sample just a few of the problems, come away with hydrogeological lessons that last a lifetime.

A TBM that grinds to a halt in massive quartzite, that later gets buried in sand, gravel and water needing a dedicated drainage tunnel, that gets squeezed in phyllite, and finally succumbs to permanent burial in a deep, unplanned graveyard, bears witness to heroic struggles, which are sometimes only solved by admitting defeat. Comparing these realities with another of nature's wonders, the Sugar Loaf monolith in Rio de Janeiro, which has not been challenged by continental plate collisions, it is clear that we really need classification and characterization methods that stretch over many orders of magnitude. They also need to be fully coupled, to tackle the huge ranges of properties exhibited by present-day hydrogeologies – following millions of years of geometric turmoil, and too many millennia in the rain and sun.

2001

North American Tunnelling 2002, Ozdemir (ed.)

Rock joint sealing experiments using an ultra fine cement grout

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Nick Barton
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Axel Makurat
Shell International Exploration and Production, The Netherlands

ABSTRACT: Rock joint sealing experiments have been conducted in a laboratory using a bi-axial coupled shear- flow test apparatus (CSFT). The laboratory test programme was designed to investigate the penetrative potential of a grout using different water/cement ratios on joints having different joint roughness (JRC) and joint conducting aperture in different stress conditions (total normal stress, joint water pressure and grouting pressure). The results indicate that joints with a conducting aperture (e) as small as approximately 25 microns can be grouted using a stable mixture of superfine cement, water and a super plasticizer (dispersing agent). The penetration capacity of a specific cement grout depends, in addition to the joint's characteristics, on the maximum grain size, the water/cement ratio and the injection pressure used. The tests reveal that the minimum physical aperture (E) that can be grouted corresponds to approximately four times the cement's maximum grain size.

1 INTRODUCTION

In many cases water leakages are governed by flow along the joints. An understanding of how the groundwater moves in rocks is one of the most important factors in the solution of rock engineering problems. This is especially true with regards to the planning and design of tunnels, storage caverns and underground waste disposals. Concerning nuclear waste repository safety a key aspect is the confidence of being able to successfully seal underground excavations and demonstrate methods of reducing the permeability of adjacent rock by sealing joints and fissures.

This paper describes the laboratory sealing experiments conducted on rock joints using a unique testing equipment designed by NGI. The equipment, called coupled shear flow temperature testing (CSFT) apparatus, has basically been used to derive the experimental data needed to quantify the effect of joint deformation on joint conductivity (Fig. 1). With the CSFT apparatus, joints can be closed, sheared and dilated under controlled normal stress conditions and at the same time cold or hot fluids can be flushed through the joint. Deformations, flow rate and stresses are recorded simultaneously. The CSFT test is designed to simulate as closely as possible the in situ state of critical joints and its modification by increases or decreases in normal and/or shear stress. In the present series of tests cement grout mixture was injected in the joint samples with increasing injection pressures. The rate of grout flow and the injection pressure versus time were recorded simultaneously to study the penetrability of a grout.

2 PENETRATION POTENTIAL OF GROUT MIXES

From a rheological point of view, a grout mix corresponds to a Bingham body exhibiting both cohesion and viscosity. A stable grout mix is defined as a mix having virtually no sedimentation e.g. less than 5% sedimentation in 2 hours (Lombardi, 1985). Water on the other hand follows Newton's law and is therefore a Newtonian body due to its viscosity and its lack of cohesion. A Newtonian fluid is represented by the following equation:

$$\tau = \eta \; dv/dx \qquad (1)$$

where τ=shear stress (Pa); η= kinematic viscosity (Pa·sec), dv/dx= strain rate (sec^{-1})

A Bingham body or a stable mix is represented by the following equation:

$$\tau = c + \eta \; dv/dx \qquad (2)$$

where c= cohesion (Pa)

2002

PERGAMON

International Journal of Rock Mechanics & Mining Sciences 39 (2002) 185–216

International Journal of Rock Mechanics and Mining Sciences

www.elsevier.com/locate/ijrmms

Some new *Q*-value correlations to assist in site characterisation and tunnel design

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Accepted 5 February 2002

Abstract

The rock mass quality *Q*-value was originally developed to assist in the empirical design of tunnel and cavern reinforcement and support, but it has been used for several other tasks in rock engineering in recent years. This paper explores the application of *Q* and its six component parameters, for prediction, correlation and extrapolation of site investigation data, and for obtaining first estimates of some input data for both jointed distinct element and continuum-approximation modelling. Parameters explored here include P-wave velocity, static modulus of deformation, support pressure, tunnel deformation, Lugeon-value, and the possible cohesive and frictional strength of rock masses, undisturbed, or as affected by underground excavation. The effect of depth or stress level, and anisotropic strength, structure and stress are each addressed, and practical solutions suggested. The paper concludes with an evaluation of the potential improvements in rock mass properties and reduced support needs that can be expected from state-of-the-art pre-injection with fine, cementicious multi-grouts, based on measurements of permeability tensor principal value rotations and reductions, caused by grout penetration of the least favourable joint sets. Several slightly improved *Q*-parameter ratings form the basis of the predicted improvements in general rock mass properties that can be achieved by pre-grouting.

1. Introduction

The traditional application of the six-parameter *Q*-value in rock engineering is for selecting suitable combinations of shotcrete and rock bolts for rock mass reinforcement and support. This is specifically the permanent 'lining' estimation for tunnels or caverns in rock, and mainly for civil engineering projects.

The *Q*-value is estimated from the following expression:

$$Q = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\mathrm{SRF}} \qquad (1)$$

where RQD is the % of competent drill-core sticks >100 mm in length [1] in a selected domain, J_n is the rating for the number of joint sets (9 for 3 sets, 4 for 2 sets, etc.) in the same domain, J_r is the rating for the roughness of the least favourable of these joint sets or filled discontinuities, J_a is the rating for the degree of alteration or clay filling of the least favourable joint set or filled discontinuity, J_w is the rating for the water inflow and pressure effects, which may cause outwash of discontinuity infillings, and SRF is the rating for faulting, for strength/stress ratios in hard massive rocks, for squeezing or for swelling.

The above ratings, and some important new footnotes, are given in full in Appendix A. The three quotients appearing in Eq. (1) have the following general or specific role:

RQD/J_n is the relative block size (useful for distinguishing massive, rock-burst-prone rock), J_r/J_a is the relative frictional strength (of the least favourable joint set or filled discontinuity), and J_w/SRF is the relative effect of water, faulting, strength/stress ratio, squeezing or swelling (an 'active stress' term). An alternative combination of these three quotients in two groups only, has been found to give fundamental properties for describing the shear strength of rock masses. This aspect will be described towards the end of the paper, after exploring a number of simple correlations between engineering parameters and *Q*-values, the latter normalised to the form Q_c, for improved sensitivity to widely varying uniaxial compression strengths.

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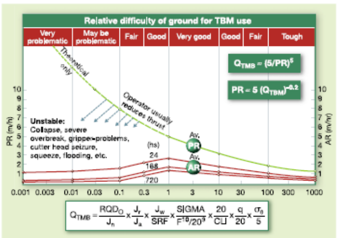
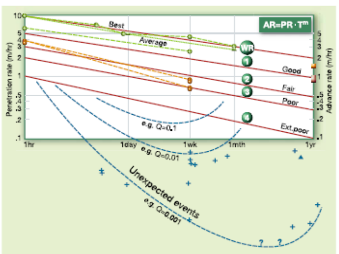
1365-1609/02/$ - see front matter
PII: S1365-1609(02)00011-4

2002 (Errata: see Pr, p. 21, equation 15, corrected in 2nd download)

Employing the Q_{TBM} prognosis model

Dr Nick Barton, of Nick Barton & Associates, and geologist Ricardo A Abrahão, of Fundação de Ciências Aplicadas e Tecnologia Espaciais (FUNCATE), explain the workings of the Q_{TBM} tunnelling prognosis model, using a variety of geological conditions that give big ranges of tunnelling performance. Their main example reinforces the idea of hybrid tunnelling when great contrasts of conditions are found (see companion article in T&TI, June 2003)





Top: Fig 1 - Some principles and definitions of the Q_{TBM} model, that are now incorporated in the numerical version of Q_{TBM}
Above: Fig 2 - The (usual) law of deceleration as time increases. Typical gradients of decline are m = -0.15 to 0.25, except in fault zones where (m) is steeper, when Q is <0.1

In a previous article by the first author, entitled 'TBM or Drill and Blast' (*T&TI*, June 2003), mention was made of the QTBM prognosis model. This is a time saving Excel program, developed by co-author Ricardo Abrahão, for estimating the consequences of TBM tunnelling in the differing geological and rock mechanics properties of a site. The input for this model are a variety of engineering geological parameters that are commonly collected during project feasibility and design phases. The output are estimates of penetration rates (PR), and actual advance rates (AR) over calculated periods of time, as each zone (or geological/structural/rock mechanics domain) of given length are penetrated, and supported as necessary.

The program calculates (and graphs) the overall performance, and estimated time of tunnel completion. We will show various examples, and discuss both strengths and weaknesses, and areas where industry input from new TBM designs needs to be incorporated now, and of course in the future.

Basic elements of Q_{TBM}

There are those who think that the standard Q-system for describing rock masses has too many parameters, and others that feel it is already too much of a simplification. The fact is, that we all need design assistance – whether from a life-time of experience, half a life time, or from models, or from trusted 'rules-of-thumb'.

The Q_{TBM} model has a lot of parameters, for which no apology is made. The rock-machine interaction in TBM tunnelling is very complex after all. But with its foundation in the analysis of 140 TBM case records[1], and frequent application since then, a certain feeling for its strengths and weaknesses has been acquired.

Two important diagrams for a rapid understanding of the principles need to be reproduced here for ready reference. They have been seen before by *T&TI*'s readers. Figure 1 shows an example of the basic model with PR and AR on left and right axes, and the 'logarithmic' scale of Q_{TBM} defined along the bottom axis. Figure 2 shows log PR and log AR on left and right axes and log TIME along the bottom axis, showing 1 day, 1 week, 1 month, etc.

Deceleration

The classic equation relating PR and AR via utilisation (U) is refined as follows in the Q_{TBM} model, to allow for the important factor of time (and tunnel length):

2003

P-03-29

KFM 01A

Q-logging

Nick Barton, Nick Barton & Associates

March 2003

Summary

The first Forsmark potential repository site borehole KFM 01A provided core from 101.8 to 1000.7 m depth. This was independently Q-logged by NB&A during a two-day period (19th–20th February, 2003), without access to BORMAP results or regional jointing frequencies or orientations. The Q-logging was intended to be an independent check for subsequent BORMAP-derived Q-parameter estimation.

The Q-logging was accomplished using the manually-recorded 'histogram method' which allows the logger to enter Q-parameter ranges and depths directly into the appropriate histograms, which facilitates subsequent data processing using Excel spreadsheets. Successive pairs of core boxes, which contain an average of 11 meters of core in ten rows, were the source of ten opinions of each of the six Q-parameters, giving a total of 4920 recordings of Q-parameter values for the 164 core boxes.

Data processing was divided into several parts, with successively increasing detail. The report therefore contains Q-histograms for the whole core, for four identified fracture(d) zones combined as if one unit, and then for the whole core minus these fracture(d) zones. This background rock mass quality is subsequently divided into nine depth zones or slices, and trends of variation with depth are tabulated. The four identified fracture(d) zones, which are actually of reasonable quality, are also analysed separately, and similarities and subtle differences are discerned between them.

The overall quality of this first core is very good to excellent, with Q(mean) of 48.4, and a most frequent Q-value of 100. The range of quality is from 2.1 to 2130, which is the complete upper half of the six order of magnitude Q scale. Even the relatively fracture(d) zones, representing some 13% of the 900 m cored, have a combined Q(mean) of 13.9 and a range of quality of 2.1 to 150.

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2003 (Note misprint on P. 33: $\sigma_c/100$, not $\sigma_c/10$)

Failure Around Tunnels and Boreholes and Other Problems in Rock Mechanics

Letter to the ISRM News Journal
from Nick Barton
7 October 2003

Introduction

At the recent 10th ISRM Congress held in South Africa, thanks to a well organized and different from normal format, there was ample room for discussion and numerous points of view could be expressed. The dominant "workshop format" with 5 minute presentations was an excellent "innovation" for ISRM — first experienced by the undersigned in an even better format, in an unforgettable U.S. Symposium in Minnesota in 1976—where 2 hours of prepared and spontaneous discussions of a few minutes each were presented within each main theme.

Congresses have more delegates these days, and multi-sessioning is unfortunately the inevitable result — making choice of session a major headache, as there is so much of interest. This was certainly true of the 10th ISRM Congress.

A preliminary discussion of brittle failure

Thanks to sterling work by workshop coordinators in South Africa, many of the topics for discussion were thought provoking and extremely relevant to the further development of our subject. Both in the "Rock Fracture" and "Numerical Modelling" workshops, the topics of failure modes arose — yet again — sandwiched between, by chance, a very topical plenary presentation on "stress-strength" induced failure mode observations when excavating the Lötschberg Tunnel — specifically when TBM tunnelling from the Steg portal (Rojat et al. 2003).

The depths of the "dog-ear" type failures — at 3 and 9 o'clock due to the vertical principal stress, compared favourably with a semi-empirical model (Kaiser et al. 2000) involving the ratio of σ_{max}/σ_c, where as usual, $\sigma_{max} = 3\sigma_1\text{-}\sigma_3$. Continuum FEM modelling using a Hoek-Brown failure criterion, reportedly gave a degree of "match" to the depth-of-failure observations in the tunnel, and to the empirical model ***if*** a rule-of-thumb was used that the "damage limit" is reached when the contours of the ratio of principal stress difference ($\sigma_1\text{-}\sigma_3$) to uniaxial strength (σ_c), are of the order of 0.33 (as suggested by Martin et al. 1999).

It has long been known that brittle failure around tunnels initiates when the ratio of stress to strength is a certain ***fraction*** of 1.0, whether the "stress" term is defined as σ_1, or $\sigma_1\text{-}\sigma_3$, or σ_θ the maximum tangential stress. Two of these three ratios have been important components of the Q-system too, in deciding upon appropriate SRF values for selecting support for stress-slabbing problems in numerous deep tunnels in Norway and elsewhere.

It is perhaps long overdue that we acknowledge that continuum models, with conventional soil mechanics derived strength criteria, are missing the realities of rock failure, which by the nature of intact rock and failed rock can be considered to occur perhaps in two parts — breakage of cohesion (localization) at small strain and mobilization of friction at larger strain. A Mohr Coulomb type of law may work well for a material that is already "particulate," but perhaps not very well where significant "multi-megaPascal" breakage is to occur, despite the presence of some jointing.

There are some very encouraging recent efforts—and achievements—in modelling the reality of failure around excavations, using for example, *cohesion softening* and *frictional strengthening* devices in continuum codes such as FLAC, and *"stress corrosion"* devices in particulate PFC models. Cundall, Diederichs, Kaiser and Martin and co-workers, Hajiabdolmajid, and Christine Detournay are among the growing number of prominent names in this new field of realism. There are also several other innovations in modelling, such as use of the *tensile strain criterion* of Stacey, the linear elastic and time-dependent *fracture mechanics* methods of Baotang Shen and co-workers in FRACOD, the use of tessellation patterns in displacement discontinuity models by Napier,and the recent *elastic-brittle plastic elemental degradation* modelling reported by Fang and Harrison. Each seem to be producing quite realistic models of rock failure processes, such as pillar failures and borehole and tunnel failures — without needing any more, to choose a contour of "stress/strength" where failure is "anticipated."

At this same 2003 ISRM Congress, Stacey presented a very thought-provoking discussion on how small the above stress/strength ***fraction*** can be at failure, in many different practical cases, thus emphasising the need for improved understanding and modelling of *relevant* failure processes. (Stacey and Yathavan, 2003.)

Modelling failure with continuum models and conventional failure criteria

A carefully excavated and well documented case record—the AECL-URL *line-drilled* test tunnel—with its classic 11 o'clock and 5 o'clock break-outs, has been one of the favourite objects of modelling, as illustrated in Figure 1, which is a composite of models used in a recent Martin et al. 2002 review of brittle failure modelling made for SKB in Sweden.

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2003

DIFFERENT SCALES OF DISCONTINUOUS BEHAVIOUR IN PETROLEUM ENGINEERING

Nick Barton

Resumo

Na engenharia de petróleo, o comportamento de meio descontínuo ocorre numa grande variedade de escalas, embora a moda, no geral, tenha sido dirigida para o uso de modelos contínuos para representar o meio descontínuo. Na escala do reservatório, fatores de relevância na produtividade como: o importante papel desempenhado pela intensa atividade tectônica na disposição de volumes substanciais de petróleo, o surgimento de forças de cisalhamento durante a ocorrência de fenômenos relacionados com subsidência-compactação - de influência no colapso de revestimentos - ou a existência de planos conjugados de juntas, são abordados e discutidos pelo autor. As propriedades fundamentais da mecânica das rochas, são de tolerância limitada, no caso das tensões diferenciais originadas nos folhelhos sobrepostos a camadas de arenito. Estas conduzem ao aparecimento de uma tensão principal mínima no folhelho, a qual induz a formação de uma descontinuidade por tensão, considerado um dos fatores de maior importância na vedação dos reservatórios de petróleo. Na escala do furo de sondagem, quando se consideram os poços de exploracão, os planos de cisalhamento na forma de espiral logarítimica, desenvolvendo-se próximo às regiões submetidas a tensões elevadas, comprovam, sem relutância, a violação dos princípios contidos nas hipóteses sobre o comportamento de um meio contínuo, isotrópico. A teoria de Mohr-Coulomb, admitindo uma mobilização simultânea da resistência ao atrito e da coesão, constitui-se na principal fonte de erro, já que as superfícies de ruptura se desenvolvem com deformações menores do que aquelas exigidas para a mobilizacão do atrito.

Abstract

Discontinuous behaviour is occurring at many different scales in petroleum engineering, despite the fashion for continuum modelling in general. Major faulting activity that has greatly affected in-place volumes of petroleum, bedding plane shearing in a compacting-subsiding environment causing numerous casing collapses, and shearing of conjugate reservoir jointing which helps to maintain productivity in a depleting reservoir will each be examined. The fundamental rock mechanics property of limited tolerance of differential stress in cap-rock shales, and therefore higher minimum stress in the shale, giving a stress discontinuity, is one of the most important sealing requisites for petroleum reservoirs. At borehole scale the log-spiral shear planes that can develop around over-stressed sections of exploration wells, actually violate the usual assumptions of isotropic continuum behaviour. The Mohr-Coulomb assumption of simultaneous mobilization of cohesive and frictional strength is the main source of error, since failure surfaces develop at smaller strain than needed to mobilize friction.

Keywords
Fault, joint, subsidence, shearing, borehole stability, effective stress

INTRODUCTION

In this keynote lecture, a rapid journey through widely different geological and petroleum engineering scales will be made – seeing the possible effects of regional faulting across sedimentary basins, shearing bedding planes in sedimentary overburden, joint deformation in reservoir rocks, and fracturing at borehole scale. An important effect is even seen at the grain-boundary, micron-size scale of discontinuities. This is the effect of weakening caused by water when water-flooding. Some of the phenomena reviewed are positive for petroleum production, others are negative

As in much of engineering design, we will often be comparing stresses and available material strength, or more precisely – *induced effective stresses* and the available strength of the rock or rock discontinuities. There is an interesting principal involved in this area, which is fundamental to today's presence or absence of oil and gas in petroleum reservoirs. Sometimes we will also be comparing fluid pressures and existing minimum rock stresses, to investigate hydraulic fracturing potential.

UNEQUAL PETROLEUM RESERVES

A start can be made by broadly comparing the petroleum fortunes of Russia and Norway in the 1,000,000 km^2 Barents Sea region of the Arctic north. There are proven reserves of 'only' 0.3 billion Sm^3 (oil-equivalent volume) on the Norwegian side (already about to be developed at Snøhvit). By comparison there are proven reserves of 8 billion Sm^3 on the Russian side (and possibly as much as 100 billion Sm^3 yet-to-be discovered reserves). This huge inequality obviously deserves some attention, since the Russian's *proved* reserves are already three times more than all the British and Norwegian North Sea reservoirs together [1]. At a petroleum conference in Northern Norway in 1989, Western oil experts were first convinced that the Russians had added one too many zeros, in their description of the 3.2 *trillion* m^3 reserves of gas in their newly discovered Shtockman reservoir, which is of the order of 30x40 km in area.

Understanding why oil and gas is, or is not found, despite available source rocks, and a similar level of drilling (50 exploration wells) and extensive seismic investigations (costing a total of about USD 2.5 billion during 25 years, for

2003

The Q-system following thirty years of development and application in tunneling projects

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ABSTRACT

Some key recent developments in using the Q-system for describing rock mass quality and some rock mass parameters are described. Links to seismic P-wave velocity, deformation modulus and simple rock mass cohesive and frictional strength terms CC and FC are discussed and tabulated. The updating of permanent support recommendations is taken two stages further, with specification of energy absorption requirements for the S(fr), and with dimensioning rules for RRS or rib reinforced shotcrete arches.

1 INTRODUCTION

A sound engineering approach should always precede construction in rock. Due however to the complexity of rock masses there is a need for a defensible simplification of the multitude of conditions actually present. One therefore builds a model of the site conditions which, though simplified, is a reflection of the main classes of rock conditions expected to be present. By collecting information from surface logging, sub-surface (core and boreholes) logging, and remote sensing such as refraction seismic and cross-hole seismic tomography, a fairly comprehensive picture of the likely range of conditions and of dominant rock classes is developed, and then successively updated and refined during excavation. This in a nutshell is the framework within which the Q-system is applied. It now has a 30 year international track record, thanks to interest by many engineering geologists and rock engineers in a method for realistically synthesising key information, whether from core logging investigations, or from logging of conditions revealed during excavation. Parts of key publications from the S(fr) shotcrere updating in 1993/94, and extensions of the system in 1995 and 2002/03 are briefly summarized, to help track the improvements.

2 Q-SYSTEM INTERPRETATION FROM SUPPORT NEEDS

The source of the Q-method of quantitatively describing rock masses was some 210 case records (Barton, Lien and Lunde, 1974), mainly describing the need for shotcrete (mesh reinforced cases from the 1960's and 1970's), and fully grouted rock bolts for permanently stabilising tunnels and caverns. A major updating of the Q-system based support recommendations was published by Grimstad and Barton in 1993, with 1050 new case records, this time involving S(fr) i.e. steel-fiber reinforced shotcrete in place of the older mesh reinforced S(mr). Grimstad collected these case records over many

2004

Underground pressures

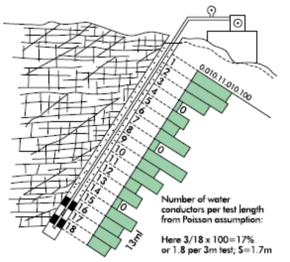


Fig 1: Lugeon testing and zero flow sections as a percentage of the total

N Barton and E Quadros offer an understanding of high pressure pre-grouting effects for tunnels in jointed rock

NORWEGIAN UNLINED pressure tunnels took many years to reach heads of 1000m. It has also taken many years to reach 10 MPa injection pressures when pre-grouting ahead of tunnels, where inflows need to be controlled to say 1 or 5 l/m/100m, or where tunnel stability needs improvement (and both of the above).

Do we know the actual effects of this high pressure injection on the rock mass? Can effects be quantified in any way? It has been found from recent Norwegian tunnelling projects that high pressure pre-injection may be fundamental to a good result: i.e. much reduced inflow and improved stability. The pressures used are far higher than have traditionally been used at dam sites, where in Europe, Brazil and the US, maximum grouting pressures (for deep dam foundations) have been limited to about 0.1, 0.05 and 0.023MPa/m depth respectively (Quadros and Abrahão, 2002).

According to a recent report by Klüver (2000), a shallow tunnel in phyllite with 5m of cover was injected at invert level to a final pressure of 6.5 MPa, and to 5 MPa even at the shallow depth of the arch, only 5m below the surface. (However, establishment of an outer screen was advised by Klüver in such extreme situations.)

The reality is that while grout is still flowing , there is such a steep pressure gradient away from the injection holes (from logarithmic to linear depending on joint intersection angle) that 'damage' to the rock mass is limited to local, near-borehole, joint aperture increase. On at least one joint set there may be local shear and dilation. Each of these effects are probably in the region of small fractions of a mm, judging by the local grout take of the rock mass, which may be about 1 to 5litres/m^3 of rock mass.

Basic elements of Snow's method

Figures 1 and 2 show how, using Snow (1968), one can make a preliminary estimate of the mean spacing of water- conducting joints, using Lugeon tests and the assumption of their Poisson distribution down the borehole. A further key simplifying assumption is that the water conductors can be roughly represented by a cubic network of parallel plates, i.e. the conductors only. There are many more joints found in cores through most rock types, due to limited connectivity.

In Figure 3 a simplifying attempt to represent 'reality', using the isotropic model of Snow (1968) is illustrated. The reality may be anisotropic and less homogeneous. It is further emphasised that in reality, stress transfer across the joint walls is required. Because of points of contact, and tortuous flow, and actual rough joint walls, the average physical aperture (E) which is potentially groutable, is usually larger than (e) the hydraulic aperture.

Assuming the cubic law is sufficiently valid for engineering purposes that we can ignore non-linear or turbulent flow, we can write permeability $K = e^2/12$ for one parallel plate, while

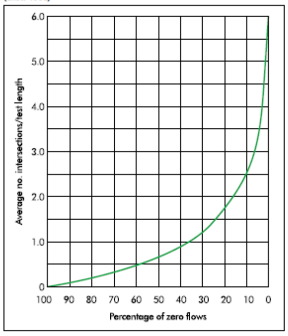
$$K_1 = {}^{e^2}\!/_{12} \times {}^{e}\!/_{S} \qquad (1)$$

for one set of parallel plates of mean spacing (S). Snow (1968) further assumed that the 'rock mass permeability' would be constituted, on average, by flow along two of the three sets of parallel plates. Thus:

$$K_{mass} = {}^{2e^2}\!/_{12} \times {}^{e}\!/_{S} = {}^{e^3}\!/_{6S} \qquad (2)$$

Making further simplifications that 1 Lugeon ≈ 10-7m/sec ≈

Fig 2: Poisson distribution for interpreting average number of water conductors (Snow 1968)



2004

Course on Geotechnical Risk in Tunnels
Aveiro, Portugal 2004

Fault Zones and TBM

Nick Barton, (Nick Barton & Associates)

Abstract

This paper is based on the lecture with this title that was given at the Aveiro Course on Geotechnical Risk in Tunnels in 2004. The present subject matter: *fault zones and TBM*, is richly illustrated with case record figures and pictures, mostly from the personal experiences of the author. The following main topics are covered:

1. Fault zone experiences in TBM tunnels in Italy, Greece, Kashmir, Hong Kong and Taiwan. 2. Fault zone cases in the Qtbm data base. 3. Attempts to be prepared using seismic and core logging and the Qtbm prognosis model.

Introduction

Three views of the world's first TBM tunnel in the UK Channel Tunnel Folkestone Warren area investigated by Beaumont in 1880. 1) A gravity-induced wedge fall-out in the chalk-marl. 2) The same tunnel with increased (cliff-induced) overburden showing stress-induced failure. 3) The same tunnel under the sea, with pore-pressure induced roof failures to bedding planes, with the added effect of ***time****.*

Figure 1. Three types of failure in the world's oldest TBM tunnel (Beaumont, 1880).

2004

Interpretation of exceptional stress levels from back-analysis of tunnelling problems in shallow basalts at the ITA Hydroelectric Power Project in S.E. Brazil

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Keywords: hydropower, rock stress, cracks, rock failure, Q-classification, anisotropy

ABSTRACT: A 1450 MW hydroelectric project was recently completed and commissioned in south-eastern Brazil. Extensive and unexpected rock engineering problems occurred during construction. A narrow basalt ridge separating a long meander of the Uruguai River was the site of the project, which was potentially very favourable due to the relatively short length of the five diversion and five pressure tunnels through this 150 to 200 m high ridge of basalt. Extensive tangential stress related popping and spalling was experienced when driving the auxiliary 15×17m diversion tunnels, both in the arch and invert, even when the depth was as little as 50m. The horizontal stress, trending NE in the region, appeared to have been seriously concentrated in the narrow N-S oriented ridge, and further concentrated in two massive basalt flows having highest Q-value. When the 9m diameter pressure tunnels: mostly as 53° shafts were driven, tangential stress related popping occurred again during excavation, but the greatest problem occurred when contact-grouting the reinforced concrete linings, which caused tensile cracking and extensive needs of repair, due to augmentation of the negative, effective tangential stress. The negative effect of stress fracturing was experienced again when operating the emergency spillway for just a few hours. Extensive N-S and horizontally oriented stress fracturing caused an unexpected high rate of scour in the unlined chute. The paper discusses the likely magnitudes of the major horizontal stress, based on back-analysis and some in situ tests. An interesting phenomenon was the measurement of highest permeability in the most massive basalt flows, due presumably to tensile cracking caused by the exceptional horizontal stress anisotropy which may have exceeded 20:1 or even 25:1.

1 INTRODUCTION

The UHE Ita hydroelectric project is located on the Uruguai river, in southern Brazil. Construction started in March 1996 and the first turbine was commissioned in June 2000. Installed capacity is 1450 MW, provided by five 290 MW Francis turbines fed by 9,0m diameter, 120,0m long power tunnels inclined at 53° (designated TF-1 to TF-5). At the project site the Uruguai river has a sharp bend, designated Volta do Uvá, where the river describes an 11 km long meander. This special geomorphology favoured a very compact layout of the project which is detailed in Figure 1.

River diversion was accomplished through five tunnels: two main tunnels (TD-1 and TD-2) 14,0m wide by 14,0m high, with control gates, permanently in operation, and three auxiliary tunnels (TD-3, TD-4 and TD-5) 15,0m wide by 17,0m high, without control gates, which operate during floods.

An auxiliary spillway VS-2, with 4 gates, is located over the upper-diversion tunnels (TD-3, TD-4 & TD-5), in a way that the stilling basin coincides with the downstream out-flowing portals of these three tunnels.

The Project was built through a "Turn Key Lump Sum" contract by CONITA (Consorcio Ita), a partnership lead by the ODEBRECHT Group and formed by CBPO Engenharia Ltda (civil works), TENENGE (assembling), ABB/ALSTOM/VOITH/COEMSA-ANSALDO/ BARDELLA (eletro-mechanical equipment) and ENGEVIX Engenharia (design). The authors were consultants to the civil contractors of Consorcio Ita, and are grateful for the opportunity to be involved in these challenging and unusual rock mechanics problems.

2004

The why's and how's of high pressure grouting - Part 1

Nick Barton, of Nick Barton & Associates, explains the theory behind high pressure-injection of stable particulate grouts ahead of tunnel faces in jointed, water-bearing rock. Although opinions differ about the need for high pressure, its effectiveness has been proved in numerous projects. Conversely, the use of insufficient pressure has often led to wet and less stable tunnels

A recent consultant's article (Pells, *T&TI* March, p34[10]) and an experienced contractor's reactions (Garshol, *T&TI* May, p36[8]) has again put focus on the results obtained by pre-grouting ahead of tunnels. Garshol emphasised the need to use high pressures to get an acceptable result. The proviso is of course that the work is done ahead of the face (as in most of Garshol's excellent grouting case records) and not from behind the face where lower pressures have to be used (as in most of Pell's case records).

With a long-standing rule for injection pressure gradients of 0.23 bars/m depth for dam foundation grouting in the US, but usually higher elsewhere, it is clear that there will be reactions when 50 to 100 bars is recommended by an experienced contractor for a stretch of tunnel whose 20m depth suggests only 5 bars.

The reasons for performing high pressure (50 to 100 bars) injection when pre-grouting ahead of tunnels is that inflows have to be controlled, perhaps down to 1 to 2lt/min/100m. Permeabilities lower than 10^{-8}m/s or lower than 0.1 Lugeon are implied – and these are also achieved when owners and consultants become aware of what is achievable today, with stable ultrafine and microcements and the vitally necessary additives like microsilica and plasticizers.

It has been found from recent Norwegian tunnelling projects that high pressure pre-injection may be fundamental to a good result, i.e. much reduced inflow and improved stability. The pressures used are far higher than have traditionally been used at dam sites, where in Europe, Brazil and the US, maximum grouting pressures (for deep dam foundations) have been limited to about 0.1, 0.05 and 0.023MPa/m depth respectively (Quadros and Abrahão, 2002[12]).

According to a recent report by Klüver (2000[9]), a shallow tunnel in phyllite with 5m of cover, with

Drilling holes for successful high-pressure pre-injection at the Jong-Asker rail tunnel west of Oslo
Left: What can happen without these measures!

2004 (Note: correct author's article title in Part 2)

The theory behind high pressure grouting Pt 2

Here Nick Barton, of Nick Barton & Associates, concludes his two part article explaining the theory behind high pressure pre-injection of stable particulate grouts ahead of tunnel faces in jointed, water-bearing rock

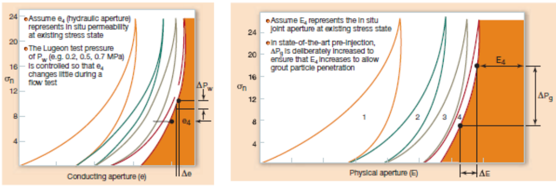
Effects of high pressure on joint deformation

Improvements due to high pressure pre-injection?

Three dimensional effects

Before pre-grouting $Q = \frac{30}{6} \times \frac{1}{2} \times \frac{0.5}{1} = 0.8$

After pre-grouting $Q = \frac{50}{6} \times \frac{2}{1} \times \frac{1}{1} = 17$



Bottom left and right: Fig 7 - An illustration of grouting pressure effects on joint aperture changes. Δe during a Lugeon test is supposed to be small, while a ΔE during high pressure grouting is supposed to be large. But this is only locally around each hole due to logarithmic-to-linear pressure decay

2004 (Note: this is the correct title for both-parts)

A critique of QTBM

Dr Nick Barton introduced the Q_{TBM} as a new prediction model for TBM performance in T&TI, 1999[1], followed up by several papers also in T&TI about its characteristics and use[2,3,4]. Dr Olav T Blindheim has serious doubts about several of the basic assumptions and finds that the method is clouding rather than clarifying the complex interaction between ground conditions and TBM performance

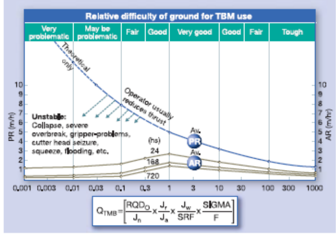


Fig 1 – Penetration Rate (PR) and Advance Rate (AR) as a function of QTBM (Barton, 2003[4])

Penetration rate (PR)

Content of QTBM

RQD_0/J_n ("block size")

2005

Risk and risk reduction in TBM rock tunnelling

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Keywords: tunnels, TBM, risk, rock quality, characterization, seismic velocity, stress.

ABSTRACT: There are many, many potential sources of geotechnical risk in rock tunnels. To state the obvious first, *unexpected discoveries* of significant fault zones, adversely oriented planar clay-coated joints, very weak rock, very hard massive rock, very abrasive rock, very low or high stress, high volumes of stored water and high permeability are clearly among the foremost risk factors. Their partial combination in a given tunnel can be catastrophic. Through appropriate pre-investigation measures, using a combination of *sufficient* surface mapping, refraction seismic, core logging, borehole testing, and their extrapolation by means of geo- and hydro-geological rock mass characterization, the above risk factors can clearly be minimized. However, these risk factors can seldom be eliminated and so-called *unexpected events* may still occur. If in sufficient numbers, both schedule and cost, and the continued existence of a TBM excavation choice (or even the contractor) may each be threatened, in a given tunnel. Clearly, the deeper the tunnel in relation to these mostly near-surface investigation methods, the greater the need for sufficient planning and (unit) pricing of contingency measures for tackling the *unexpected*. Depth effects on seismic velocity also need careful consideration, as a false sense of improved rock quality with depth may be experienced.

1 INTRODUCTION

After a tunnel collapse or TBM cutter-head blockage in a tunnel, it usually becomes quite clear to the experienced tunnelling engineer or engineering geologist what the cause(s) of the collapse or blockage were. *Before* the event it would usually be necessary to have been exceptionally pessimistic to have foreseen the 'unthinkable' – which more often than not is the combination of several adverse factors, which separately are 'expected' though serious events, but when combined are, quite logically, 'unexpected events'.

In the following table, some of the more obvious sources of geotechnical risk are tabulated as reference to the cases cited in the paper.

significant fault zones, adversely oriented planar clay-coated joints,
very weak rock, very hard massive rock, very abrasive rock,
very low stress, very high stress, exceptional stress anisotropy,
high volumes of stored water, high permeability

Sometimes of course, an *individual* risk factor may have been of such magnitude that it could not reasonably have been predicted. Below are listed some cases which are familiar to the writer, where either an *unexpected combination of factors* led to temporary or final failure of the projected tunnel excavation method, or alternatively an *unexpected magnitude of a single factor* led to the problem encountered – which could equally well be multiple problems as a result of this single factor.

A short list of TBM tunnels that suffered (catastrophically) from multiple unexpected events

1. Unpredicted fault swarm parallel to valley-side, together with very high (and fault-eroding) water pressures, at depths of 700-900m. TBM tunnel (diameter 5m) eventually ran sub-parallel to individual faults, each requiring at least half a year for each 1m wide fault (AR ≈ 0.005m/hr). TBM finally abandoned; new contractor for D+B from other end of tunnel. (*Pont Ventoux HEP, N. Italy*).
2. Alternating massive quartzite (minimum PR ≈ 0.2m/hr), talcy sheared phyllites ('over-excavating' and stand-up time limitations), and fractured quartzite 'aquifer'. Early blow-out of 4000 m3 rounded gravels at 750m depth and

2004

Comments on 'A critique of Q_{TBM}' by Blindheim

Dr Nick Barton replies to the recent critique of the Qtbm model given by Blindheim in T&TI, June 2005. He disputes many of the comments of Blindheim, acknowledges some limitations of the Q_{TBM} prognosis model, and explains some improvements that are planned. He believes that Blindheim has misunderstood several aspects of TBM and of the Q_{TBM} method.

As a developer of something new, it is inevitable that one virtually 'invites' critique by those who assume they are 'the establishment'. Ideally both parties, and the advancing subject matter, benefit from the process. It is to be hoped that this will be the case here.

In this reply to Blindheim will be found elements of agreement, much dissent, and places where one must assume that our opinions differ due to experience of contrary behaviour.

In his introduction to the need for practical and reliable TBM performance models, Blindheim claims that I attempt to cover the 'whole range' (geological/geotechnical factors, machine factors, organisational factors). While both the 'geo-factors' are squarely addressed in the Q_{TBM} method and in Barton, 2000, there has (of course) been no attempt to address organisational factors, unless he means I should not address hours/week etc. Apart from (average) cutter thrust, TBM diameter, utilization (%) and support needs, other machine factors are presently neglected. The important open or single-shield, double-shield aspect will be discussed later.

Blindheim proceeds through the parameters used in the Q_{TBM} model in a thorough manner, mostly as translated from his 2002 article in Norwegian. Unfortunately he starts by giving the first version of the 'Q_{TBM}' formula in his Figure 1, from an early chapter of my book, not the final version nor that coded in the numerical Q_{TBM} -model.

RDQ_o/J_n ('block size')

Based on Palmström et al. 2002 and earlier Palmström opinions, Blindheim claims that RQD 'is insensitive for high and low frequencies of joints', and 'RQD/J_n poorly characterises the block size'. In fact, RQD/J_n has remarkable sensitivity to situations causing blocked buckets, blocked schutes, and clogged or damaged conveyors. In particular it is its combination with the inter-block shear strength (i.e. J_r/J_a) that is important.

Blindheim appears not to fully accept that when the Q-parameters are used both for PR prognosis, through Q_{TBM}, *and* for utilization and AR prognosis, through the gradient of *deceleration* (-)m, Q may act in (legitimately) 'opposite' directions. Fast PR for short periods can be associated with a low utilization and low actual advance rate AR, *in the same tunnel length*, due to the stop for support.

Figure 1 shows the core-logged values of RQD/J_n from two campaigns of site investigation in Brisbane in 1993-95, and 1998. The '+3/-3' columns, as an example, represent 3m above to 3m below the (future) tunnel level. The 'blocky rock' problems occurred when relative block size (RQD/J_n) and inter-block shear strength (J_r/J_a) were *both* below that predicted. This latent, and unexpected condition was a part of the basis for successful claims by the contractor.

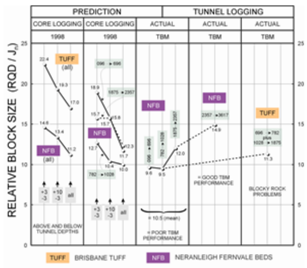

Fig 1 – A demonstration of reduced RQD/J_n ratios in a TBM tunnel compared to those predicted. Barton 2004.

Fig 2 – An exposure of the Brisbane NFB meta-sediments that illustrates the importance of a combined RQD/J_n and J_r/J_a on block release. PR *and* AR may be affected in opposite directions: easier boring, subsequent delays.

When promoting the idea of volumetric joint count (J_v = *sum* of number of joints/m for each set) in place of RQD and J_n, both Blindheim and Palmström should realize that one cannot assume that RQD ceases to have value when *average* blocks are below 10^{-3} m^3 (the 10 cm 'limit') or above about 0.4m^3. The reality of the rock mass is a distribution of block sizes, which 'stretches' the use of RQD a long way into both low and high J_v. RQD *does not* cease to have value when the *average* block size is about 10cm. At the other end of the

2005

12th IWSA, Beijing 2006

Fracture-induced seismic anisotropy when shearing is involved in production from fractured reservoirs

N. Barton
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ABSTRACT
Conducting, 'open' joints, fractures or microcracks parallel to the classic direction $\sigma_{H\,max}$ are commonly referred to in the geophysics literature. They are the focus for most of the shear wave polarization studies, and are often assumed to be stress-aligned microcracks. Nevertheless, measurements in deeper wells reported during the last 10-15 years by Stanford University researchers, do not show conducting joints parallel to the 'classic' direction $\sigma_{H\,max}$. The *non-conducting* fractures in these deep wells are in the directions relative to $\sigma_{H\,max}$ that are normally assumed to be *conducting directions* in geophysics literature. The conducting joints in deep wells are found to be consistently in conjugate directions, bisected by the 'classic' $\sigma_{H\,max}$ direction, so shear stress may therefore be acting to assist in their permeability. Numerous fractured reservoir cases in fact show 20º to 40º rotations of the polarization axes of qS_1 and qS_2, relative to interpreted $\sigma_{H\,max}$ directions, possibly because more than one set of fractures is present, as expected in most rock masses. Shearing induced by reservoir production and compaction, on one or more sets of fractures, is also known to be an important contributor to the maintenance of permeability in the face of increased effective stress. Shearing of conjugate sets of fractures is also considered by the author as a potential source of the temporal rotation of seismic anisotropy and attenuation, as recently recorded in 4D seismic at the Ekofisk and Valhall reservoirs in the North Sea.

1 INTRODUCTION

A standard assumption in the geophysics literature is that shear wave polarization and splitting occurs due to stress-aligned structure. This structure has been considered by many to be stress aligned microcracks, by others with reservoir interests, as a desirable 'open' set of sub-vertical conducting fractures that are also assumed to be parallel or sub-parallel to the maximum horizontal stress.

2006

TBM TUNNELLING IN SHEARED AND FRACTURED ROCK AND THE APPLICATION OF Q_{TBM} MODEL CONCEPTS

Nick Barton, Nick Barton & Associates, Oslo, Norway

ABSTRACT

Sheared and faulted rock encountered at great depth, and rock masses that are deeply weathered, and that are encountered when tunnelling is carried out at too shallow depth, represent frequent challenges for TBM tunnellers. In this paper, various experiences with TBM tunnelling problems will be addressed, with particular reference to fault zone and sheared zone experiences in TBM tunnels in Italy, Greece, Kashmir, Hong Kong and Taiwan, together with fault zone cases in the Qtbm data base. TBM achieve remarkable advance rates when conditions are favourable, out-performing drill-and-blast by a wide margin. However, favourable conditions are interrupted by infrequent, sometimes frequent challenges, which are not widely reported. Unless the rock mass character is in the central area of the Q-diagram on a consistent basis, with Q of about 1.0 near the centre of the distribution, marked superiority to drill-and-blast may not be achieved, especially if the tunnel is long, since there is a generally-observed gradual deceleration of the TBM tunnelling advance rate, a reduction unlikely to be seen with drill-and-blast tunnels. Double-shield TBM, designed for thrusting from PC-element liners while resetting grippers, may represent 'over-design' regarding support needs, for much of a given rock mass, but they reduce these deceleration gradients by about half. All but the most serious shear zones and faults may be tackled well by these machines.

INTRODUCTION

TBM tunnelling and drill-and-blast tunnelling show some initially confusing reversals of logic, with best quality rock giving best advance rates in the case of drill-and-blast, since support needs may be minimal, whereas TBM may be penetrating at their slowest rates in similar massive conditions, due to rock-breakage difficulties, cutter wear, and the need for too-frequent cutter change, the latter affecting the advance rate AR. This 'reversed' trend for TBM in best quality, highest velocity (V_P) rock is demonstrated by the PR-V_P data from some Japanese tunnels, reproduced in Figure 1, from Mitani et al., 1987.

As may be imagined, the advance rate (AR) is a function of opposite effects in the best rock, namely the need for frequent cutter change, yet little need or delay for support. At the low velocity, high PR end of this data set, there will not be frequent need for cutter change (slowing AR), but conversely there will be delays for much

2006

INTEGRATING Q-LOGGING WITH SEISMIC REFRACTION, PERMEABILITY, PRE-GROUTING, TUNNEL AND CAVERN SUPPORT NEEDS, AND NUMERICAL MODELLING OF PERFORMANCE.

Nick Barton[1]

Abstract: When the 'Q-system' was launched in 1974, the name referred to rock mass classification, with focus on tunnel and cavern support selection. Since that time 'a system' has indeed been developed. The Q-system now integrates investigation geophysics, rock mass characterization, input for numerical modelling, empirical design of support, and excavation performance assessment. The Q-value has proved easy to correlate with required support capacity, relative cost and time for tunnel construction, seismic P-wave velocity, deformation modulus, cavern deformation, and in modified form with permeability. Recent research has also shown encouraging links between Q, the depth dependant deformation modulus, and the *seismic quality* Q_{seis}, which is the inverse of attenuation. There are also indications that Q has captured important elements of the cohesive and frictional strength of rock masses. The above sensitivities are most likely because Q is composed of fundamentally important parameters that were quantified by exhaustive case record analysis. The six-orders-of-magnitude range is a reflection of the potentially enormous variability of geology and structural geology. Some of the empirical relationships are illustrated with a summary of Gjøvik Olympic cavern investigations, and of the *discontinuum* modelling of performance. The paper concludes with a critical assessment of the potential shortcomings of *continuum* modelling of highly stressed excavations in intact rock, and of shallow excavations in anisotropically jointed rock.

INTRODUCTION

This lecture will be an illustrated journey through some of the useful linkages and concepts that have been absorbed into the 'Q-system' during the last ten years or so. From the outset the focus will be on sound, simple empiricism, that works because it reflects practice, that can be used because it can be remembered, and that does not require black-box software solutions. Some of the empiricism will be illustrated by reference to investigations and to empirical and numerical modelling performed at the Gjøvik Olympic cavern in Norway.

Nature varies a lot and therefore Q does too

It is appropriate to start by illustrating contrasting rock mass qualities. Figure 1 shows a core box from a project that has not been *completed* during ten years of trying. The second project may not be *started* for at least ten years. The first should already have passing high-speed trains, the other high-level nuclear waste some time in the future. They are both from the same country and may have six orders of magnitude contrast in Q-value. A second pair of examples shown in Figure 2, requires a cable car for access on the one hand, and successive boat trips to fault-blocked flooded sections of tunnel on the other.

The contrasting stiffness and strength of intact rock and wet clay is easy to visualize. One may be crushed by one and drowned in the other. There are sad and multiple examples of both in the tunnelling industry. They merit a widely different quality description, as given by the Q-value.

2006

City metro tunnels and stations that should have been deeper

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M. Abrieu
CVA Consortium, São Paulo, Brazil

ABSTRACT: The first major underground construction project in the SE corner of the 17 million-population city of São Paulo is the Line 4 Yellow Line of the city metro, which is currently under construction. Following the normal, but unfortunate wish of all owner-operators for *shallow* stations and *short* escalators, the contractor is currently struggling to build 4.5 km of shallow tunnels and 5 shallow stations, as a much needed addition to the city metro. Problems encountered inevitably include mixed-face rock-saprolite conditions, deep differential weathering when there is biotite gneiss, deeply weathered *core-stone* conditions when in granite, and generally more difficult saprolite and soil conditions than anticipated by the experienced contractor, who supplemented the owner's extensive vertical site exploration with some 30 deviated boreholes. A single break-through to street level was also experienced, on this occasion caused by penetration of a very long, several hundred tons slab of gneiss through bolt and shotcrete reinforcement. The failure was caused by the smooth-planar and deeply-weathered vertical boundary jointing, and was also aided by a fully saturated saprolite cover of some 20 m thickness, with immediately preceding heavy rainfall. As usual, several adverse factors all occurred at the same time and place, forming a typical scenario for failure, fortunately without fatalities.

1 INTRODUCTION

Although shallow tunneling in tropical climates is known to frequently cause difficult tunneling conditions, there seems to be a built-in optimism for owners, contractors and consultants to assume that a new project will not suffer the fate of other projects. The possibility of break-through to surface due to too shallow siting seems to visit many well prepared projects, as fairly recent events in Porto, Portugal (Babendereide et al. 2006) and Singapore (Zhao et al. 2006) have shown. With such projects in mind, the contractor-consortium CVA in São Paulo, managed to persuade the owner São Paulo Metrô, to accept NATM-style tunneling, in place of an originally planned hybrid EPB TBM.

In retrospect, and having regard to the frequent mixed face conditions, this switch of methods is probably extremely fortuitous, despite the more numerous access shafts that were constructed, to increase the number of faces for drill-and-blast construction.

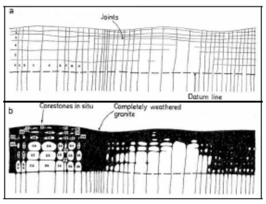

Figure 1. A price has to be payed for tunnel construction too close to the 'dotted line', illustrated from the particular case of weathered granite from Dartmoor, SW England, after Linton (1955) and Fookes et al. (1971). In general, Q-parameters are not easily determined in the Grade V (black) saprolite.

2 CONSEQUENCES OF THE NEAR-SURFACE

The classic core-stone differential weathering resulting from a previous tropical weathering, shown in Figure 1, brings with it the risk of low or zero RQD (Figure 2), low uniaxial compressive strength (Figure 3a), and low deformation modulus (Figure 3b).

2007

Anisotropy and 4D caused by two fracture sets, four compliances, and sheared apertures

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Conducting, "open" joints, fractures, or microcracks parallel to the classic direction of maximum horizontal stress σ_H are commonly referred to in the geophysics literature. In a remarkable number of these studies, stress-aligned microcracks are automatically assumed to be the source of shear-wave polarization. Fractured reservoirs, being biased samples of the "near" surface, may indicate a supplementary anisotropy, caused by a set of open fractures, again with conventional interpretation. These are also assumed to be stress-aligned.

Yet monitoring of deep wells shows fracture sets that are under shear stress as the significant conductors, such as a conjugate pair either parallel to, or intersected by, the maximum horizontal stress. Measurements in deep wells in hard crystalline rocks reported during the last 10–15 years do not show single sets of open conducting fractures parallel to the "classic" direction of σ_H. The steeply dipping fractures that are conductors in deep wells are found to be consistently in conjugate directions. They may strike parallel to the classic direction, but are acted on by shear stress caused by the inequality of σ_V and σ_H acting perpendicular to their strike.

The nonconducting fractures in these deep wells are presumably held "closed" by the resultant normal stress, which would be consistent with geomechanics modeling, unless fracture roughness and rock wall strength, and therefore also apertures, are larger. Mobilized friction coefficients μ of mostly 0.5–0.9 have been interpreted in the case of numerous deep wells with such conducting fractures. This mechanism of shear, whether prepeak or postpeak, may also occur in a downdip sense, perpendicular to strike, caused by matrix compaction in weak, porous reservoir rocks such as chalk. Here the conjugate fracturing is caused by the domal or anticlinal structures, typical of reservoirs like Ekofisk and Valhall. This we shall investigate later.

In both cases, the normal and shear compliance of both sets will be contributing to the "stress-aligned" axis of maximum shear velocity V_S, and to the strength of the anisotropy. The shear-wave splitting will therefore also be sensitive to fluid type as normal and shear compliances are mobilized. Unequal contributions caused by one dominant set may be the source of 4D rotations of anisotropy and attenuation axes. The detailed geomechanics within individual nonplanar fracture planes may be contributory here.

A question naturally arises from Figure 1 in the context of this introduction. Are there two sets or one set of fractures causing the registered anisotropy from shear-wave polarization? A significant number of fractured reservoir cases in fact seem to be showing as much as 20–40° rotations of the polarization axes of qS_1 or maximum V_S relative to interpreted σ_H directions. This is possibly because more than one set of fractures is present, as expected in most rock masses. It may also be because of the logic that fractures under shear stress are usually by far the best conductors, both from geomechanics principles, and from the actual deep well measurements referred to earlier.

At the recent 12th IWSA in Beijing, numerous examples were presented of fast shear-wave rosettes from earthquake studies in the Fujian district of China by Wu et al. (2006), and from the Beijing capita area by Gao et al. (2006). The

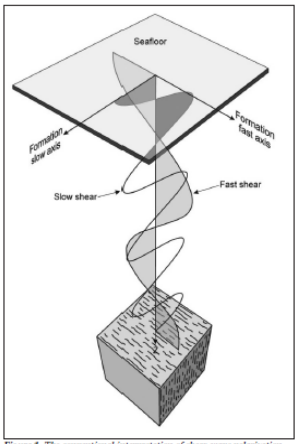

Figure 1. The conventional interpretation of shear-wave polarization, with the fast axis caused by one set of stress-aligned fractures or by microcracks. From Barkved et al. (2004).

shear-wave polarization rosettes the authors presented actually resemble the symmetries seen in structural geology fracture-set rosettes. The same has been seen in the Iceland data published numerous times by Crampin and his coauthors, but interpreted as stress-aligned microcracks.

As shown by Boness and Zoback (2007), shear-wave anisotropy may be caused by preferential closure of fracture sets that are not aligned with σ_H, and of course by the potential "openness" of the set that is major-stress-aligned. There seems to be limited reason to always assume that microcracks are involved, when there is such a lot of tectonic structure (joint or fracture sets) that may be bisected by σ_h, or by σ_H as suggested later.

The fast shear wave may be interpreted, conventionally, as parallel to a single set of fractures, when the reality may be that it is composed of two fracture-set components. The two sets are quite likely to have different density, and their roughness and wall strength (JRC and JCS, see Box 1), may also be different. This unequal two-set system, could be good reason for a nonparallel-to-stress seismic anisotropy axis. One could take this a stage further and suggest that if

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Future directions for rock mass classification and characterization – Towards a cross-disciplinary approach

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ABSTRACT: Rock mass classification has come to be associated with the selection of a rock mass quality class on the basis of prior classification or rating of various rock mass parameters. The presence of a tunnel or slope or similar is implied, and the disturbance to local rock mass characteristics, caused by the excavation disturbed zone, is supposed to be captured in the local rock class, as support will be chosen. Rock mass characterization reflects a broader mission to describe the character of a rock mass where a future project is likely to be realized, but no excavation presently exists, except of borehole scale. In this paper, cross-disciplinary examples of rock mass classification and characterization are selected from various civil engineering construction projects, making much use of seismic velocity for emphasizing the links between rock quality, deformability, permeability and velocity, and for helping to distinguish between classification and characterization. However these terms obviously overlap in common usage.

1 INTRODUCTION

Rock mass classification has come to be associated with the need to select a rock mass class on the basis of prior classification or rating of various rock mass parameters. Most frequently, classification is used in association with tunnel support, and also as a basis for payment. It may also reflect a need for pre-treatment. Implicit here is the existence of the tunnel, and the effect this may have on the expected rock mass response, in particular that within the EDZ. All of the above may also apply to rock slopes, but here post-treatment is more likely.

Rock mass characterization reflects a broader mission to describe the character of a rock mass where a future project is likely to be realized. Besides rock quality description with one of the standard measures such as RQD, or RMR, or Q, or GSI, or several of these, it should also include site characterization fundamentals such as rock stress, water pressure, permeability and seismic velocities. Ideally each of the above should be measured as a function of depth and azimuth, and of course reflect lateral variation and variation in specific domains.

Various simple index parameters of the matrix and joint sets can also considered characterization, like UCS and the JRC-JCS roughness-strength character that can be estimated during core logging. A Schmidt hammer and short ruler are sufficient equipment here. Cross-disciplinary characterization involving Q, velocity, permeability, and deformability will be used to illustrate possible future trends.

2 THE EXCAVATION DISTURBED ZONE

The cross-hole seismic description of the site for a future ship lock shown in Figure 1 (diagram a) can be considered one form of characterization of the site. The RQD, RMR and Q-values of the two cores would be essential supplementary data. Ideally, core or subsequent borehole logging should be oriented due to kinematic stability assessment needs.

Subsequent cross-hole seismic between supplementary holes shows the increasing development of an EDZ actually much better than our rock mass classification would be capable of, andthe 1 year delay between c) and d) would be hard to emulate with (predicted) reductions of RQD, RMR and Q.

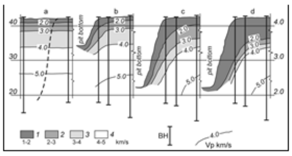

Figure 1. Cross-hole site characterization (left), and monitoring of ship-lock excavation stages. There is a one-year delay between diagrams c) and d). Savich et al. 1983.

2007

Near-surface gradients of rock quality, deformation modulus, Vp and Qp to 1 km depth

Nick Barton*

Introduction

In hard rock areas, the uppermost 50 m of the ground may consist of soil, weathered jointed rock, and increasingly sound, more massive rock as depth increases. From experience with seismic refraction work, it is well known that there are extreme seismic velocity gradients in this zone. This is so even if we discount the step increase in P-wave velocity, V_p, at the water table. There are many reasons for the rapid increases in velocity with depth. These include increased stresses, increased rock quality because less weathering has occurred, fewer open joints, less clay, and usually a reduced frequency of jointing.

Besides steep velocity-depth gradients in the top 25m, which are well into double figures when measured in units of s^{-1}, there are marked increases in the rock mass quality rating Q, corresponding increases in the rock mass deformation modulus E_{mass}, and therefore also marked increases in the seismic quality factor for P-waves, Q_p. Velocity and quality depth gradients generally reduce in steepness beyond some 100-200 m depth, but the correlations between these rock and seismic parameters are reviewed here for depths to 1 km, covering the zone of interest for civil engineering and many mining applications. The importance of these linkages is that the seismic parameters V_p and Q_p, which may be determined from seismic refraction and crosshole tomography surveys during site investigation, can be used to estimate the rock mass parameters Q and E_{mass} which are needed for engineering design. Applications include excavations in good quality rock, weathered rock, and more porous, weaker rock (Barton, 2006).

In this article, empirical relationships between the rock mass parameters used in engineering design and seismic parameters are presented with reference to the databases from which they were derived. The first section below introduces the engineers' rock mass parameters. This is followed by separate sections on the linkages between V_p and rock mass parameters at shallow depths and at greater depths, down to 1 km, and between Q_p and rock mass parameters. Finally, the relationships are illustrated by a real example.

Rock mass parameters in engineering

The rock mass quality rating Q introduced by Barton et al. (1974) is one of the standard international methods of classifying the engineering quality of rock masses, used primarily to assist in the selection of suitable combinations of shotcrete and rock bolts for rock mass reinforcement and support in tunnels and caverns, and to provide input to numerical models. It is determined from surface logging and core logging of the rock mass and has values in the range 0.001 to 1000.

Rock quality Q is defined as

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{1}$$

where RQD is the rock quality designation, defined by the percentage recovery of competent core pieces in lengths >10 cm;
the value of J_n depends on the number of joint sets;
the value of J_r depends on the joint roughness;
the value of J_a depends on the degree of joint alteration and clay filling;
the value of J_w depends on the amount of water inflow or pressure; and
SRF is the stress reduction factor which captures loosening effects due to faulting, and also the stress/strength ratio in the case of massive rock that may fracture under high stress.

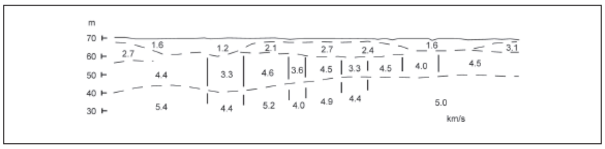

Figure 1 *Example of the steep P-wave velocity gradients seen when conducting shallow seismic refraction at a hard rock, low porosity site in Scandinavia (from Sjøgren, 1984).*

2007

ROCK MASS CHARACTERIZATION FOR EXCAVATIONS IN MINING AND CIVIL ENGINEERING

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Abstract: When the 'Q-system' was launched in 1974, the name referred to rock mass classification, with focus on tunnel and cavern support selection. Besides empirical design of support, the Q-value, or its normalized value Q_c, has been found to correlate with seismic P-wave velocity, with deformation modulus, and with deformation. The Q-system provides temporary or permanent support for road, rail, and mine roadway tunnels, and for caverns for various uses. It also gives relative cost and time for tunnel construction in a complete range of rock qualities. There are also indications that Q has captured important elements of the cohesive and frictional strength of rock masses, with Q_c resembling the product of rock mass cohesion and rock mass friction coefficient.

ROCK MASS VARIABILITY

From the outset the Q-system has focussed on sound, simple empiricism that works because it reflects practice, and that can be used because it is easily remembered.

It is appropriate to start by illustrating the widely contrasting rock mass qualities that may challenge both the civil and mining professions, fortunately not on a daily basis, but therefore also 'unexpectedly'.

Figure 1 shows a core box from a project that has not been *completed* during ten years of trying. The massive core is from a project that may not be *started* for at least ten years. The first should already have passing high-speed trains, the other may have high-level nuclear waste some time in the future. They are both from the same country, but may have six orders of magnitude contrast in Q-value. A second pair of examples shown in Figure 2, requires a cable car for access on the one hand, and successive boat trips to fault-blocked flooded sections of tunnel on the other.

The contrasting stiffness and strength of intact rock and wet clay is easy to visualize. One may be crushed by one and drowned in the other. There are sad and multiple examples of both in the tunnelling and mining industry. They merit a widely different quality description, as for instance given by the wide-range of the Q-value.

The term Q is composed of fundamentally important parameters that were each (besides Deere's RQD), quantified by exhaustive case record analysis. The six-orders-of-magnitude range of Q is a *partial reflection* of the potentially enormous

2007

Thermal over-closure of joints and rock masses and implications for HLW repositories

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ABSTRACT:. Rough joints can be *over-closed*, and remain over-closed by a previous application of a higher normal stress. This is an exaggerated form of hysteresis. Rough joints in igneous and metamorphic rocks can *over-close* even due to temperature increase alone, due to better fit, which is something beyond hysteresis. The *rock mass* deformation moduli, thermal expansion coefficients, hydraulic apertures, and seismic velocities may each be affected. Well-controlled laboratory HTM tests, *in situ* HTM block tests, and large-scale heated rock mass tests, lasting several years at Stripa, Climax and Yucca Mountain, have produced evidence for this *extra* fully-coupled response. *Over-closed* laboratory direct shear tests give elevated strength envelopes in the case of tension fractures and joint replicas. Heating alone also increases the shear strength of natural joints. The coupled *thermal-OC* effect in HTM numerical modelling will require, as a minimum, thermal expansion coefficients that *include* rather than *exclude* relevant joint sets, if these have marked roughness and if they originated at elevated temperature. Subsequently elevated deformation moduli that attract higher stress must be expected.

1 INTRODUCTION

Hydro-thermo-mechanical HTM modelling of high level nuclear waste disposal scenarios has been actively sought in the last 30 years. In simplified form, the HTM (and chemical) effects of excavation, heating and cooling (with eventual seismic loading from major earthquakes in the very long term), have each to be simulated. The effects of heating and cooling on rock joints likely to exist in the 'geological containment' will be the focus of this paper.

A phenomenon revealed almost 40 years ago, that has proved to have relevance for both HTM field experiments and HTM modelling, concerns *over-closure* of joints. Under ambient conditions we may refer simply to hysteresis effects, but when heat is added, *thermal over-closure* appears to accentuate closure effects in the rock mass. This sounds 'positive' for waste isolation: in fact it may be adverse, due to the subsequent cooling that requires shrinkage in a rock mass that may have *over-closed* rough joint sets that remain closed despite cooling.

Difficulties in obtaining excavation-induced failure of artificial rock slope models, each consisting of 40,000 blocks, reported in Barton, 1971 and 1972, has proved to have an unexpected link to the above concerns. Steep, gravity- and horizontally-stressed slopes with adversely-dipping sets of tension fractures 'would not fail', in relation to slope stability calculations based on strengths obtained from conventional 1:1 direct shear tests.

When loading to 4 or 8 times *higher* normal stress, *prior to unloading and shearing*, successively steeper shear strength envelopes were obtained, as illustrated in Figure 1. The excessively stable slopes (Figure 2) were actually caused by *over-closure* of the rough tension fractures. As observed sometimes in real slope failures, there was evidence in slope-failure debris, of 'over-closed' masses of blocks, which might be interpreted as 'discontinuous jointing' or evidence of 'cohesive strength' in field observations.

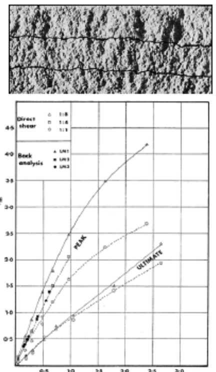


Figure 1. Over-closure (OC) ratios of 8:1, 4:1 and 1:1 (conventional) prior to direct shear testing of rough tension fractures. Barton, 1972. An example of the model tension fractures, and their surface roughness is also shown. 'Back-analysis' refers to the model slope failures.

2007

RMR and Q - Setting records

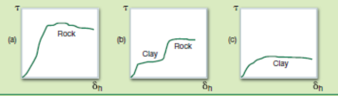
After 35 years of use throughout the tunnelling world, the RMR and Q classifications have proved themselves on numerous projects. They still face misconceptions however, as reflected in recent articles in T&T International. Here, Nick Barton, of Nick Barton & Associates, Norway, and ZT Bieniawski, of Bieniawski Design Enterprises, USA, clear common misunderstandings and provide the "ten commandments" for proper use of these rock mass classification systems

The RMR and Q rock mass classifications were independent developments in 1973 and 1974, whose common purpose was to quantify rock mass characteristics previously based on qualitative geological descriptions. They were originally developed for assisting with the rock engineering design of tunnels.

The value of thorough geological exploration was never disputed, indeed it was always emphasised. In addition, it was repeatedly stated that these classification systems were not "cookbooks" but had to be used for the purpose for which they were developed, as part of the engineering design process. This is clearly an iterative procedure in the case of underground works, where detailed knowledge of the ground develops from day to day.

At the time of the development of RMR and Q, geologists often worked in separate teams from those of engineers, leading to potential misunderstanding of what was required by whom, for engineering purposes. In fact, the advent of our rock mass classifications seems to have stimulated an opportunity to combine the efforts of engineers and geologists to act as one team, with clear statements of basic tunnel engineering needs and some carefully selected and quantitative geological data requirements. Needless to say, neither the engineering nor the geological parameters involved when using the two systems are exhaustive specifications in either the RMR or Q systems.

In essence, geologists should not be 'afraid' of quantified RMR and Q parameter ratings. The need for such quantification is perhaps appreciated more by certified engineering geologists who, although in short supply, do set an example to the traditional geologists, who are more the 'free spirits' of these basic earth science disciplines. Alas, the geological profession, even today, is not always in agreement on the scope of competence needed by engineering geologists.



(a) Rock wall contact	Jr	Ja = 0.75	1.0	2	3	4
(thin coatings)		tan⁻¹ (Jr/Ja)°				
A. Discontinuous joints	4	79°	76°	63°	53°	45°
B. Rough, undulating	3	76°	72°	56°	45°	37°
C. Smooth, undulating	2	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°

(b) Rock wall contact when sheared	Jr	Ja = 4	6	8	12
(thin fillings)		tan⁻¹ (Jr/Ja)°			
A. Discontinuous joints	4	45°	34°	27°	18°
B. Rough, undulating	3	37°	27°	21°	14°
C. Smooth, undulating	2	27°	18°	14°	9.5°
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°
E. Rough, planar	1.5	21°	14°	11°	7.1°
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°

(c) No rock wall contact when sheared	Jr	Ja = 5	6	8	12	13	16	20
(thick fillings)		tan⁻¹ (Jr/Ja)°						
Nominal roughness of discontinuity rock walls	1.0	11.3°	9.5°	7.1°	4.8°	4.4°	3.6°	2.9°

Above: Fig 1 - The parameters Jr and Ja are clearly related with 'rock behaviour', despite Goodman's reference to Riedmüller's doubts on this score. Other parameters used in RMR and Q are also clearly related to rock behaviour

The scope of RMR and Q systems

The RMR and Q systems are particularly well suited in the planning stage of a tunnelling project when a preliminary assessment of the most likely tunnel support requirements is required, based on core logging, field mapping, and refraction seismics. In the case of plans for cavern construction, even details of location may be influenced by the results. During construction, application becomes even more essential, as the appropriate support classes are selected on a day-by-day basis.

It is obviously incorrect to state they play no role during construction or final design, as those involved more frequently in tunnelling consultancy will surely acknowledge. The reasons for this are as follows:

1) RMR and Q originated, and have been specifically updated, for estimating tunnel support. Later[1,2,3,4] they were extended for assessing rock mass properties, such as the modulus of deformation, interpreting seismic velocities, and for assisting with the interpretation of monitoring during

2008 (Drafting error, Fig, 2. 1.5m bolt at 2m span: Move given bolt lengths up one interval).

TIME-LAPSE INTERPRETATION USING FRACTURE SHEAR PHENOMENA

Nick Barton *Nick Barton & Associates, Oslo, Norway*

ABSTRACT
If production causes down-dip shearing on conjugate dipping fractures, as deduced a long time ago from rock mechanics modelling and from new slickensiding in the case of Ekofisk, then temporal changes to the pseudo-static compliances, and changes to hydraulic apertures due to slight dilation, must each be expected. There may also be changes in the stretching over-burden in the case of the subsidence, which will cause temporal changes to the strength of shear-wave anisotropy and attenuation, due to intra-bed joint opening and shear. It is insufficient in each of the above cases to refer to 'stress or strain' effects, as if a continuum alone was reacting to the multiple effects of production in a multi-km³ fractured reservoir, with a multi-km³ overburden.

Compliance and fluid effects
The problem of vertical shear wave propagation in jointed or fractured media with *off-vertical dips* was addressed by Sayers, 2002, using the example of two conjugate sets with oppositely oriented dip angles. The shear wave components qS_1 and qS_2 depend here on both the shear *and* normal compliances, since the incident angles are no longer parallel to the fracture planes.

As normal compliance is *reduced* (i.e. stiffened) by fluids of non-zero bulk modulus, a moving gas to oil front should also be distinguishable by respectively greater followed by less shear wave anisotropy, as the stiffening effect of the oil makes the fracture normal stiffness less contrasted to the back-ground medium (Van der Kolk et al., 2001). For dipping joints or fractures, there proves to be a significant *decrease* in shear wave anisotropy if the fluid has a higher bulk modulus, making the normal *stiffness* of the fractures greater when oil replaces gas.

Discontinuous behaviour of fractured chalk
Carbonate or chalk reservoirs of high porosity, and therefore rather low strength, with steeply dipping, as opposed to flat-lying jointing or fracturing, can apparently continue to be successful oil producers despite strong compaction, because of a remarkable joint shearing mechanism. Down-dip shearing can occur despite the one-dimensional (vertical) strain boundary conditions that apply during the production-induced compaction of a large tabular reservoir. Matrix shrinkage under an increasingly large increase of effective stress, actually 'makes space' for down-dip shearing of the fractures. This helps to maintain joint aperture due to shear-induced dilation.

When intense investigations were occurring in the mid-eighties, to try to understand the scope and mechanisms behind the unexpected seabed subsidence above the 3 km deep Ekofisk reservoir, efforts were made to investigate the *discontinuous* natures of reservoir compaction *and* subsidence, which are normally ignored because of modelling size-limitations. This was done at two specific scales, both of which now prove to have potential influence on current 4D time-lapse interpretation options, both for the reservoir and for the over-burden.

Phillips Petroleum geologist's core logging interpretation (H. Farrell, pers. comm.. 1985), of the *conjugate* steeply-dipping jointing or fracturing in the porous, highly productive sections of the reservoir, indicated about 10 to 12 dominant (perhaps > 1 m long) set #1 joints crossing a '1 m window', with oppositely dipping set #2 joints showing about 4 to 6 shorter joints (perhaps 30-50 cm long) in this same 'volume'. These are shown in Figure 1a in an idealized form with constant dip within each set. This of course is a simplification of reality.

The assumed jointing in Figure 1 can be shown to represent an accumulated (two-set) *crack (or fracture) density* ($\varepsilon = N.\ r^3/V$) as high as 1.4, which is much higher than the more limited range often referred to in geophysics literature (Barton, 2006). When fracture densities are as high as 1 to 2, as in such well-jointed, domal chalk reservoirs, dimming of the amplitudes of the slow shear-wave, due to

2007

A UNIQUE METRO ACCIDENT IN BRAZIL CAUSED BY MULTIPLE FACTORS

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INTRODUCTION

On Friday 12th January 2007, a dramatic metro *construction* accident occurred in São Paulo, Brazil. Nearly the whole of one of the station caverns of 40 m length suddenly collapsed, immediately followed by collapse of nearly half of the adjacent 40 m diameter and 35 m deep station shaft. Seven people lost their lives in the collapse.

These station and shaft constructions are close to the Pinheiros River, in the SW sector of the city, and are part of the new Line 4 (Yellow Line) of the presently expanding São Paulo Metro. The consortium CVA, Consorcio Via Amarela, composed of most of the major contractors in Brazil, were awarded the detailed design and construction of Line 4 in 2004.

Figure 1. The Pinheiros station cavern and shaft collapse of 12th January 2007. The white arrow indicates the axis of the station cavern, and the fallen white car indicates the rear discontinuity.

The accident occurred so rapidly that there was no time for warning to be given. It is probable that suction, caused by the rapid fall of a huge undetected ridge of jointed, foliated and often deeply weathered rock weighing some 15,000 to 20,000 tons, causing an air blast in the running tunnel, actually sucked the seven Rua Capri victims to a lower level in the debris than they would have fallen if materials had been more uniform. Five of the victims were in a small bus, others were pedestrians in Rua Capri, seen to the right-side of Figure 1.

2008

IMPORTANT ASPECTS OF PETROLEUM RESERVOIR AND CRUSTAL PERMEABILITY AND STRENGTH AT SEVERAL KILOMETRES DEPTH

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Abstract

A classic assumption in geophysics is that shear wave polarization and splitting occurs due to stress-aligned structure, previously considered to be stress-aligned microcracks. This structure is now more often considered to be a desirable 'open' set of sub-vertical conducting fractures that are also assumed to be parallel or sub-parallel to the maximum horizontal stress. Geomechanics modeling unfortunately demonstrates that unless fractures are rather rough and wall strength rather high, or that there is over-pressure, there are likely to be only very small hydraulic apertures at several kilometers depth. Deep-well measurements demonstrate that fractures that are under differential shear stress are more likely to be water conducting, and those that are principally under normal stress are less likely to be water conducting. In this paper, alternative interpretations of shear-wave polarization directions are examined, including the contribution of two, maybe unequal joint sets, intersected by the major stress, having different stiffnesses and dynamic compliances, and possibly with pre-peak non-linear shear strength and dilation contributions to their enhanced permeability. Shearing induced by reservoir production and compaction is also considered, both as a source of permeability maintenance, and as a potential source of temporal rotation of seismic anisotropy, as recently recorded in 4D seismic at the Ekofisk and Valhall reservoirs in the North Sea. The shear stresses, or the mobilized frictional strength assumed to be acting on sheared joints or minor faults in deep well analyses is very high, such as μ of 0.9, and the possibility of an error, due to application of stress transformation equations that do not include dilation, is therefore addressed.

Keywords: joints, shear strength, shear waves, anisotropy, permeability, reservoirs, deep wells

1. Introduction

1.1 *Shear waves for detecting jointing*

The use of polarized shear waves, for indicating the presence of aligned and perhaps fluid-conducting structure at depths in petroleum reservoirs, has been a topic of interest for at least 20 years. The classic assumption has been that the aligned structure that causes frequency-dependent shear-wave anisotropy, is usually parallel or sub-parallel to the major stress.

1.2 *Unequal joint sets may be present*

In fact it has been shown in an extensive review of the literature [1], that deviation between the assumed major stress and apparent structure, or deviation between the axes of anisotropy and the assumed major stress, may each occur, each being more likely when one is no longer close to the surface. The reasons might be that more than one set of unequal fractures could be contributing to the anisotropy, and that if these are *bisected* by the major stress, permeability at depth could also be more easily explained, due to the shear stress actually causing slight, but desirable, dilation.

The need for this alternative explanation is due to the difficulty of explaining 'open' fractures at depth: joints or fractures are likely to be held 'closed' by a minimum effective normal stress of ten's of MPa. However, mineral-bridging, or joints with rough surfaces in hard rock are possible alternative explanations of 'open' features at depth.

1.3 *Geomechanical modelling and testing*

Parametric studies of typical reservoir rocks and

2008

Invited Editorial: J. Rock Mech. Tunn. Tech. Dec 2008/Jan2009

By Dr. Nick Barton

Extending the Boundaries in the Himalayas

During two recent trips to India, the first to the September 2008 ITA Congress in Agra, the second to hold a three-days short course in New Dehli in December, the undersigned was impressed by two things in particular (leaving aside, of course, the incomparable Taj Mahal, admired equally on any occasion).

While more Indian engineering geologists may now be utilizing the Q-system in large hydroelectric projects in India and surrounding countries, the limitations in certain extreme conditions are not to be ignored. Taking one 'boundary' first, the writer was informed that beyond Q=5, the Q-system had' little application' in the northern foothill regions of the continent.

While this point is taken as a *possible generality*, it was questioned by others with more experience, and my personal experiences at Dul Hasti suggest that Q as high as 500, giving only 1 m/hr TBM penetration rate, and only 5 to 20 m drill-bit life in the same massive quartzites, does require flexibility of the classification method, which soon may show Q < 0.01, or worse, in an adjacent shear zone, or in the alternating beds of over-excavating, low stand-up-time, sheared and talcy phyllites. These layers, following disturbance, may resemble dry bars of soap – and are more difficult to climb than a sandhill – following their collapse to the floor of a big tunnel.

Dramatic descriptions of some of the conditions at the Tala Project in Bhutan, and an ITA congress author suggesting that '*Q<0.001'* is needed for exceptional inrush/burst and tunnel drowning, is certainly correct. The Q-system, without additional engineering advice, can also not tackle the recent case of an Indian TBM machine-and-tunnel burial, simply from the remarkably high pressure (100 bars?) 'production' of water, mud, silt and sand from a single pilot hole.

A personal experience from a difficult sub-sea TBM project in Hong Kong, where the undersigned was consultant to Skanska, is a useful illustration of the message to be focussed on in this guest editorial. On the second visit to this 3.3 m diameter sewage tunnel, to verify the continuing difficulties, the contractor had commissioned a 720 m horizontal drill-core, drilled back-wards towards the TBM, from the tunnel-completion-shaft on Stonecutter Island (now part of one of the world's largest container ports). The TBM had yet to approach and penetrate a regional fault zone in the last 900 m, a wide zone which was mostly missed during sub-sea seismic, due to 'impossble' ship-traffic conditions for the seismic exploration vessel.

2008

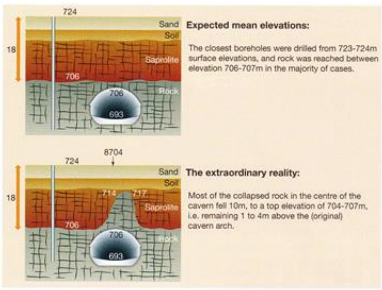
A unique metro accident in Brazil

Right: The Pinheiros station cavern and shaft collapse of 12th January 2007

The sudden collapse of Pinheiros Station and station shaft during construction of the São Paulo Metro Line 4 shocked the industry. Consultant, Nick Barton describes the events

On Friday 12th January 2007, a dramatic metro construction accident occurred in São Paulo, Brazil. Nearly the whole of one of the station caverns of 40m length suddenly collapsed, immediately followed by collapse of nearly half of the adjacent 40m diameter and 35m deep station shaft. Seven people lost their lives in the collapse.

These station and shaft constructions are close to the Pinheiros River, in the SW sector of the city, and are part of the new Line 4 (Yellow Line) of the presently expanding São Paulo Metro. The consortium CVA, Consorcio Via Amarela, composed of most of the major contractors in Brazil, were awarded the detailed design and construction of Line 4 in 2004.



Above: Fig 1 - Top) Sketch of the anticipated top-of-rock elevations based on the five nearest boreholes, including one hole near the centre of the cavern. Bottom) Sketch of the extraordinary reality, in over-simplified form

The accident occurred so rapidly that there was no time for warning to be given. It is probable that suction, caused by the rapid fall of a huge undetected ridge of jointed, foliated and often deeply weathered rock weighing some 15,000 to 20,000 tons, causing an air blast in the running tunnel, actually sucked the seven Rua Capri victims to a lower level in the debris than they would have fallen if materials had been more uniform. Five of the victims were in a small bus, others were pedestrians in Rua Capri.

Boreholes for site investigation

Prior to final design and construction of the 18m span station cavern, numerous boreholes had been drilled through the soil, saprolite and weathered Pre-Cambrian gneiss. There were eleven boreholes drilled around the shaft and eastern station cavern. The four boreholes located close to the sides of the cavern, and one almost in the centre of the cavern, had indicated some zones of deeply weathered rock, especially in the biotite gneiss. Foliation was mostly steeply dipping to vertical.

The arch of the Pinheiros station was at a mean elevation of 703m. Borehole 8704 drilled near the centre of the cavern, had correctly indicated a (local) top-of-rock elevation of 706m. This was exactly the same as the mean rock elevation found in the four other closest holes.

2008

Combining borehole characterization and various seismic measurements in tunnelling

Combinando la caracterización de pozos de sondeo y distintas mediciones sísmicas en la construcción de túneles

Nick Barton

Nick Barton & Associates, Oslo, Norway

Abstract

Cross-disciplinary examples of rock mass classification and characterization are selected from a recent major review by the author, from various civil engineering construction projects, with emphasis on tunnelling, and making much use of cross-hole seismic measurements and refraction seismic. The links between velocity and rock quality, deformability, permeability and velocity are developed and demonstrated. The combined use of seismic and Q-logging, allows classification and characterization to be distinguished, the former with an excavation EDZ, the latter pre-construction.

Resumen

Ejemplos multi-disciplinarios de clasificación y caracterización de masa rocosa son seleccionados de un importante estudio reciente realizado por el autor, y de varios proyectos de construcción de ingeniería civil, haciendo énfasis en la construcción de túneles, y con un amplio uso de mediciones sísmicas transversales a los pozos y estudios de refracción sísmica. Este documento desarrolla y demuestra las relaciones existentes entre velocidad y calidad de roca, grado de deformación, permeabilidad y velocidad. El uso combinado de perfilaje sísmico y de Q permite que la clasificación y caracterización tengan un sello distintivo, el primero con excavación EDZ, y el segundo previo a la construcción.

INTRODUCTION

Rock mass classification has come to be associated with the need to select a rock mass class on the basis of prior classification or rating of various rock mass parameters. Most frequently, classification is used in association with tunnel support, and also as a basis for payment. It may also reflect a need for pre-treatment. Implicit here is the existence of the tunnel, and the effect this may have on the expected rock mass response, in particular that within the EDZ. All of the above may also apply to rock slopes, but here post-treatment is more likely.

Rock mass characterization reflects a broader mission to describe the character of a rock mass where a future project is likely to be realized. Besides rock quality description with one of the standard measures such as RQD, or RMR, or Q, or GSI, or several of these, it should also include site characterization fundamentals such as rock stress, water pressure, permeability and seismic velocities. Ideally each of the above should be measured as a function of depth and azimuth and of course reflect lateral variation and variation in specific domains. Various simple index parameters of the matrix and joint sets can also considered characterization, like UCS and the JRC-JCS roughness-strength character that can be estimated during core logging.

Cross-disciplinary characterization involving Q, velocity, permeability, and deformability will be used to illustrate the frequent differences between classification and characterization.

THE EXCAVATION DISTURBED ZONE

The cross-hole seismic description of the site for a future ship lock shown in Figure 1 (diagram a) can be considered one form of characterization of the site. The RQD, RMR and Q-values of the first two cores would be essential supplementary data. Ideally core or subsequent borehole logging should be oriented due to kinematic stability assessment needs. Subsequent cross-hole seismic between supplementary holes shows the

2008

Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design

N.R.Barton *Nick Barton & Associates, Oslo, Norway*

Abstract

The peak shear strength of rock joints obtained from direct shear tests, and the peak shear strength of rockfill, as interpreted from large-scale triaxial tests, have common non-linear strength envelopes. An extremely low stress index test for rock joints, the tilt test, with an apparent normal stress as low as 0.001 MPa when sliding occurs, can also be performed to characterize rockfill. However for rockfill or rock dumps, larger samples with relevant particle sizes are desirable. Some full-scale tests at a dam site in Italy, using a 2x2x5 m tilt-shear test, were able to sample the as-compacted-as-built rockfill, with no need for using parallel (model) grading curves with reduced-sized particles. Interfaces between the rockfill or rock dump and eventual rock foundations, can be handled with similar shear strength estimation methods. In each case, a low-stress index test result is extrapolated to full scale and to engineering stress level by related non-linear strength laws. It is possible to estimate each through inexpensive characterization. The non-linear, stress-dependent friction angles suggest that large rock dumps with constant slope angle will have strongly reducing factors of safety from top to bottom and from outside to inside.

1 Introduction

The real contact stress levels are believed to be close to compressive failure where rock joint asperities and rockfill stones are in contact (e.g. Figure 1 for the case of rock joints). It is perhaps therefore that it is possible to use a common form of constitutive equation for extrapolating the strength measured at very low (index test) normal stress levels, to stress levels of engineering interest, as inside a large rockfill dam, inside a rock dump or under a rock slope formed of jointed rock.

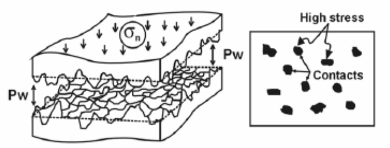


Figure 1 When peak shear strength is approached (joints and rockfill), the actual rock-to-rock contact stress levels are extremely high, due to small contact areas

It is believed that the real ratios of σ_{cn} /JCS (contact normal stress/joint wall compressive strength, in the case of rock joints) and σ_{cn} /S (contact normal stress/particle strength,in the case of rockfill) are equal to the ratio A_0 / A_1 representing the ratio of true contact area/assumed contact area. The terms JCS and S represent the joint compressive strength and the particle strength, respectively. In other words, contact area is a rock strength or particle strength regulated phenomenon at peak strength.

Tilt tests are performed on a regular basis to characterize the roughness of rock joints. A schematic example of tilt testing for rock joints is shown in Figure 2, while a suggested method for testing rockfill at full scale (without needing parallel grading curves) is shown in Figure 3, from Barton and Kjærnsli, (1981).

2008

The main causes of the Pinheiros cavern collapse

Nick Barton
Nick Barton & Associates, Oslo, Norway

Abstract
An extremely large collapse occurred at a metro cavern and station shaft, along the new Line 4 in São Paulo in early 2007. Despite extensive investigation with eleven boreholes close to and even in the centre of the cavern, a high-standing central ridge of less weathered gneiss, with one misleading low point, was missed by all drill holes. Low rock cover was assumed, but the reality was arching compromised by an adverse, wedge-shaped, 10 m high and 15-20,000 tons undiscovered ridge of rock that grossly over-loaded the structural arch and its wide footings.

INTRODUCTION

On Friday 12th January 2007, a dramatic metro construction accident occurred in São Paulo, Brazil. Nearly the whole of the station caverns of 40 m length suddenly collapsed, immediately followed by collapse of nearly half of the adjacent 40 m diameter and 35 m deep station shaft. Seven people lost their lives in the collapse.

These station and shaft constructions are close to the Pinheiros River, in the SW sector of the city, and are part of the new Line 4 (Yellow Line) of the presently expanding São Paulo Metro. The Consortium CVA, Consorcio Via Amarela, composed of most of the major contractors in Brazil,

Figure 1. The Pinheiros station cavern and shaft collapse of 12th January 2007.

were awarded the detailed design and construction of Line 4 in 2004. The accident occurred so rapidly that there was no time for warning to be given.

It is probable that suction, caused by the rapid fall of a huge undetected ridge of jointed, foliated and often deeply weathered rock weighing some 15,000 to 20,000 tons, causing an air blast in the running tunnel, actually sucked the seven Rua Capri victims to a lower level in the debris than they would have fallen if materials had been more uniform. Five of the victims were in a small bus; others were pedestrians in Rua Capri, seen to the right-side of Figure 1.

BOREHOLES FOR SITE INVESTIGATION

Prior to final design and construction of the 18 m span station cavern, numerous boreholes had been drilled through the soil, saprolite and weathered Pre-Cambrian gneiss. There were eleven boreholes drilled around the shaft and eastern station cavern. The four boreholes located close to the sides of the cavern, and one almost in the centre of the cavern, had indicated some zones of deeply weathered rock, especially in the biotite gneiss. Foliation was mostly steeply dipping to vertical.

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2008

TBM tunnel construction in difficult ground
Construcción de túnel con TBM en un terreno difícil

Nick Barton
Nick Barton & Associates, Oslo Norway

Abstract
Experiences with tunnelling problems are addressed, with particular reference to fault zone and sheared zone experiences in TBM tunnels in Italy, Greece, Kashmir, Hong Kong and Taiwan, together with fault zone cases in the Q_{TBM} data base. TBM achieve remarkable advance rates when conditions are favourable, out-performing drill-and-blast tunnelling by a wide margin, but they suffer great problems when conditions are very poor. The theo-empirical reasons for this are illustrated, and Q_{TBM} prognosis examples are given.

Resumen
Las experiencias con problemas en la construcción de túneles se abordan haciendo referencia especial a los casos con zonas de fallas y cizalla en túneles con TBM en Italia, Grecia, Cachemira, Hong Kong y Taiwán, junto con casos de zonas de fallas existentes en la base de datos Q_{TBM}. El TBM alcanza una velocidad de avance notable cuando las condiciones son favorables, sobrepasando por lejos el rendimiento de construcción de túneles con perforación y voladura, sin embargo, tienen grandes problemas cuando las condiciones son muy precarias. Este documento ilustra las razones teórico-empíricas de esto, junto con entregar ejemplos de la prognosis de Q_{TBM}.

INTRODUCTION

TBM tunnelling and drill-and-blast tunnelling show some initially confusing reversals of logic, with best quality rock giving best advance rates in the case of drill-and-blast, since support needs may be minimal, whereas TBM may be penetrating at their slowest rates in similar massive conditions, due to rock-breakage difficulties, cutter wear, and the need for too-frequent cutter change, the latter affecting the advance rate AR. This 'reversed' trend for TBM in best quality, highest velocity (Vp) rock is demonstrated by the PR-Vp data from some Japanese tunnels, reproduced in Figure 1, from Mitani et al., 1987.

At the low velocity, high PR end of this data set, there will not be a need for frequent cutter change, but conversely there will be delays for much heavier support. If velocities reach as high as about 5.5-6.5 km/s (i.e. Q > 100, and high UCS) in exceptionally massive rock, this is also 'difficult ground' for TBM, and in exceptional cases PR may dip below 0.5 m/hr, if under-powered. Older cases of PR = 0.1 and 0.2 m/hr are known, but rare (Barton, 2000).

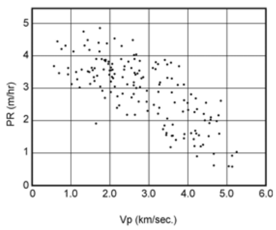

Fig. 1 Declining TBM penetration rate with elevated seismic velocity, due to lack of jointing. The actual advance rate will be a function of opposite effects in the best rock, namely need for frequent cutter change, but little delay for support. Mitani et al., 1987.

2008

INTEGRATING Q-LOGGING WITH SEISMIC REFRACTION, PERMEABILITY, PRE-GROUTING, TUNNEL AND CAVERN SUPPORT NEEDS, AND NUMERICAL MODELLING OF PERFORMANCE.

Nick Barton[1]

Abstract: When the 'Q-system' was launched in 1974, the name referred to rock mass classification, with focus on tunnel and cavern support selection. Since that time 'a system' has indeed been developed. The Q-system now integrates investigation geophysics, rock mass characterization, input for numerical modelling, empirical design of support, and excavation performance assessment. The Q-value has proved easy to correlate with required support capacity, relative cost and time for tunnel construction, seismic P-wave velocity, deformation modulus, cavern deformation, and in modified form with permeability. Recent research has also shown encouraging links between Q, the depth dependant deformation modulus, and the *seismic quality* Q_{seis}, which is the inverse of attenuation. There are also indications that Q has captured important elements of the cohesive and frictional strength of rock masses. The above sensitivities are most likely because Q is composed of fundamentally important parameters that were quantified by exhaustive case record analysis. The six-orders-of-magnitude range is a reflection of the potentially enormous variability of geology and structural geology. Some of the empirical relationships are illustrated with a summary of Gjøvik Olympic cavern investigations, and of the *discontinuum* modelling of performance. The paper concludes with a critical assessment of the potential shortcomings of *continuum* modelling of highly stressed excavations in intact rock, and of shallow excavations in anisotropically jointed rock.

INTRODUCTION

This lecture will be an illustrated journey through some of the useful linkages and concepts that have been absorbed into the 'Q-system' during the last ten years or so. From the outset the focus will be on sound, simple empiricism, that works because it reflects practice, that can be used because it can be remembered, and that does not require black-box software solutions. Some of the empiricism will be illustrated by reference to investigations and to empirical and numerical modelling performed at the Gjøvik Olympic cavern in Norway.

Nature varies a lot and therefore Q does too

It is appropriate to start by illustrating contrasting rock mass qualities. Figure 1 shows a core box from a project that has not been *completed* during ten years of trying. The second project may not be *started* for at least ten years. The first should already have passing high-speed trains, the other high-level nuclear waste some time in the future. They are both from the same country and may have six orders of magnitude contrast in Q-value. A second pair of examples shown in Figure 2, requires a cable car for access on the one hand, and successive boat trips to fault-blocked flooded sections of tunnel on the other.

The contrasting stiffness and strength of intact rock and wet clay is easy to visualize. One may be crushed by one and drowned in the other. There are sad and multiple examples of both in the tunnelling industry. They merit a widely different quality description, as given by the Q-value.

2008

FJELLSPRENGNINGSTEKNIKK
BERGMEKANIKK/ GEOTEKNIKK 2008

TRAGIC COLLAPSE OF A STATION CAVERN DURING CONSTRUCTION OF THE SÃO PAULO METRO: UNEXPECTED AND UNPREDICTABLE GROUND DESPITE ELEVEN BOREHOLES

Tragisk kollaps av stasjonshall under bygging av Sao Paulo metro: Uforutsett og uforutsigbar grunnforhold tross elleve borhull

Dr. N. R. Barton, Nick Barton & Associates

SUMMARY

In January 2007, a dramatic metro construction accident occurred in São Paulo. Nearly the whole of one of the station caverns of 40 m length and 19 m span suddenly collapsed. Despite extensive drilling around and even within the cavern centre, a misleading top-of-rock elevation was indicated, giving an assumed 3 m of rock cover above the arch of the cavern, beneath about 18 m of saprolite, soil and sand. Heavy lattice girders at 0.85 m centres and steel-fibre reinforced shotcrete of at least 35 cm thickness were used as primary support. Subsequent excavation through all the collapsed rock and soil during 15 months of investigations revealed a previously undiscovered, 10 m high ridge of rock with adversely oriented steep sides, caused by differential weathering of vertically foliated gneiss. A secondary planar joint set, a major bounding discontinuity, and probable elevated pore pressure from a cracked storm drain constituted an unpredictable set of adverse conditions at an adverse location beneath a road, causing the death of seven people when sudden collapse occurred.

SAMMENDRAG

I januar 2007 skjedde det et dramatisk ulykke under bygging av São Paulo metro. Nesten hele volum av en stasjonshall av lengde 40 m og spennvidde 19 m plutselig kollapset. Tross utstrakt borhullsundersøkelse rund omkring og i midten av fjellhallen, var en misvisende fjellkvote indikert, med antatt 3 m bergoverdekning over hengen, under 18 m med dypforvitret fjell (saprolitt), jord og sand. Tunge stålbuer eller 'lattice girders' med 0.85 m senteravstand, sammen med minimum 35 cm stålfiberarmert sprøytebetong var tatt i bruk som primærsikring. Senere utgraving gjennom alt jord og fjell som falt ned til fjellrommets bunn, som tok 15 måneder å gjennomføre, viste rester av en uoppdaget 10 m høy fjellrygg med nesten vertikale og ugunstige orienterte sider, forårsaket av differentiert forvitring av gneis med nesten vertikal foliasjon. Et sekundær, plan sprekkesett, pluss en stor diskontinuitet som begrenset kollapset, og i tillegg en antatt sprukket og lekkende stormdren, konstituert et uforutsigbar sett med ugunstige forhold samt ugunstig lokalisering rett under en traffikert vei, som forårsaket tap av syv menneskerliv, når det plutselig var kollaps.

1. INTRODUCTION

On the afternoon of Friday 12[th] January 2007, a dramatic accident occurred during the

2008

Metro construction at the most unfavourable depth caused a major metro station collapse in Brazil due to a unique sub-surface structure

N.R. Barton
Nick Barton & Associates, Oslo, Norway

ABSTRACT: A metro project that was constructed in the most difficult elevation possible, with constantly changing rock-to-saprolite-to-soil-to-rock conditions, due to São Paulo metro operator requirements, suffered the predicted consequences of severe overbreak and slow progress. On two occasions there was break-through to surface. This paper describes one of these events that involved a set of adverse circumstances that tragically converged in time and location. On January 12[th] 2007, the following dramatic accident occurred. Nearly the whole of one of the station caverns of 40 m length and 19 m span suddenly collapsed. Despite extensive drilling around and even within the cavern centre, a misleading top-of-rock elevation was indicated, giving an assumed average 3 m of rock cover above the arch of the cavern, beneath about 18 m of saprolite, soil and sand. Heavy lattice girders at 0.85 m centres and steel-fibre reinforced shotcrete of minimum 35 cm thickness were used as primary support. Subsequent excavation through all the collapsed rock and soil during 15 months of investigations revealed a previously undiscovered, 10-11 m high ridge of rock with adversely oriented steep sides, caused by differential weathering of the foliated gneiss and an amphibolite band. A secondary planar joint set, a major bounding discontinuity, and probable elevated pore pressure from a cracked storm drain, constituted an unpredictable set of adverse conditions at an adverse location beneath a road, causing the death of seven people when sudden collapse occurred. Lessons learned the hard way confirmed the prior opinions of several prominent consultants who had called for either shallower, or deeper construction, either options in order to avoid frequently changing mixed-face conditions, which create a range of unnecessary difficulties.

1. INTRODUCTION

There are several possible choices for expansion of metro lines involving the addition of new stations in major cities. The most difficult from the point of existing infrastructure and buildings is of course cut-and-cover. In less developed parts of cities under expansion, this is nevertheless the most viable option, and there are many examples from around the world. We can then consider two remaining basic options: shallow tunnels with stations developed from large diameter shafts or deep tie-back excavations, and the third option of deeper construction, probably entirely in rock, with all major developments from the underground including the station caverns. In this third option there remains the need for an inclined escalator shaft, or in a few cases vertical lift shafts. These of course have to tackle soil, saprolite and rock transitions, but they are of limited dimensions, making for a faster and cheaper project.

From a tunneling viewpoint, the second option is by far the most complicated, as deep but differential weathering may mean frequent mixed-face tunnelling and cavern construction. In the present expansion of the São Paulo Line 4, there is an example of a station with one end entirely in rock, and the other entirely in soil. Photographs from construction of this (Butanta) station are reproduced in Figure 1, to emphasise adverse conditions even in the end in rock. The main topic of this paper is however what happened at the next station. On the afternoon of Friday 12[th] January 2007, a dramatic accident occurred at the next station (Pinheiros) along Line 4 of the São Paulo metro, about 1 km away on the other side of the Pinheiros River. Nearly the whole of one of the station caverns of 40 m length suddenly collapsed, immediately followed by collapse of nearly half of the adjacent 40 m diameter and 35 m deep station shaft. The reasons are clear after the event.

2009

Letters

The Pinheiros disagreement

Left: The Pinheiros station collapses in January 2007, killing seven

Dear Sir
I write to *T&TI* on behalf of the CVA consortium, which is composed of the five principal Brazilian contractors and two consulting companies. CVA were recently blamed by the Brazilian Technological Research Institute (IPT) team working for the prosecuting authorities, for causing the tragic metro station collapse in São Paulo, which took the lives of seven people in early January, 2007. The authorities nominated IPT and some national and international consultants to perform the 18 months investigations (see resumé in the recent Barros et al., Nov. 08, *T&TI* report "Lessons from Brazil: Pinheiros examined"). IPT's official report runs to 3,000 pages and 46 volumes.

I would first like to address *T&TI* Editorials of May, 2008 ("Let's get (geo-) physical" by Tris Thomas) and November, 2008 ("Risky business?" by Amanda Foley). *T&TI* has understandably tied significant parts of both editorials directly to the two 'opposing-parties' articles, now printed in *T&TI*. (The first, by the undersigned, was titled 'A unique metro accident in Brazil", *T&TI*, May, 08).

As an international tunnelling consultant having practiced in more than 30 countries, in a wide variety of tropical and undesirably exotic geological conditions, I would have liked to be able to share *T&TI* implied belief that 'unforeseen ground conditions' could be virtually removed from the vocabulary of tunnellers. With sufficient access, sufficient (geo)physical techniques, and sufficient time and budget, this ideal could no doubt be approached. Deeper construction of metro stations from underground, would of course be safer in tropical terrains, but clearly not more expensive, as *T&TI* suggested in the first editorial. Cities without suitable geology always have to go deeper. Only the escalators and their shafts are more expensive. Stations and metro tunnels are much cheaper, and faster driven when at greater depth, in more uniform geology. One needs to look no further than London, Prague, Moscow etc. Large-diameter shaft concepts should cease to be a valid construction option, when major construction can be from the underground. Longer escalators are a small price to pay.

As editors of any published material, it is logical that one must believe in material that is submitted for publication. The editorial of November 08, in which *T&TI* suggest, based on a synthesis of the IPT-and-consultants article, that "the ground conditions were found to be more or less exactly as predicted at the time of bidding" is understandable in the circumstances in which it was written, taking in good faith the veracity of the submitted article. In view of the fact that painstaking excavation through 16,000m^3 of collapsed soil, sand, saprolite, gneiss and mylonite, actually revealed the presence of an undiscovered ridge-of-rock, with top elevations mostly 9 to 11m higher than the evidence of the eleven nearest boreholes, one of them drilled almost in the cavern centre, this conclusion by both IPT and the *T&TI* editor has to be challenged. Besides the centre-line hole, the four closest boreholes were drilled immediately around the cavern walls. The real situation was illustrated, in a simplified diagrammatic manner, in the May, 08 *T&TI* article referred above.

The above rock-head elevation discrepancy, clearly not as predicted at the time of bidding, is miraculously passed over in the IPT article, and in their 3000 pages report, perhaps because their painstaking drawings of collapsed rock have erroneous (-5m) elevations in relation to the contrary evidence of thousands of photographs, relatively few of which they reproduced. Even after falling 9 to 10m, rock levels were still as high as presumed from borehole evidence. In other locations in the 46 volumes, their dip-and-strike records of jointing show the correct high, central ridge elevations. IPT geologists perhaps did not notice, nor do they comment this discrepancy.

Their report, reflected in the Nov 08 *T&TI* resumé, bears witness to many misunderstandings of rock mechanics principles and tunnel stability concepts, including the idea that the adjacent shaft excavation would reduce, rather than increase the tangential stress acting above the cavern arch. The IPT-and-consultants have also assumed that Ko of 1.5 is more conservative for cavern stability analyses than the designer's choice of Ko of 0.33. Their team of geologists, engineers and even professors, also draw incorrect conclusions from some of their own 3D continuum analyses concerning direction-of-excavation effects.

Rock cover was expected to be a mean 3 to 4m above the cavern arch, based on the mean of the five nearest boreholes referred to above. The reality: an inverted wedge or ridge of higher quality rock up to 11m higher, surrounded by weak, weathered material, may have weighed as much as 15,000 tons, taken together with the loading from relic-jointed saprolite. Collectively, this provided the adverse loading in the arch, sufficient both to fail some of the elephant footings at the base of the lattice-girder reinforced 40cm thick S(fr), or to cause yielding and plastic-hinge development of this load-bearing structure, where footings were more resistant. In places, the lattice-girder steel bars of 30mm and 25mm diameter were seen to have been plastically stretched and failed in tension. This matches post collapse modelling with UDEC and structural element force-moment N-M analyses.

Some points in the IPT-and-consultant *T&TI* article of November 2008 need particular comment. It is not correct that the rate of excavation accelerated during the excavation of the first bench. It is not correct, due in fact to the number of days lost in the Christmas recess. However, it is normal that bench excavation goes faster than tunnel-front excavation: that is why it is performed throughout the world. It is also normal and expected that increased deformation results. The accelerating deformation shown in the last three to five days prompted additional stabilizing measures by CVA, but even if there had been time to carry these out, failure, with the benefit of post-collapse analysis, would have been inevitable, due to the unprecedented loading.

2009

TBM prognoses in hard rock with faults using Q_{TBM} methods

N.R.Barton
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ABSTRACT

As an indirect result of several seriously delayed TBM projects, where the writer was eventually engaged as an outside consultant, a wide-reaching survey of case records was undertaken Barton (2000), in order to try to find a better basis for TBM advance rate prognosis, *that also included poor rock conditions*. It appeared that 'poor conditions' as relating to faults were usually treated as 'special cases' in the industry, with concentration mostly on solving the penetration rate (PR) and cutter life aspects of TBM prognosis. Experiences with actual tunnelling problems are therefore addressed, in order to show how good performance may be altered either in only minor ways by faults, or sometimes with dramatic consequences. A satisfactory range of penetration rates (PR) is only part of the possible success of TBM. These machines can achieve remarkable advance rates (AR) only when overall conditions are favourable, then out-performing drill-and-blast tunnelling by a wide margin. Without this pre-condition, the TBM results may be less than desired.

1 INTRODUCTION

TBM tunnelling and drill-and-blast tunnelling show some initially confusing reversals of logic, with best quality rock giving best advance rates in the case of drill-and-blast, since support needs may be minimal, whereas TBM may be penetrating at their slowest rates in similar massive conditions, due to rock-breakage difficulties, cutter wear, and the need for too-frequent cutter change, the latter affecting the advance rate AR. This 'reversed' trend for TBM in best quality, highest velocity (V_P) rock is demonstrated by the PR-V_P data from some Japanese tunnels, reproduced in Figure 1, from Mitani et al. (1987).

At the low velocity, high PR end of this data set, there will not be a need for frequent cutter change, but conversely there will be delays for much heavier support. If on the other hand velocities reach as high as about 5.5-6.5 km/s (i.e. Q > 100, and high UCS) due to exceptionally massive rock, this is also 'difficult ground' for TBM, and in exceptional cases PR may dip below 0.5 m/hr, if under-powered.

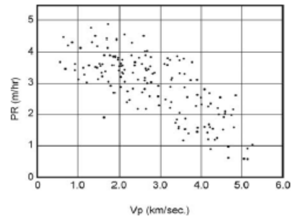

Figure 1: Declining TBM penetration rate PR with elevated seismic velocity. Mitani et al. (1987).

2009

MAIN CAUSES OF THE PINHEIROS CAVERN COLLAPSE

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Keywords: cavern, collapse, site investigations

INTRODUCTION

In January of 2007, seven people in a São Paulo street, four of them in a small bus, were suddenly sucked into falling soil and saprolite, from a street (Rua Capri) located about 20 m above a metro station cavern of 19 m span and 40 m length. This was under construction in Brazil's largest city. Despite the evidence of four surrounding and one central borehole, and six more boreholes around the adjacent station shaft, the assumed mean rock cover of just 3 m above the 20 m deep cavern arch, proved locally to be more than 10 m in error, due to a buried ridge of rock running high above the cavern arch, with one fateful low point exactly where drilled on the cavern centre-line.

SUB-SURFACE RIDGE OF ROCK WENT UNDETECTED

Due to the assumed low rock cover, heavy lattice girders, embedded in 40 cm of S(fr) were used as temporary support. The feet of the lattice girders were founded on broad 'elephant' footings. Due to the unknown loading from an adversely wedge-shaped, clay-bordered, ridge of rock and saprolite, weighing some 15,000 tons, all forms of temporary support would eventually have failed. Post-collapse, painstaking, police-supervised excavation of the entire 20 by 20 by 40 m of collapsed materials, taking some 15 months, finally revealed large remnants of the arch and wall support, crushed and folded beneath the ridge of fallen gneiss and amphibolite, plus saprolite, sand and soil. On the way down through collapsed material, the deformed remnants of the ridge were exposed all along the centre of the excavation. Even after falling 10 m to the floor of the cavern, the ridge of rock was 1 to 4 m above the original cavern arch. This fact seems to have been overlooked by the official investigators, the institute IPT. This is remarkable, but may be due to errors in elevations on their drawings of the collapsed rock. Nevertheless in dip-and-strike recordings, elsewhere in their 46 volumes report for the prosecuting authorities, IPT give the correct elevations of the fallen ridge.

Figure 1. The dramatic cavern collapse in São Paulo, during construction of the Line 4 subway. The feature 'FF' marks the limit of the collapse, in the street Rua Capri., where six people died.

2009

APPLICATION OF THE Q-SYSTEM AND Q_{TBM} PROGNOSIS TO PREDICT TBM TUNNELLING POTENTIAL FOR THE PLANNED OSLO-SKI RAIL TUNNELS

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ABSTRACT

Application of statistics-based rock mass characterization of more than 300 rock exposures totalling some 6 km is described. This was followed by utilization in TBM prognoses for the planned route of a major tunnelling project. Jernbaneverket's planned new high speed Oslo-Ski Follobanen can have up to 19 km of tunnel length, depending on the final decision on alignment. In addition to the logging of the numerous surface exposures, JBV's drillcore logging and GeoPhysix's seismic refraction measurements were also utilised, the latter both focussed on data acquisition for the crossings of assumed weakness zones. The data collection, principally using the Q-system histogram method, was the first stage of input to the Q_{TBM} prognosis modelling of potential penetration rate PR and actual advance rate AR for the two twin tunnels that are likely to be driven by TBM. Laboratory test data from SINTEF concerning strength and abrasion parameters for the mostly granitic/tonalitic gneiss, and also for the quartz- and feldspar-rich gneisses of sedimentary origin, were combined with the Q-data statistics to give estimates of potential tunnelling speeds, assuming five rock mass classes and three weakness zone classes. Characterization of the weakness zones was based on the core logging data and refraction seismic measurements. Prognoses were compared for hard rock open gripper TBM and for double-shield TBM, where robust PC element liner construction concurrent with gripper thrust gives a potentially very fast method of tunnelling. This more expensive method of tunnelling, is compensated by its general efficiencies enabling conversion of a possibly 'poor' PR into a 'good' AR, due to the high utilization, and it also gives a water-tight and fully supported tunnel. It has been used with notable success in some other high-speed rail projects through hard rock masses, despite the need for frequent cutter-changes.

SAMMENDRAG

Det er utført bergmassekarakterisering av mer enn 300 lokaliteter langs utvalgte fjellskjæringer. Bergmassekarakterisering/kartleggingen omfatter totalt en lengde av cirka 6 km. Arbeidet er utført i forbindelse med TBM prognoser for Jernbaneverkets planlagte tunnelprosjekt Oslo-Ski, Follobanen. Follobanen planlegges som en høyhastighetsbane med opptil 19 km tunnellengde avhengig av hvilket trasealternativ som velges. I tillegg til karakterisering av et stort antall fjellskjæringer er det anvendt data fra JBV's borkjernelogging og refraksjonsseismikk utført av GeoPhysix. Disse dataene er spesielt utnyttet for tilleggsinformasjon om svakhetssoner. Dette datagrunnlaget med hovedvekt på Q-histogram logging var første stadium i anvendelse av Q_{TBM} for å gi prognoser på netto inndrift PR og brutto inndrift AR. Denne Q_{TBM} prognosen gjelder for ett av tunnel alternativene hvor TBM vurderes som drivemetode. Laboratorieforsøk angående trykkfasthet og kutterslitasje foretatt av SINTEF for hovedsakelig granitisk/tonalitisk gneis og for kvarts- og feltspatrik gneis av sedimentær opprinnelse, var anvendt i kombinasjon med disse statistiske Q-data, for å gi drivhastighetsvurderinger gjennom fem antatt bergmasseklasser og tre antatt svakhetssoneklasser. Karakterisering av svakhetssonene er basert på borkjernelogging og seismikk. Disse TBM prognoser sammenligner både åpen-gripper TBM og dobbelt-skjold maskiner, de siste med simultan bygging av betongelementforing og boring

2010

Low Stress and High Stress Phenomena in Basalt Flows

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ABSTRACT: Contrasting geophysical, rock mechanics and rock engineering experience in basalts, caused by either exceedingly low or extremely high stress are described, from projects in the USA and Brazil. The first involves a nuclear waste characterization project in Hanford basalts in the USA, and the second describes, in much more detail, stress-fracturing problems in numerous large tunnels at the 1450 MW Ita hydro-electric project in SE Brazil's basalts. Particular phenomena that were noted, include linear stress-strain loading curves when columnar basalt is loaded horizontally, and a k_0 value reaching about 20-25 at Ita HEP.

1 INTRODUCTION

The beauty of columnar basalt, and the huge areal extent of basalt flows across large tracts of many countries, are perhaps the features that characterize basalt most profoundly. The Colombia River basalts in USA, and the Parana Basin basalts of S.E. Brazil, are just two of these major accumulations of 10's of thousands of km^2 of basalt. In this paper, some sophisticated characterization in the first location mentioned, in the hope of finding a nuclear waste disposal candidate, and some major rock engineering problems due to extreme horizontal stress in the second location, will form the core of this paper.

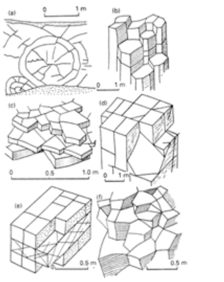


Figure 1. Basalt forms blocks of many shapes and forms.

2. STRESS-DEFORMATION CHARACTER

One of the USA's nuclear waste disposal candidates was the mid-eighties was the 900 m deep Cohasset flow of the extensive Colombia River basalts. This was found some distance away at a more convenient *shallow* depth for preliminary but extensive characterization studies, at the so-called Hanford BWIP (basalt waste isolation project).

Some interesting joint deformation effects were caused by the low horizontal stress levels at this (too) shallow location, as revealed in an *in situ* block test, and at larger scale in some cross-hole seismic measurements in a tunnel wall, showing strong EDZ effects. At each scale, behavior was affected in special ways by the anisotropic joint properties and by anisotropic stress levels, particularly the low horizontal stress. The latter could be controlled in the block test, and thermal loading logically caused joint closure: the original state. An unexpected linear stress-deformation behaviour was measured in the block test, apparently due to the contribution of both shear and normal components of joint deformation.

Some site characterization was performed by the author, along exposures of the candidate Cohasset Flow (Figure 2), which formed impressive cliffs along the distant Colombia River. Both joint properties and rock mass properties were described, in an attempt to evaluate their potential effect on disposal tunnels planned for 900 m depth at the candidate site, and possible tunnel support quantities.

Drilling and stress measurements had indicated strongly anisotropic stresses of approximately 60, 40 and 30 MPa, and some cores, presumably drilled in the midst of columnar basalt, displayed strong core

2010

Tunel 19. ročník - č. 2/2010

PODZEMNÍ STAVBY PRAHA 2010 UNDERGROUND CONSTRUCTIONS PRAGUE 2010

KEYNOTE LECTURE No. 4:

PROGNÓZOVÁNÍ VÝKONU STROJŮ TBM PŘI RAŽBÁCH VE SKALNÍCH HORNINÁCH S PÁSMY OSLABENÍ POMOCÍ TBM BEZ ŠTÍTU NEBO TBM S DVOJITÝM ŠTÍTEM

TBM PROGNOSES FOR HARD ROCK WITH WEAKNESS ZONES, USING OPEN-GRIPPER OR DOUBLE-SHIELD SOLUTIONS

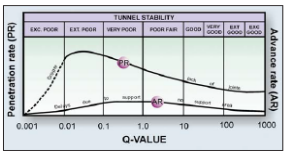
NICK BARTON

SHRNUTÍ

Dalekosáhlé posouzení ražby tunelovacími stroji (TBM) s použitím TBM bez štítu v porovnání s TBM s dvojitým štítem, kde je popis horninového prostředí méně přesný, vytváří základnu pro metodu prognózování nazývanou Q_{TBM}. V příspěvku jsou uvedeny některé poznatky z četných posuzovaných případů, následované krátkým úvodem k empirickým metodám založeným na klasifikaci. Údaje o výkonech strojů TBM vycházejí z informací z asi 145 případů, respektive asi 1000 km ražeb, jejichž celkové výkony jsou sloučeny. Od vzniku této metody se datová základna zvětšila tak, aby obsahovala data pro ražbu stroji TBM s dvojitým štítem ve tvrdých vyvřelých horninách a s abrazivitou podobnou té, která je v případě popsaném dále v tomto příspěvku, konkrétně na dvou tunelech ražených v tvrdé hornině s pásmy oslabení, které se mají provádět buď pomocí TBM bez štítu, nebo TBM s dvojitým štítem. Plánované tunely budou tvořit vysokorychlostní norskou trať z města Ski do Osla. Mají se razit buď pomocí trhacích prací z několika čeleb, nebo pomocí TBM, které mají být použity na tunely délky 7,9 km a 9,6 km. Z důvodu nebezpečí sedání je nutno v různých místech omezit přítoky vody. Data se sbírala většinou pomocí klasifikace podle Q-histogramu více než 300 skalních zářezů, zatímco v případě pásem oslabení se použilo refrakční seismické profilování.

ÚVOD

Ti, kteří znají Q-systém klasifikace horninového masivu, budou znát řadu hodnot Q a přídavná jména „slabý", „uspokojivý", „dobrý" atd., uvedené na obr. 1. Namísto doporučení pro zajištění výrubu se však zde užívá klasifikační diagram, podle kterého se zjišťují obecné trendy



Obr. 1 Metoda Q-systém pro klasifikaci stability výrubu a potřeb jeho výstroje se zde používá k vyjádření relativní obtížnosti ražby tunelu pomocí TBM, s výstupy představy o rychlosti pronikání PR a rychlosti postupu AR

Fig. 1 The Q-system method of classifying tunnel stability and support needs is used here to indicate the relative difficulty of driving tunnels by TBM, with curves to suggest PR (penetration rate) and AR (advance rate)

ABSTRACT

A wide-reaching review of TBM tunneling with open-gripper TBM, as opposed to double-shield TBM, where description of ground is less accurate, forms the basis of a prognosis method called Q_{TBM}. Some lessons from the numerous reviewed cases will be given, followed by a brief introduction to the classification-based empirical method. The TBM performance data base numbers some 145 cases or about 1000 km of TBM tunneling, whose overall performance is synthesized. Since the development of this method the data base has been increased to include double-shield TBM driving in hard igneous rock, also of similar abrasiveness to the case to be described in this paper: namely two tunnels in hard rock with weakness zones, to be tackled either by open-gripper or double-shield TBM. The planned tunnels will form a high-speed Norwegian rail link to Oslo from Ski in the south, to be driven either by drill-and-blast from several fronts, or using TBM for 7.9 km and 9.6 km long tunnels. Water ingress must be limited in various localities due to settlement risks. Data collection was mostly by Q-histogram classification of more than 300 rock cuttings, while some drilling and seismic refraction profiling was utilized in the case of weakness zones.

INTRODUCTION

Those familiar with the Q-system of rock mass classification will recognize the progression of Q-values and adjectives "poor', 'fair', 'good' etc. given in Figure 1. However, instead of tunnel support recommendations, the classification diagram has been used here to suggest the general trends of penetration rate (PR) and advance rate (AR) with TBM tunneling. Clearly the Q-system parameters RQD, Jn (number of joint sets), Jr (joint roughness), Ja (joint alteration), Jw (joint water inflow) and SRF (stress reduction factor) need additional machine-rock interaction parameters, in order to tackle the much more complex field of TBM tunneling, where PR can vary from extremes of 0.1 m/hr to 10 m/hr, and AR can vary from 0.0 m/day to 125 m/day, or indeed from 0.0 km/yr to 16 km/yr. This is a demanding field for classification and for understanding the special circumstances of TBM tunneling.

Figure 2 shows the scheme of classification that was developed by trial and error analysis of many of the 145 cases derived from 1000 km of TBM tunneling reviewed by Barton, 2000. Because open-gripper cases formed the great majority, and significant detail had therefore been given of the rock mass conditions and rock properties, this must be considered the basis of the overall trends of deceleration with increased time period, shown in Figure 3. Experience with double-shield TBM has since suggested a possible halving of these gradients.

The classic equation used to describe all TBM performance data is following:

$$AR = PR \times U \qquad (1)$$

(where U = utilization in a given time period, such as 24 hours, 1 week, or 1 month)

It may be noted in Figure 3 that U has been recast in the form Tm, where T is (total) hours, and *(-) m* is the negative gradient of deceleration seen in the performance lines in Figure 3. Therefore:

$$AR = PR \times Tm \qquad (2)$$

2010

TBM prognosis for open-gripper and double-shield machines tunnelling through hard jointed rock with weakness zones

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Abstract

A wide-reaching review of TBM tunneling with open-gripper TBM, forms the basis of a prognosis method called Q_{TBM}. Double-shield TBM case records were not used in this initial development as description of the geology and jointed ground is difficult and therefore rather poor. The method of prognosis, using a simple input and calculation model Q_{TBM} has been used on many projects since its development in 2000. The TBM performance data-base numbers some 145 cases representing about 1000 km of TBM tunneling. Since the development of this method, the case record data base has been increased to include double-shield TBM, specifically driving in hard igneous rocks, and of similar abrasiveness to the case to be described in this paper. Open-gripper or double-shield TBM may be used to form a future high-speed rail link from Oslo to Ski in the south, involving 9.6 km and 7.9 km long tunnels. The predicted times for individually driving the two tunnels, using the two TBM options, ranged from about 13 to 41 months.

The shorter tunnel in the south is quite shallow. Data collection to represent likely rock mass conditions, for input to the Q_{TBM} prognoses, was based on Q-histogram classification of more than 300 rock cuttings, during a three weeks field logging campaign. Core logging of the lowest quality sections of seven inclined drillholes, for correlation with local seismic refraction profiling, was utilized as input, when modelling weakness zones. P-wave velocities of 2.2, 2.7 and 3.4 km/s were found to be the mean values for three major groups of weakness zones. Penetration rates and advance rates were estimated for five rock mass classes, with Q-values ranging from 1 to about 200, and for the three weakness zone classes, with a range of widths approximating 18 to 20 m. The most frequent rock type logged was granitic tonalitic gneiss, with lesser frequencies of quartz- and feldspar-rich gneiss, granitic gneiss, and amphibolites. Mostly UCS was equal to or greater than 200 MPa, and the adverse cutter life index CLI values were typically from 5 to 10. Cutter forces modelled were generally from 22 to 32 tnf, but lower in the weakness zones.

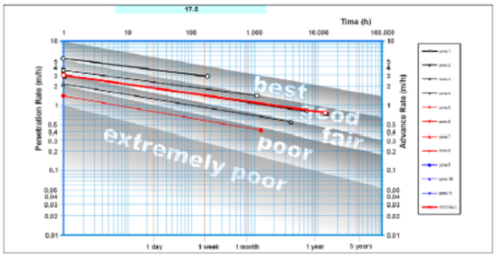

Fig.1 Example of the Q_{TBM} prognosis for the 9.6 km North Tunnel (18 months), assuming double-shield excavation, hence the lesser gradients than for open-gripper TBM (see 'open-gripper' adjectives in figure back-ground). Altogether, some 300 rock cuttings were Q-logged to obtain necessary rock mass quality data for the two tunnels. This was combined with the rock-machine data.

2010

An Engineering Assessment of Pre-Injection in Tunnelling

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ABSTRACT: Water is one of the most difficult of the adverse parameters needing control when driving tunnels. If significant inflows are suddenly occurring at the new tunnel face, the needed control is already too late, as post-injection has to be at lower pressure, and even sealing of leaking bolt holes is time-consuming and frustrating work. The water under pressure is drawn down to atmospheric pressure in an irresistible manner, and any soft materials may also be eroded, possibly allowing rock-blocks to fall and sudden in-rushes to be facilitated. Pre-injection of the rock mass some tens of meters ahead of the face, using high pressure if possible, has been shown to 'normalise' progress, largely removing surprises, and making penetration of even serious fault zones possible. This paper addresses successful use of pre-injection, in which the prediction of groutable joint apertures, grout penetration limitations, and possible grout take volumes per cubic metre of rock, can each be estimated, as a result of 5 to 10 MPa pre-injection pressures. Joints are obviously opened more than in the preceding Lugeon tests, and many rock mass properties can apparently be improved if stable, non-bleeding, non-shrinking cement-based materials are used. The one day delay for each grouting screen, when planned for, proves a good investment in overall tunnelling progress.

1 INTRODUCTION

Norwegian unlined HEP pressure tunnel designs took many years to reach heads of 1000m, after eventually learning to trust in the larger minimum rock stress that prevents leakage. It has also taken many years to reach 10 MPa injection pressures when pre-grouting ahead of tunnels, where inflows need to be controlled to between 1 and 5 litres/min/100m, or where tunnel stability needs improvement, or both of the above. Three recent high-speed rail tunnels, driven through variable geology under built-up areas towards the capital city Oslo, have benefited from a total of 12 km of systematic pre-injection. These experiences have demonstrated the possibilities for pre-injection prognosis, and most importantly have shown that rock mass properties are improved, and support needs are reduced. Progress is a constant 15 to 20 m per week for the completed tunnels.

The pre-injection performed in the first tunnel was focussed on the natural (above-tunnel) environment, and different classes of inflow were pre-designed, according to assumed sensitivity to ground-water draw-down. The last tunnel was injected more strictly, with emphasis also on the long-term tunnel environment. Completely dry arches (observed), dry walls (observed) and dry inverts (presumed), seem to have been achieved in 99.9% of the typical limestone, shale and igneous-dykes geology. Inflows as low as 1 litre/min./100 m were achieved, roughly equivalent to 10^{-9} m/s permeability. Overbreak was greatly reduced, and support needs also reduced.

Table 1 Approximate costs of pre-injection needed to achieve various levels of 'dryness' in 90 m² tunnels.

Inflow (approx)	Cost
20 l/min/100 m	1,400 US $ /m
10 l/min/100 m	2,300 US $ /m
5 l/min/100 m	3,500 US $ /m
1-2 l/min/m/100 m	≈ 5,000 US $ /m

Do we know the actual effects of this high pressure injection on the rock mass? Can effects be quantified in any way? The answers are yes to both questions, because it has been found from recent Norwegian tunnelling projects that high pressure pre-injection may be fundamental to a good result: i.e. much reduced inflow (usually zero), improved stability, little over-break, and an obvious need for less support. Part of the reason for a good result is that the injection pressures used ahead of Norwegian tunnels are far higher than have traditionally been used. Even at dam sites, where, maximum grouting pressures for deep dam foundations have been limited to about 0.1, 0.05 and 0.023 MPa/m depth in Europe, Brazil and USA respectively: Quadros and Abrahão (2002). Increased seismic velocity is seen as one of the results, plus at least some of the desired reduction in permeability. Various results of pre-injection have been reviewed in Barton (2006), and estimations of improved rock mass properties were presented in Barton (2002).

2011

Numerical modelling of two stoping methods in two Indian mines using degradation of *c* and mobilization of φ based on *Q*-parameters

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ARTICLE INFO

Article history:
Received 27 May 2010
Received in revised form
19 May 2011
Accepted 11 July 2011

Keywords:
Q
Q_c
Cohesion component
Frictional component
Displacements
Modulus
Depth dependence

ABSTRACT

Two Indian mines are the subject of a comparative study of a strain-softened Hoek–Brown and FLAC 3D modelling, and a novel '*c* then tan φ' strain-softening-strain-mobilization approach, using *Q*-system based input data. This approach is also used with FLAC 3D, using identical stope geometries. The parameters *CC* and *FC*, denoting the cohesive component and frictional component of shear strength, are extracted directly from the *Q*-logging and knowledge of *UCS*, and are the source of the peak values. Measured deformations, or the strains recorded over the total length of pre-mining installed MPBX, are compared and effectively calibrate the models, in view of the very similar deformations obtained from empirical formulations based on *Q* using the competence factor approach, as in SRF. The '*c* then tan φ' approach appears to give the most realistic match to observations in the mines, including the modelling of a shear band within the back or roof of a stope, rather than at the surface of the stope. The *Q*-based approach also uses a depth-dependent modulus, and this is perhaps the reason why the strain-softened Hoek–Brown model, without this stiffening with depth, shows 'global failure' in a second mine having a wider range of depths within one model, and many openings, since modulus is not increased in standard-method approaches.

1. Introduction

Determination of input parameters for numerical modelling of rock masses, though apparently made 'simple' if one follows the GSI-based Hoek–Brown formulations and standard commercial software, is inevitably a very poorly quantified area of rock mechanics, when one considers the actual complexity and variation within any given rock mass. Those whose job it is to log core, mining drifts or tunnel walls, and record the variability, know they are committing a gross simplification if they later have to choose, or allow modellers to apply, single *RMR*, *Q* or *GSI* values, even for single domains.

The geotechnical behaviour of the rock mass, whether of the real variably jointed-partly intact medium, modelled with different joint-set properties in UDEC or 3DEC (including numerically glued 'joints'), but especially when simplified as an isotropic continuum, is inevitably rather poorly quantified. This is despite the 'good feeling' one may have in seeing nicely defined linear or non-linear strength envelopes. Actual deformation and failure modes are a subject of great uncertainty, and controversy, especially in the case of attempted 'continuum' modelling, forced on us 'by the scale of the problem'.

A long time ago, in the late 1960s, there was a move to try to advance beyond the confines of continuum modelling, and focus on the possible actual effects of jointing on the performance and reinforcement needs of rock excavations, be they tunnels, stopes, slopes or dam abutments. Thanks to the late 1960s modelling developments of Goodman and his colleagues with joint elements in FEM codes, immediately followed by Cundall, first with μDEC, then UDEC and later with 3DEC, this focus on greater reality could be fulfilled by an increasing number of rock mechanics practitioners around the world. However, utilizing these codes correctly, with realistic input data, including geometric aspects, needs experience, time and therefore budgets to match. Furthermore 'the scale of the problem', at least in mining, still causes the need to approximate with continuum modelling and elasto-plastic behaviour approximation. This 'fall-back' method, used only because of necessity, is also reported here, but with some important differences in relation to conventional methods of developing input data and its application in the models. Promising trends are indicated.

2. Shear strength of rock masses is a non-trivial subject

The conventional addition of cohesion (*c*) and the tangent of friction angle (tan φ), in continuum models, either in linear Mohr–Coulomb form, or in a non-linear Hoek–Brown formulation, is unfortunately suspect, when one considers that the

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doi:10.1016/j.ijrmms.2011.07.002

2011

From Empiricism, Through Theory, To Problem Solving in Rock Engineering: a shortened version of the 6th Müller Lecture

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ABSTRACT: The behaviour of the jointed-and-faulted-anisotropic-water-bearing media that we call rock masses, was an abiding pre-occupation of Leopold Müller. The author has been similarly pre-occupied. So starting with modest developments from tension-fractured physical models, and progressing to the real jointed and three-dimensional world in due course, a few of the numerous lessons learned and subsequently applied in rock engineering practice will be described. These included non-linear and block-size dependent shear strength, no actual cohesion, and the possibility of thermal over-closure if rock joints are rough. A six orders of magnitude rock quality Q-scale has proved essential. Discontinuous behaviour provides rich experiences for those who value reality, even when reality has to be simplified by some empiricism.

KEYWORDS: rock joints, rock masses, physical modelling, empiricism, site characterization, tunnelling, rock failure

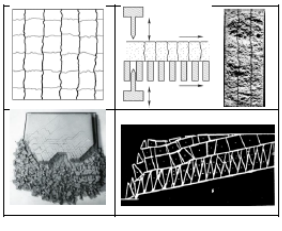
1. INTRODUCTION

Figure 1. Confronted with this potentially unstable jointed rock slope, multiple reasons for the over-break and instability suggest themselves. There are clearly adverse values of JRC, JCS, and ϕ_r, and there are also adverse ratings of Jn, Jr, Ja (and Jw on occasion).

The lessons learned during the development of these empirical parameters, which are now widely used in many countries, will be summarised in the following pages. Their application has been in widely diverse projects.

2. TWO-DIMENSIONAL ROCK MASSES SIMULATED WITH PHYSICAL AND NUMERICAL MODELS

The desire to model the behaviour of jointed rock slopes in late nineteen sixties Ph.D. studies at Imperial College, led to tension-fracture models by the writer, and numerical modelling developments (pre-μDEC) in the case of student colleague Peter Cundall. The relative *inflexibility* and



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flexibility of the two approaches is readily imagined from Figure 2. The single numerical slope model demonstrates the influence of changed friction angles, and was reported some years later, in Cundall et al., 1977 (1975 conference).

Despite the shortcomings of physical tension-fracture models, the writer nevertheless discovered that the peak shear strength of these rough and clearly unweathered tension fractures could be described by a simple relation involving the *uniaxial compression strength* (σ_c) of the model material (Barton, 1971). This was to prove useful.

$$\tau = \sigma_n \tan [20 \log (\sigma_c/\sigma_n) + 30°] \quad (1)$$

This equation, and simple links to peak dilation angle, proved to be the unweathered and roughest 'end-member' of the Barton and Choubey, 1977 equation for the peak

2011

Assessing Pre-

Nick Barton of Nick Barton & Associates, Oslo, Norway addresses successful use of pre-injection, in which the prediction of groutable joint apertures, grout penetration limitations, and possible grout take volumes per cubic metre of rock, can each be estimated, as a result of 5 to 10MPa pre-injection pressures. Joints are obviously opened more than in the preceding Lugeon tests, and rock mass properties can apparently be improved if stable, non-bleeding, non-shrinking cement-based materials are used. The one-day delay for each grouting screen, when planned for, proves a good investment in overall tunnelling progress.

NORWEGIAN UNLINED HEP pressure tunnel designs took many years to reach heads of 1000m, after eventually learning to trust in the larger minimum rock stress that prevents leakage. It has also taken many years to reach 10MPa injection pressures when pre-grouting ahead of tunnels, where inflows need to be controlled to between 1 and 5 litres/min/100m, or where tunnel stability needs improvement, or both of the above. Three recent high-speed rail tunnels, driven through variable geology under built-up areas towards the capital city Oslo, have benefited from a total of 12km of systematic pre-injection. These experiences have demonstrated the possibilities for pre-injection prognosis, and most importantly have shown that rock mass properties are improved, and support needs are reduced. Progress is a constant 15 to 20m per week for the completed tunnels.

The pre-injection performed in the first tunnel was focussed on the natural (above-tunnel) environment, and different classes of inflow were pre-designed, according to assumed sensitivity to groundwater draw-down. The last tunnel was injected more strictly, with emphasis also on the long-term tunnel environment. Completely dry arches (observed), dry walls (observed) and dry inverts (presumed), seem to have been achieved in 99.9% of the typical limestone, shale and igneous-dykes geology. Inflows as low as 1 litre/min /100m were achieved, roughly equivalent to 10^{-9}m/s permeability. Overbreak was greatly reduced, and support needs also reduced.

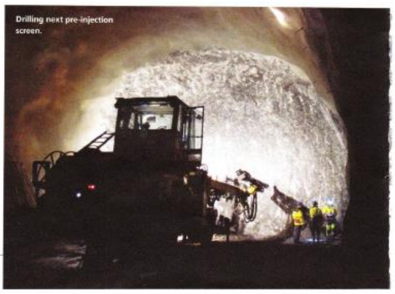
Table 1: Approximate costs of pre-injection needed to achieve various levels of 'dryness' in 90m² tunnels.

Inflow (approx.)	Cost
20 l/min/100m	US$1,400/m
10 l/min/100m	US$2,300/m
5 l/min/100m	US$3,500/m
1-2 l/min/m/100m	≈ US$5,000/m

Do we know the actual effects of this high pressure injection on the rock mass? Can effects be quantified in any way? The answers are yes to both questions, because it has been found from recent Norwegian tunnelling projects that high pressure pre-injection may be fundamental to a good result: i.e. much reduced inflow (usually zero), improved stability, little over-break, and an obvious need for less support. Part of the reason for a good result is that the injection

pressures used ahead of Norwegian tunnels are far higher than have traditionally been used. Even at dam sites, where, maximum grouting pressures for deep dam foundations have been limited to about 0.1, 0.05 and 0.023MPa/m depth in Europe, Brazil and USA respectively: Quadros and Abrahão (2002), increased seismic velocity is seen as one of the results, plus at least some of the desired reduction in permeability. Various results of pre-injection have been reviewed in Barton (2006), and estimations of improved rock mass properties were presented in Barton (2002).

In the photos below, the typical appearance of pre-injected tunnels is shown. The foreign visitors with yellow reflection vests, far out-number the specialist tunnel workers. In the left photo, the second (final) layer of S(fr) covers the systematic CT (corrosion protected) bolting, and drilling for the new pre-injection screen has begun in the right invert. In the right photo, the first layer of shotcrete (still curing) has been followed by systematic CT bolting. Due to the lack of overbreak despite the limestone and shale, the permanent support of B + S(fr) appears to be, and indeed is very conservative. However this is for twin-



2011/2012

A measured response

In a letter addressed to the editor, Nick Barton outlines the discrepancies and inaccuracies that he sees in the paper 'Designs in Jointed Rock', by Walter Wittke and René Sommer, published last month in *World Tunnelling*

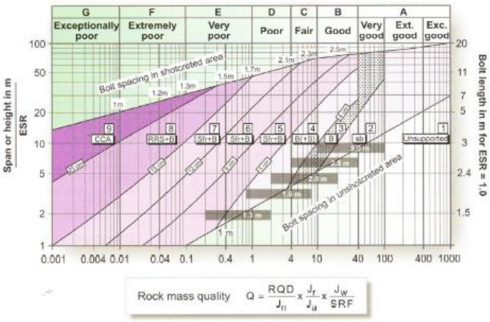


Figure 1: the 1993 Q-support chart that was omitted by Wittke and Sommer, who preferred to use the outdated B+(S)mr chart from 1974, in their critique of classification and Q-based methods. The six Q-parameters are described in the right margin and the relevant adjectives are listed above each individual histogram. Appropriate ratings appear below each histogram

IT should be gratifying to have one's first paper on tunnelling referred to, and criticised, in the pages of *World Tunnelling* – 37 years after publication. However, the Wittke and Sommer (WBI company) comparison of their favoured FEM-based tunnel design method, with their limited understanding of rock mass classification, shows that the authors have made little attempt to follow developments in this field.

They reproduce the very first Q-based 'support selection' figure from 1974, with 38 support categories (needing tables for support selection), not mentioning that B + S(mr) – mesh reinforced shotcrete – was the recommended 'lining' at that time.

Since 1993, the Q related support selection chart was specifically updated for B + S(fr) – fibre-reinforced shotcrete, which has been used commercially in Norway since 1978. Perhaps the authors have another reason for not mentioning this difference if the cumbersome S(mr) is still in use on some of their German projects. It still seems to be in use in Austria, judging by NATM advertisements.

Surprisingly, Wittke and Sommer do not reproduce the widely referred B+S(fr) support selection chart, originating from Grimstad and Barton, 1993, and Barton and Grimstad, 1994 (the latter published in Austria in English).

However, they must have seen the more updated support chart (reproduced here in colour (figure 1)) since, as one of their topics for Q-critique, they selected a figure about bolt spacing from this period.

Possibly, this figure has been reproduced from others who were also criticising Q. Such people have demonstrated a tendency to gather support from each other, some not even reading the paper being criticised, judging by the comments made.

In Scandinavia and many other countries, the less efficient and less safe to apply S(mr) tunnel 'lining' method ceased to be used decades ago. Tunnel workers have been injured in several countries during its three-process application.

The term 'shotcrete membrane', frequently used by the authors, is not used in English. They do not define, perhaps deliberately, what they mean by this; ie whether S(mr) or S(fr). There is an important difference because S(fr) can be →

"Such people have demonstrated a tendency to gather support from each other, some not even reading the paper being criticised"

2011

Designs in jointed rock

Dr Walter Wittke and Dr René Sommer compare rock mechanical models and classification systems, and ask whether their application involves any risk

TWO frequently applied design methods for tunnels in jointed rock are the rock mechanical models and corresponding analysis, and the plan based on classification systems.

The first method is mainly based on the results of comprehensive geotechnical investigation and stability analysis, as well as on monitoring during construction. It is applied predominantly in German-speaking countries, and has proven to be successful for the safe and economic design of tunnels in jointed rock. Confidence in the method has also been gained by numerous post-analyses of completed tunnels.

A characteristic feature of classification systems is that rock-mass properties and other factors influencing the stability of a tunnel, such as in-situ stress and groundwater conditions, are condensed into a single numerical value, referred to as the 'rock mass rating index'.

According to Mr ZT Bieniawski, the developer of the RMR classification system, "a classification system is not intended to replace analytical modeling, site investigations and monitoring, but should be used in conjunction with these tools of rock engineering design".

In contrast, classification systems in the recent past have been used increasingly as design methods in their own right, without any kind of analysis.

This paper will review critically two design methods for tunnels in jointed rock and compare them using examples.

DESIGN: ROCK MECHANICAL MODELS

This design method's basic procedure is outlined in figure 1. A rock mechanical model is established, based on the results of geotechnical investigations, and engineering and geological judgement. This includes models describing the structure and texture of the intact rock and discontinuities. In addition, the rock mechanical model includes parameters that describe the stress/strain behaviour and permeability of the rock mass for all of the units through which the proposed tunnel will pass.

The in-situ stress state must be considered. The orientation of the discontinuity system with respect to the tunnel considerably influences the type and amount of support measures, and thus is an important aspect of every model (figure 1).

As an example, figure 2a represents schistose rock (clay slate) and the corresponding rock mechanical model. The clay slate is separated mainly by three sets of discontinuities, denoted with D1, D2 and Sch. Their orientations are found to be approximately vertical (D1 and Sch) and horizontal (D2). The intact rock has a planar grain structure caused by the schistosity Sch, which, in this particular case, has the same orientation as the bedding. The structural model of this rock mass and the most important rock mechanical parameters leading to the corresponding rock mechanical model are represented in figure 2b.

On the basis of such rock mechanical models, stability analyses and serviceability proofs are carried out, taking into consideration parameter

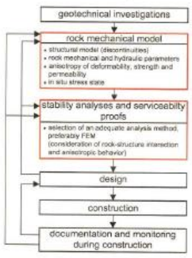
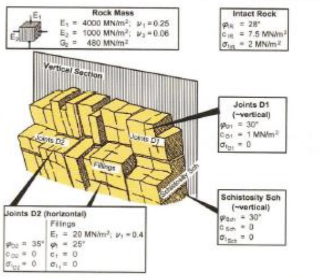


Figure 1: design based on rock mechanical models

Figure 2: example of a rock mechanical model –
a) clay slate
b) corresponding rock mechanical model



2011

Cue: Nick Barton

Nick Barton is an internationally acclaimed tunnel engineer, known for devising the Q-system of rock classification, who has researched and worked on a large number of tunnel projects worldwide. He talks to George Demetri

Describe your education and career to date

Teenage experiments in constructing miniature earth dams for impounding water and creating artificial floods near our stone cottage in Wales led me to a BSc degree course in civil engineering at King's College, London, in 1963.

Guidance from my 'prof', Kevin Nash, then led me to a newly formed rock-slope research team at Imperial College, where my PhD student colleagues included John Sharp (director of Geoengineering) and Peter Cundall, of subsequent UDEC and 3DEC numerical modelling fame. Perhaps Mr Cundall was stimulated by my inflexible, intersecting tension-fracture model studies of steep, excavated rock slopes using 40,000 miniature 'rock' blocks so that he soon created something more user-friendly for the profession.

Halfway through my PhD (1966-70), a memorable Thames-side lunch with Mr Nash's Danish colleague, Dr Laurits Bjerrum, director of the Norwegian Geotechnical Institute (NGI), eventually led me to Norway and employment in NGI's dam, rock and avalanche division.

The 25 years spent at NGI (1971-80 and 1984-2000) formed the primary influence in my professional career in rock mechanics and rock engineering. The first period involved both time and research budgets for developments in rock engineering and rock mechanics (Q-system, and rock and joint-strength criteria; the latter with the roughness parameter JRC). It was also the beginning of extensive national hydropower and foreign-project tunnelling work.

After four valuable years of research and in-situ testing at Terra Tek, Salt Lake City, my second period at NGI comprised five years of administrative and technical duties as division director. Reservoir engineering and rock mechanics laboratory groups were added to our previous dam and avalanche groupings, following the challenges of North Sea petroleum developments, including reservoir subsidence and borehole stability.

In my last ten years at NGI, I held project manager and technical advisor roles in numerous foreign projects, involving nuclear waste site characterisation (eg UK Nirex, SKB Stripa), hydropower project tunnels, caverns (eg Gjøvik) and dams, road tunnels, and bridge foundations (Hong Kong).

Since 2001, Nick Barton & Associates – a mostly one-man international consultancy – has brought me to many more challenging projects in a total of 35 countries. New experiences and travels occur every few weeks in a never-ending contact with the frequently unpredictable in-detail hydrogeological environment found in numerous exotic project sites. These include double-curvature arch dams that exceed 300m in height, and railway tunnels (and TBMs) stuck in lower Himalayan thrust belts. In 2000 and 2006, I wrote a book on TBM prognosis (QTBM) and also a cross-discipline text book on rock quality and seismic velocity.

Proving a source of pride, but also a significant challenge for 2011, has been my designation as the sixth Mueller Award lecturer at the next International Society for Rock Mechanics (ISRM) congress. This award honours the memory of our first ISRM president.

My chosen title will be: 'From empiricism, through theory, to problem solving in rock engineering'. Both in India and Hong Kong (frequently), and in China, this lecturer or short-course deliverer is introduced as "having a PhD from Empirical College", so the selected title of this lecture was self-evident.

You devised the Q-system of rock-mass characterisation in 1973-74. Describe how this came about and why it took so long for something like that to happen in the tunnelling sector

The Norwegian State Power Board (subsequently Statkraft) posed a request for a technical explanation as to why Norwegian hydropower caverns were displaying widely different magnitudes of deformation. This agency, which owns most of the world's electricity-generating capacity, was apparently not hurt by waiting more than six months for my report, which could not be written until a rock-mass classification method had been developed.

The nature of the question (chance or fate?) meant that rock-mass quality, rock-support needs (shotcrete) and rock-reinforcement needs (bolts and anchors) for different sized openings, situated at widely different depths, needed to be linked to the different deformations recorded. This was a different and more challenging problem than addressed when Bieniawski developed RMR (rock mass rating) one year earlier – which I was not aware of.

The Q-value scale, and its six orders of magnitude, was gradually developed over six months of trial and error. The scale proved capable of answering the question posed, and has since proven to have simple links to rock mass, and joint and discontinuity shear strength, deformation modulus, seismic velocity and seismic attenuation, as well as tunnel and cavern-support needs, at depths from the surface to about 3km.

Why such a system (and Bieniawski's RMR from 1973) was not developed long before may perhaps relate to the increasing use of more economic single-shell solutions; these are epitomised the world over in our big caverns of 15-60m span. But, these solutions have been slow to achieve acceptance in our much smaller-section tunnels; notably those supported by the so-called NATM, where even the use of fibre-reinforced shotcrete has been slow to arrive in relation to its early use in Scandinavia.

How have approaches to tunnel support methods changed in recent years, if at all?

Chance or fate bought me to Norwegian 'nominally unlined' hydropower tunnel territory in 1971, which eventually amounted to more than 3,500km of such tunnels. Road and rail tunnels totalling some 1,500km have had the more conservative – but also single-shell – treatment of permanent support and reinforcement.

"Why select multiple-budget projects to save a few small percentage points on maintenance of escalators?"

"Efficient pre-injection ensures project longevity and more predictable lifetime budgets"

2011

Dr. Nick Barton's interview

Zagreb, 02.06.2011

Nick Barton

Ivan Vrkljan

Dr. Nick Barton was interviewed by Prof. Ivan Vrkljan during Dr. Barton's stay in Croatia from 1 to 6 June 2011 Dr. Barton came to Zagreb to hold a short course entitled "Rock Engineering for Tunnels (Drill-and-Blast an TBM), Pre-Grouting, Caverns, Dam Abutments, Rock Slopes and Rockfill". The course was organized by th Croatian Geotechnical Society (CGS) and the event was hosted by the Faculty of Mining, Geology and Petroleun Engineering of the University of Zagreb. Dr. Barton also gave the 10th Nonveiller Lecture entitled: "Pre-Groutin for Water Control and for Rock Mass Property Improvement". Nonveiller Lectures are organized by the Croatia Geotechnical Society in honour and memory of professor Ervin Nonveiller. On the occasion of this lecture, CG awarded the plaque of recognition to Dr. Barton in deep appreciation of the scientific and professional suppor given to the Croatian Geotechnical Society. Ivan Vrkljan is a Full Professor for the Engineering Rock Mechanic at the Faculty of Civil Engineering of the University of Rijeka, and the Head of Geotechnical Laboratory at th Institut IGH in Zagreb. He is also the Secretary General of the Croatian Geotechnical Society.

Brief information about Nicholas R. BARTON

Dr. Nick Barton was educated at the University of London from 1963 to 1970 and has a B.Sc. degree in civ engineering from King's College, and a Ph.D. degree on rock slope stability from Imperial College.

One of Dr. Barton's principal contributions to rock mechanics is his work related to discontinuities in the roc mass. In the course of 1972, while he conducted research work at the Norwegian Geotechnical Institute, h developed the peak shear strength criterion for rock joints, which had already been presented in his Ph.D. thesi on Rock Mechanics defended in 1971 at the London's Imperial College. He introduced a modification of thi criterion in 1976 (basic frictional angle was replaced by residual frictional angle ϕ_r), and in 1978 (mobilization an degradation of joint roughness JRC with displacement). He also introduced the Barton-Bandis Model linkin deformation, dilation and aperture. In 1985 the Barton-Bandis model was installed as a subroutine in th Cundall's remarkable UDEC code, in form of UDEC-BB.

Dr. Barton developed the well known Q system for rock classification which is used in the design of suppor systems, both in tunnels and in large underground caverns. The Q system is also used for rock mas characterization. Dr. Barton linked his classification (Q value) with the deformations in tunnels and caverns, an with rock mass deformability modulus. These relations were improved in 1995 when he found out that th parameter Qc (Q normalised by compressive strength different from 100 MPa) correlates well with seismi velocities and deformability moduli.

In 1999 Barton developed the QTBM method for predicting TBM single-shield and double-shield performanc in jointed and faulted rock, and for estimating TBM tunnel rock reinforcement and support needs.

In 1994 and since then he has actively promoted the Norwegian Method of Tunnelling (NMT) with the Q syste for support selection, as a viable single-shell alternative for permanent tunnel support in countries outsid Norway. This is an alternative to double-shell methods including inter alia the New Austrian Tunnelling Metho (NATM), at least when rock mass conditions are 'normal' (poor, fair, good etc).

2011

SHEAR STRENGTH CRITERIA FOR ROCK, ROCK JOINTS, ROCKFILL, INTERFACES AND ROCK MASSES.

Nick Barton,
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Summary. Although many intact rock types can be very strong, a critical confining pressure can eventually be reached in triaxial testing, such that the Mohr shear strength envelope becomes horizontal. This critical state has recently been better defined, and correct curvature, or correct deviation from linear Mohr-Coulomb has finally been found.

Standard shear testing procedures for rock joints, using multiple testing of the same sample, in case of insufficient samples, can be shown to exaggerate apparent cohesion. Even rough joints do not have any cohesion, but instead have very high friction angles at low stress, due to strong dilation.

Great similarity between the shear strength of rock joints and rockfill is demonstrated, and the interface strength between rockfill and a rock foundation is also addressed.

Rock masses, implying problems of large-scale interaction with engineering structures, may have both cohesive and frictional strength components. However, it is not correct to add these, following linear Mohr Coulomb (M-C) or non-linear Hoek-Brown (H-B) standard routines. Cohesion is broken at small strain, while friction is mobilized at larger strain and remains to the end of the shear deformation. The criterion 'c *then* tan φ' should replace 'c *plus* tan φ' for improved fit to reality. In all the above, scale effects need to be accounted for.

Keywords. Rock, rock joints, rock masses, shear strength, friction, critical state, cohesion, dilation, non-linear, scale effects.

2012

From Empiricism, Through Theory, To Problem Solving in Rock Engineering:

a shortened version of the 6th Müller Lecture

Nick Barton
NB&A, Oslo, Norway.

ABSTRACT: The behaviour of the jointed-and-faulted-anisotropic-water-bearing media that we call rock masses, was an abiding pre-occupation of Leopold Müller. The author has been similarly pre-occupied. So starting with modest developments from tension-fractured physical models, and progressing to the real jointed and three-dimensional world in due course, a few of the numerous lessons learned and subsequently applied in rock engineering practice will be described. These included non-linear and block-size dependent shear strength, no actual cohesion, and the possibility of thermal over-closure if rock joints are rough. A six orders of magnitude rock quality Q-scale has proved essential. Discontinuous behaviour provides rich experiences for those who value reality, even when reality has to be simplified by some empiricism.

KEYWORDS: rock joints, rock masses, physical modelling, empiricism, site characterization, tunnelling, rock failure

1. INTRODUCTION

Figure 1. Confronted with this potentially unstable jointed rock slope, multiple reasons for the over-break and instability suggest themselves. There are clearly adverse values of JRC, JCS, and ϕ_r, and there are also adverse ratings of Jn, Jr, Ja (and Jw on occasion).

The lessons learned during the development of these empirical parameters, which are now widely used in many countries, will be summarised in the following pages. Their application has been in widely diverse projects.

2. TWO-DIMENSIONAL ROCK MASSES SIMULATED WITH PHYSICAL AND NUMERICAL MODELS

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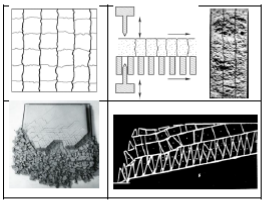

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Despite the shortcomings of physical tension-fracture models, the writer nevertheless discovered that the peak shear strength of these rough and clearly unweathered tension fractures could be described by a simple relation involving the *uniaxial compression strength* (σ_c) of the model material (Barton, 1971). This was to prove useful.

$$\tau = \sigma_n \tan [20 \log (\sigma_c/\sigma_n) + 30°] \qquad (1)$$

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2012

TunnelTalk *Direct by Design*

DISCUSSION FORUM

Defining NMT as part of the NATM SCL debate

Sep 2012

Nick Barton, International tunnelling consultant, Oslo/SãoPaulo

In response to Feedback to the *TunnelTalk* NATM and SCL article earlier this month, I suggested the addition of NMT to the pool of tunnelling method names. If we are seeking definitions, as per the longer Feedback definition of SCL contributed to the original article (Rekindled NATM debate - SCL debate opens - *TunnelTalk*, Aug 2012), then let me try defining and describing NMT a bit more thoroughly, as it is very different from NATM and quite different from SCL.

Key elements of NMT design and execution as 'office-desk'

The Norwegian Method of Tunnelling (NMT) has as one might expect from the name, an origin mostly from Norway. Numerous case records, eventually more than 1,250, were also finally mostly from Norway, but many of the early cases were from Sweden. It is this (and numerous single-shell caverns from many other countries too) that stimulated the original development of the Q-system of rock mass classification and tunnel support class definition. Q was developed in response to a State owner's question - 'why so variable deformations in Norwegian powerhouse caverns'?

The Q-system was always based on economic 'single shell' tunnel and cavern reinforcement and support concepts, for mostly hard jointed rock, which however can often be faulted and have numerous clay-bearing joints and major clay-filled discontinuities. Sometimes solutions are needed for swelling clays as well. All of the above explains why the combination B+S(fr) (rockbolting and fibre reinforced shotcrete) is needed, as both the internal friction and the cohesive strength of the rock mass may be inadequate. Maybe this also applies to London Clay with its 'greasy backs'.

There are some 5,000km of single-shell tunnels in Norway, and of these, 3,500km are for hydropower. Many of the latter are nominally 'unlined', where the Q-value is high enough in relation to the span and the tunnel's use as a water conduit, sometimes with high internal pressure. The early (mostly pre-1980s) method of B+S(mr) using systematic bolting and *mesh reinforced* shotcrete was gradually replaced, starting from about 1978 in Norway. Mesh may have been replaced at about the same time in Sweden, as contractors there also performed large-scale panel tests to demonstrate the superiority of the new fibre reinforced S(fr) product. Norway's first Ph.D. from this era dates from 1981, long before UK studies of S(fr).

2012

Journal of Rock Mechanics and Geotechnical Engineering. 2011, 3 (3): 250–259

Journal of Rock Mechanics and Geotechnical Engineering

Journal online: www.rockgeotech.org

Reducing risk in long deep tunnels by using TBM and drill-and-blast methods in the same project – the hybrid solution

Nick Barton

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Received 13 April 2012

Abstract: There are many examples of TBM tunnels through mountains, or in mountainous terrain, which have suffered the ultimate fate of abandonment, due to insufficient pre-investigation. Depth-of-drilling limitations are inevitable when depths approach or even exceed 1 or 2 km. Uncertainties about the geology, hydro-geology, rock stresses and rock strengths go hand-in-hand with deep or ultra-deep tunnels. Unfortunately, unexpected conditions tend to have a much bigger impact on TBM projects than on drill-and-blast projects. There are two obvious reasons. Firstly the circular excavation maximizes the tangential stress, making the relation to rock strength a higher source of potential risk. Secondly, the TBM may have been progressing fast enough to make probe-drilling seem to be unnecessary. If the stress-to-strength ratio becomes too high, or if faulted rock with high water pressure is unexpectedly encountered, the 'unexpected events' may have a remarkable delaying effect on TBM. A simple equation explains this phenomenon, via the adverse local Q-value that links directly to utilization. One may witness dramatic reductions in utilization, meaning ultra-steep deceleration-of-the-TBM gradients in a log-log plot of advance rate versus time. Some delays can be avoided or reduced with new TBM designs, where belief in the need for probe-drilling and sometimes also pre-injection, have been fully appreciated. Drill-and-blast tunneling, inevitably involving numerous 'probe-holes' prior to each advance, should be used instead, if investigations have been too limited. TBM should be used where there is lower cover and where more is known about the rock and structural conditions. The advantages of the superior speed of TBM may then be fully realized. Choosing TBM because a tunnel is very long increases risk due to the law of deceleration with increased length, especially if there is limited pre-investigation because of tunnel depth.

Key words: TBM, rock strength, deep tunnels, tangential stress, pre-injection, Q-values, utilization, risk

1 Introduction

The writer has been fortunate to get involved in the last stages of several TBM projects where the choice of TBM has clearly been incorrect, and the machine remains in the mountain forever. He has also been involved in projects where drill-and-blast from the other end has been advised at an early stage, but ignored until very late, with adverse consequences on completion dates, due to too late abandonment of the TBM, and fatal consequences for some workers.

Such extremes are unnecessary if more engineers were aware of the inevitable deceleration that accompanies TBM tunneling, notwithstanding 'learning curves' and some good or extremely good progress through favourable rock masses, also meaning favourable hydro-geologies.

Another factor seemingly not universally appreciated is that brittle rock starts to fail around tunnels when the TBM-concentrated tangential stress reaches about 0.4 to 0.5 of the uniaxial compressive strength. This has been independently confirmed in mining and in deep transport tunnels, and will be briefly reviewed later. It occurs in drill-and-blast tunnels, but here the damage zone is actually a favourable aspect, removing the highest stresses from the immediate tunnel periphery.

In recent years with the application of higher and higher grout pre-injection pressures, there has come a

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2012

Hybrid TBM and Drill-and-Blast from the start

Nick Barton of NB&A, Oslo, Norway describes the importance of considering a mixed approach to long and deep tunnel construction

THE WRITER HAS been involved in the last stages of several TBM projects where the choice of TBM has clearly been incorrect, and the machine remains in the mountain forever, or is severely damaged and has to be removed. He has also been involved in projects where drill-and-blast from the other end has been advised at an early stage, but ignored until very late, with adverse consequences on completion dates, due to too late abandonment of the TBM method, and fatal consequences for some workers. Such extremes are unnecessary if more designers were aware of the inevitable deceleration that accompanies TBM tunnelling, notwithstanding 'learning curves' and some good or extremely good progress through favourable rock masses, the latter also meaning favourable hydro-geologies.

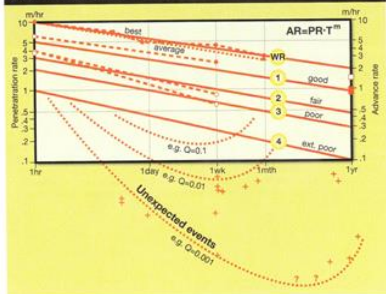
Reversed logic for TBM

TBM tunnelling and drill-and-blast tunnelling show some initially confusing reversals of logic, with best quality rock giving best advance rates in the case of drill-and-blast, since support needs may be minimal. TBMs may be penetrating at their slowest rates in similar massive conditions, if UCS and quartz % are high, due to rock-breakage difficulties, cutter wear, and therefore the need for too-frequent cutter change, the latter affecting the advance rate AR. This 'reversed' trend for slow TBM tunnelling in best quality, highest velocity (Vp) rock has been demonstrated on many projects. The improved rock mass quality associated with higher Vp may not give the expected advantages for TBMs, as less jointing makes for a reduced penetration rate, and an increased frequency of cutter change reduces advance rate.

Law of deceleration for TBM

As an indirect result of several seriously delayed TBM projects, where the writer was eventually engaged as an outside consultant, a wide-reaching survey of case records was undertaken in Barton (2000), in order to try to find a better basis for TBM advance rate prognosis, which also included poor rock conditions. It appeared that 'poor conditions' (as relating to faults) were usually treated as 'special cases' in the industry, with concentration mostly on solving the penetration rate PR and cutter life aspects of TBM prognosis. While jointing effects may be approximately accounted for, the inclusion of faulting delays is usually avoided. The variable strengths of rock masses (as opposed to UCS), compared to cutter thrust levels,

Figure 1: Results of analyses of 145 lengths of tunnel with specific properties, involving about 1000km of open-gripper TBM case records (Barton, 2000). (Note: PR = penetration rate, AR = actual advance rate, U = utilization when boring, and T = time in hours). The best performances, termed WR (world record) are represented by the uppermost lines showing best shift, day, week, and month. At the other extreme, and often explainable by low Q-values, are the so-called 'unexpected events', where faulting, extreme water, or combinations of faulting and water, or squeezing conditions, or general lack of stand-up time, may block the machine for weeks or months. Some examples of the most adverse 'crosses' will be shown later.



2012

The Editor,
Tunnelling Journal

15th July, 2012

Rock Mass Strength Criteria Revisited

The undersigned read with interest the Lowson and Bieniawski article reprinted from ITA Bangkok, concerning 'Validating Rock Mass Strength Criteria'. It is easy to share their enthusiasm for a non-linear intact rock strength criterion that matches test data, as appears to be the case for the Bieniawski, 1974 criterion. However, they go on to 'validate' the Yudhbir-Bieniawski criterion for *rock mass failure*, which involves use of Bieniawski's own RMR rock mass rating, in order to modify two of the constants A and B. A bit confusingly for the reader, they compare their non-linear strength envelopes for *rock masses*, using various RMR values, with what one had assumed was a *linear* Mohr-Coulomb criterion. Since their equations for c and φ do not have any stress dependence, where does the Mohr-Coulomb strength envelope curvature come from? There is no stress term in RMR.

As with many who are researching the strength of rock masses, the authors continue by comparing their rock mass criterion with the Hoek-Brown criterion, and show that GSI and RMR cannot necessarily be made equal (was this ever recommended?), and cannot necessarily be based on the more standard 'GSI=RMR-5', though the latter appears to give the best fit in the majority of their compared cases. Often GSI = RMR-2 seems to 'work'.

The problem is that one is building belief about the shear strength of rock masses based on *a priori* assumptions, and their subsequent use in *a priori* continuum models. Where is the *a posteriori* evidence based on experience of actual performance? Rock masses do not tend to follow continuum behaviour since anisotropic, and nor do they fail by adding cohesion and σ tan φ. Cohesion is broken at smaller strain, and friction is mobilized at much larger strain. So how can one progress far with criteria that have not been verified, or have only been tried in non-representative continuum modelling, that exaggerate 'plastic zones'?

Towards the end of their article they (the Mott MacDonald first author presumably) resort to some unexpected 'Q-bashing'. As the writer gets to review many consultants' presumed understanding of the Q-system, this is something to be more careful with. Does a 25 m long collapse (in Turkey) sound like correct application of the Q-system by those on site? Was this 'practical case' of local collapse an illustration of ....continuum behaviour?

For 35 years now the writer has heard that the Q-system does not take into account 'unfavourable joint orientations'. If this was true, where are all the failures, and why is the Q-method used by so many? Some of us have learned not to trust in unverified strength criteria used in invalid continuum models of what happily is usually a discontinuum, where also the number of joint sets is considered of importance to rock mass description?
Nick Barton, Norway.

2012

Challenges and empirical solutions when tunnelling

Nick Barton reveals some highlights of his 40-year experience of working in the tunnel industry and reviews some important lessons when it comes to hydropower tunnelling.

Nick Barton has had the privilege of working on hydropower projects in many exotic places, during a 40 year time-span, and is hoping to continue during this next decade. Hydropower projects, almost by geographic necessity, can bring one to some of the most beautiful locations in the world. Once there, often after memorable travels, like over-night travel to Canyon Number 5 Railway Station in the incomparable Zagros Mountains of Iran, the rock mass related challenges occupy one for weeks or years, depending on the label 'expert' or 'designer' or 'contractor'. Return visits also occur, as projects progress. Thankfully, the rock mass and the hydrogeology know nothing about 'continuum analyses', and indeed demonstrate this repeatedly. Figure 1 is a simple demonstration of reality. As with medical doctors and their aging patients, gravity never takes a rest, and the rock mass seldom gets stronger, in fact usually weaker on an engineering time scale. There are jobs 'for everyone' – but some widely different answers and opinions. That is part of the fascination and challenge of rock engineering.

This review of some highlights and important lessons will start at the beginning. This was a still-remembered challenge: geological mapping of a short hydro power access tunnel in Western Norway in 1972, with Reidar Lien's instruction to also decide on suitable permanent support. How was one to decide, with limited engineering geological and tunneling experience? We all have to make that first professional start to our careers. Guidelines for rock mass description linked to tunnel support needs seemed to be lacking. Help was needed.

Intense study from 1973-74 and application in the decades following, have provided guidelines that can help in decision making, also for the less experienced, like the writer at that time, and many young people met on-site around the world. The Q-system development followed a question from the Norwegian State Power Board – 'what is the reason for all the different deformation magnitudes in Norwegian power station caverns'? From the start, excavation-size, depth and support quantities and methods had to be evaluated – against a newly created rock mass description – called Q. Luckily this had soon attained almost a log-scale, spanning six orders of magnitude of quality, during all the case-record analysis and re-analysis. A range from 10^{-3} to 10^{3} is not quite enough, but it represented a good start.

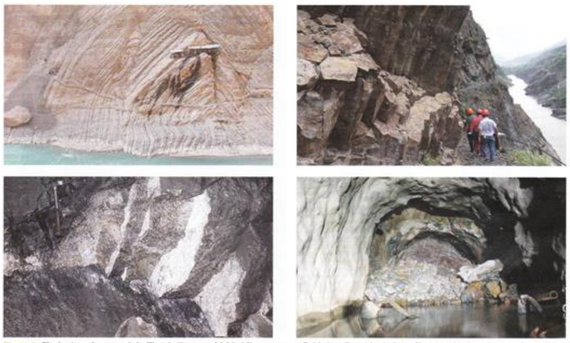


Figure 1. Clockwise – from top-left. The challenges of folded limestones at Bakhtiary Dam site in Iran. Characterization of macro-fractured columnar basalts at Baihetan Dam site in China. Fault-zone and swelling-clay induced failure in Ponte do Pedra, Brazil. Chlorite-filled and graphite-coated discontinuities in a Norwegian headrace tunnel, causing over-break.

2013

ERRORS IN CONSTITUTIVE MODELLING OF ROCK, ROCK JOINTS AND ROCK MASSES

Nick Barton, Oslo, Norway

ABSTRACT

Figure 1 illustrates a figure from Barton, 1976 which was used recently by Singh et al. 2011 to improve the quantification of the triaxial (and polyaxial) strength of *intact rock* up to the critical state of $\sigma_1 = 3\sigma_3$ where the strength envelope finally becomes horizontal. Singh et al. 2011 show that with correct formulation of the strong deviation from the linear Mohr Coulomb equation, the nearly touching Mohr circles for uniaxial strength (σ_c) and critical confining pressure ($\sigma_{3\ critical}$) do actually converge for a majority of rock types. Because the curvature of the whole envelope is more correctly (and simply) formulated, a small number of triaxial tests at low confining pressure are sufficient. Actual strength envelope curvature is greater than that suggested by the Hoek Brown formulation.

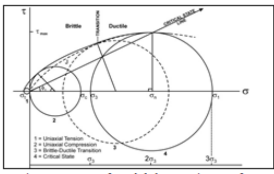


Figure 1. Singh et al., 2011 show that the uniaxial and critical confining pressure Mohr-circles (σ_c and $\sigma_{3\ critical}$) are approximately coincident for the majority of rock types. This is an elegant result.

The direct shear testing of ***rock joints*** is also performed in a large number of laboratories world-wide, and, after correction for scale effects, is important input data for rock slope and open pit design. It therefore has economic implications. Due to 'insufficient numbers' of available joint samples, the understandable practice of 'multi-stage' shear testing has been practiced for many years. This means shear testing at lowest normal stress first, followed by successively higher stresses. Some have even suggested pre-loading to the final higher normal stress before each test. Both of the above cause clock-wise rotation of the strength envelope, creating an artificial cohesion. When direct shear tests are performed only once per sample, no cohesion is registered, as shown in Figure 2.

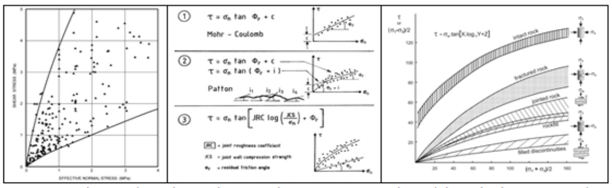


Figure 2. Rock joints do not have cohesion, unless steep steps are sheared through. There is no need to deliberate about whether to ignore cohesion in design. It does not exist. Barton, 1976, 1990, 2006.

On the subject of **rock masses**, it will be shown that it is incorrect to add the cohesion and friction components. One must degrade cohesion (at small strain) and mobilise friction (at larger strain) as in nature. It is time to think afresh about the shear failure of rock masses, as existing jointing is also involved in the process. Black-box algebra does not describe the process, nor do continuum analyses.

Keywords: shear strength, intact rock, rock joints, rock masses, cohesion

2013

Nick Barton & Associates — Rock Engineering

Opinions on most significant tunnelling techniques during the 25 years of WORLD TUNNELLING.

For some of us living and tunnelling in Norway, and also for those venturing much further afield, it was important to have an early glimpse of 'the Norwegian Method of Tunnelling' (NMT) presented in your WT pages in 1992. Although this was a multi-author and multi-company contribution, it of course raised eyebrows and protest from those who could not be included. This article was squarely founded on the remarkable properties of robotically applied steel-fibre reinforced shotcrete S(fr), which had been carefully tested and commercially applied since 1978. It was also focussed on how to select thickness (and bolt spacing) via Q-system logging.

The key concept presented in these particular WT 1992 pages was what has become more and more known as 'single shell' tunnelling, to contrast NMT from the 'double-shell' tunnelling represented by NATM. Already there was some 12-14 years commercially-acquired experience with wet process S(fr) both in Norway and Sweden, and an early Ph.D on the subject of S(fr) was from Opsahl, 1982, who was one of our prominent co-authors in 1992.

The ability to apply accelerated steel-reinforced concrete (or polypropylene-reinforced shotcrete) even from a safe distance over the muck-pile, when stability was compromised, was of course a revolution. But from 1992 and onwards, formal dimensioning guidelines for S(fr) were updated, and gradually spread to other countries. S(fr) in contrast to S(mr) – steel-mesh reinforced – has remarkable advantages, and its gradual spread outside Scandinavia would get my vote for the most important tunnelling technique, for those who wish to put long-term value on all layers of support applied, rather than rely on that delaying and costly concrete lining. Of course rock quality comes squarely into the picture, and the ability to improve rock mass conditions by high pressure pre-injection (increasingly also ahead of TBM) has to be technique number two on the list of important developments during WT's first 25 years. I am sure others are addressing the remarkable developments in the TBM industry.

Regards,

NRBarton

2013

Rock Engineering Challenges and Some Solutions for Hydropower Projects

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ABSTRACT: Hydropower projects in many mountainous regions, but especially in India and Kashmir, present geological, geo-hydraulic and construction challenges perhaps second to none. The tectonic influences, the intense jointing and continued deformation, the contrasting rock types, the high-pressure water - and barriers, the clay-filled fault zones, all combine to test the ingenuity of the designer and especially the contractor. The owner will always acquire something unique, due to the positive influence of human endurance and perseverance. This paper assembles some of the practical lessons learned by the writer during a forty-years professional career, spanning thirty five countries. The main topics will be headrace and pressure tunnels, both by drill-and-blast and by TBM, and how to make these more economic, and perhaps avoid big delays. There will be liberal use of the Q-system, also for its use in TBM prognosis through the Q_{TBM} method. Severe delays can be explained and may be mitigated. Single-shell NMT tunneling is preferred to double-shell NATM due to speed and cost.

1 INTRODUCTION

The writer has had the privilege of working on hydropower projects in many exotic places, during a 40 year time-span, and is hoping to continue during this next decade. Hydropower projects, almost by geographic necessity, can bring one to some of the most beautiful locations in the world. Once there, often after memorable travels, the rock mass related challenges occupy one for weeks or years, depending on the label 'expert' or 'designer' or 'contractor'. Thankfully, the rock mass and the hydrogeology know nothing about 'continuum analyses', and indeed demonstrate this repeatedly. Figure 1 is a simple demonstration of reality. As for medical doctors and their aging patients, gravity never takes a rest, and the rock mass seldom gets stronger, in fact usually weaker on an engineering time scale. There are jobs 'for everyone' – but some widely different answers and opinions. That is part of the fascination and challenge of rock engineering, especially when applied to the solution of hydropower problems, where there are many possible choices. Cost and time *can* be saved.

2 THE Q-SYSTEM BASED SINGLE-SHELL NMT METHOD

With 3,500 km of hydro-power related tunneling, about 180 underground power houses, and hydro-power competition with the investment needs of a growing off-shore oil industry, it was necessary to construct economic tunnels (and power-house caverns) in Norway. The Q-system development from 1974 always reflected this, and 50% of initial case records were from Norwegian and Swedish hydro power projects, with *fifty different rock types* in the first 212 case records. Contrary to popular belief, few cases from the Pre-Cambrian and mostly high quality bedrock could be used, unless they were challenging shear zones with clay-coated joints, and sometimes hydrothermally altered rocks with swelling clays. One cannot develop a rock mass classification system from cases of 'no support needed', when Q is so often in the range 10 to 100 in these basement rocks. Yet some believe Q cannot be used 'in their country' due to all the granitic gneiss that they imagine accounts for the Q-system development. This misunderstanding is perhaps understandable, but is nevertheless a pity.

Fig. 1 Chlorite-filled and graphite-coated discontinuities in a Norwegian headrace tunnel, causing over-break, a typical case record for Q-development. Fault-zone and swelling-clay induced failure in Ponte do Pedra, Brazil.

2013

Journal of Rock Mechanics and Geotechnical Engineering 5 (2013) 249–261

Journal of Rock Mechanics and GeotechnicalEngineering

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and some solutions

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ARTICLE INFO

Article history:
Received 6 July 2012
Received in revised form 27 July 2012
Accepted 2 October 2012

Keywords:
Rock masses
Critical state
Rock joints
Shear strength
Non-linear friction
Cohesion
Dilation
Scale effects
Numerical modelling
Stress transforms

ABSTRACT

Although many intact rock types can be very strong, a critical confining pressure can eventually be reached in triaxial testing, such that the Mohr shear strength envelope becomes horizontal. This critical state has recently been better defined, and correct curvature or correct deviation from linear Mohr–Coulomb (M-C) has finally been found. Standard shear testing procedures for rock joints, using multiple testing of the same sample, in case of insufficient samples, can be shown to exaggerate apparent cohesion. Even rough joints do not have any cohesion, but instead have very high friction angles at low stress, due to strong dilation. Rock masses, implying problems of large-scale interaction with engineering structures, may have both cohesive and frictional strength components. However, it is not correct to add these, following linear M-C or nonlinear Hoek–Brown (H-B) standard routines. Cohesion is broken at small strain, while friction is mobilized at larger strain and remains to the end of the shear deformation. The criterion '*c then* σ_n *tan* φ' should replace '*c plus* σ_n*tan* φ' for improved fit to reality. Transformation of principal stresses to a shear plane seems to ignore mobilized dilation, and caused great experimental difficulties until understood. There seems to be plenty of room for continued research, so that errors of judgement of the last 50 years can be corrected.

1. Introduction

Non-linear shear strength envelopes for intact rock and for (non-planar) rock joints are the reality, but traditional shear test interpretation and numerical modelling in rock mechanics has ignored this for a long time. The non-linear Hoek–Brown (H-B) criterion for intact rock was eventually adopted, and many have also used the non-linear shear strength criterion for rock joints, using the Barton and Choubey (1977) wall-roughness and wall-strength parameters JRC (joint roughness coefficient) and JCS (joint compressive strength).

Non-linearity is also the rule for the peak shear strength of rockfill. It is therefore somewhat remarkable why so many are still wedded to the '$c+\sigma_n \tan\varphi$' linear strength envelope format. Simplicity is hardly a substitute for reality. Fig. 1 illustrates a series of simple strength criteria that predate H-B, and that are distinctly different from Mohr–Coulomb (M-C), due to their non-linearity.

The actual shear strength of rock masses, meaning the prior failure of the intact bridges and then shear on the fractures and joints at larger strains, is shown in Fig. 1 (units of σ_1 and σ_2 are in MPa).

2. Intact rock

The three-component based empirical equations (using roughness, wall strength and friction) shown in Fig. 1 were mostly derived in Barton (1976). The similarity of shear strength for rock joints and rockfill was demonstrated later in Barton and Kjærnsli (1981).

At the time of this mid-seventies research by the writer, it was recognized that the shear strength envelopes for intact rock, when tested over a wide range of confining stress, would have marked curvature, and eventually reach a horizontal stage with no further increase in strength. This was termed the 'critical state', and the simple relation $\sigma_1 = 3\sigma_3$ suggested itself, as illustrated in Fig. 2.

An extensive recent study by Singh et al. (2011) in Roorkee University involving re-analysis of thousands of reported triaxial tests, including their own testing contributions, has revealed the

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

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http://dx.doi.org/10.1016/j.jrmge.2013.05.008

2013

EAGE

Characterization of fracture shearing for 4D interpretation of fractured reservoirs

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Summary

Fractured reservoirs and their successful production my need to involve fracture shearing. This important mechanism may result in slight dilation of the fractures, and therefore substantial maintenance of conducting aperture, despite effective stress increase. Unless fractures are sealed with hard minerals, and channelized flow is occurring, closure would be likely with the standard geophysics model of one set of stress-parallel fractures. The minimum principal stress would close the fractures unless they were very rough and in hard rock, such as limestone. These scenarios suggest the need for fracture characterization, with a view to geomechanical coupled modeling, so that 4D reservoir monitoring results can be interpreted better than with continuum 'stress and strain' arguments, which have little relation to the detailed reality of continued fracture flow during production of petroleum.

Introduction

The usual geophysics interpretation of shear wave splitting, seen in much of the literature of this decade, is that a stress aligned set of microcracks, or a single set of stress-aligned fractures are responsible for the polarization into fast and slow axes, parallel and perpendicular to the assumed micro or macro structures. Of course this is convenient, but do single sets of either feature constitute a naturally fractured reservoir? What about the geomechanics argument of Barton, 2006 that fractures, with their extreme aspect ratio and 'softness', may actually be almost closed at these high minimum horizontal stress levels? Shearing is needed, and estimation of shearing potential requires fracture characterization.

Rock mechanics characterization of fractures

The index tests summarized in Figure 1, allow one to acquire suitable input data for geomechanical (rock mechanics) modeling. The parameters JRC concerning roughness, and JCS concerning wall strength, are fundamental, but easily and cheaply obtained. They can be used to calculate shear strength, shear and normal stiffness, and dilation. JRC_o (the 100mm scale of roughness) allows conversion from conducting (hydraulic) apertures to average physical apertures. Shear stress-dilation-permeability coupling, and normal stress-closure-permeability coupling have been part of distinct element modeling for many years in rock mechanics, such as in UDEC-BB (Universal Distinct Element Code, where BB refers to the Barton-Bandis constitutive model). The details of local fracture deformation and flow determine 4D response, not 'stress and strain', as speculated by some geophysicists.

Evidence of the need for shear stress and deformation

Figure 2 shows a set of deep-well data, in which the hydraulically conductive fractures were distinguished from non-conductive fractures, by means of the interpreted shear stress. This data applied to fractures in harder rock. In fractured reservoirs, shearing would be even more needed to help explain continued production with depletion of reservoir pressure. The exception to this opinion would be minerally 'fixed' and channelized fractures, as are common in some petroleum regions. The coupling of shear with dilation and permeability increase for 'normal' joints and fractures, was described and modelled in detail by N.Barton et al., 1985. (Note unrelated Bartons).

2013

Shear strength of rock, rock joints and rock masses – problems and some solutions

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ABSTRACT: These three important topics, each deserving separate volumes, even when summarized, can only be treated in a single article by linking them. The all-encompassing shear strength of rock masses cannot be described with advanced algebra as in Hoek-Brown, nor as linear Mohr-Coulomb, each of which are *a priori* estimates rather than the desirable *a posteriori* based on experience. The highly non-linear shear strength of intact rock, which has finally been defined as strongly deviated from Mohr-Coulomb, and with more curvature than Hoek-Brown, is the component which fails at small strain. Deep in the crust rocks may be ductile or at their critical state. The very different and weaker joints or fractures provide stability problems in civil and mining engineering, and help maintain some permeability in fractured reservoirs. Joints are highly anisotropic features. They exhibit large differences between their high normal stiffness, and their low, scale-dependent shear stiffness. Joints obviously reach their peak shear strength at larger shear strain than intact rock, and their frictional strength 'remains' after cohesion is lost, as in the words of Müller 1966. It is not correct to add the cohesive strength of the intact rock and the shear resistance of the joints, as in c plus sigma-n tan-phi, nor as in the non-linear form of Hoek-Brown. A third shear strength component may kick-in at larger shear strain: the lower frictional strength of clay-filled discontinuities, such as in the neighbourhood of faults. Finally there is the wide-reaching problem of stress transformation, from principal stresses to normal and shear stress components on a plane. Dilation, shearing and the very presence of the plane violates the theoretical assumptions.

1 INTRODUCTION

1.1 *The importance of rock and rock joints*

Why is shear strength, consisting of *cohesion, friction, and dilation* ultra-important to earth-dwellers? Basically because without their variety there would be no mountains, no river valleys, no deserts, and no oil or gas. Since we are blessed with the variability of these three parameters, we have variable scenery ranging from mountains, to rolling hills, to deserts, and mountain- and sea-cliffs as the source of screes, beaches and eventual inland or coastal sand-dunes. The cycle continues with post-lithification (and post-tectonic) fracturing, breakage of cohesion, block formation, and once more: mobilized friction and dilation. The three strength components do not occur simultaneously, so cohesive and frictional strength need to be partially separated, by degrading cohesion, mobilizing friction, then mobilizing dilation.

1.2 *Economic influence of shear strength*

It is interesting to consider that petroleum reserves have been retained (for our present benefit, and possible downfall), due to cap-rock intolerance of shear stress. Shales and salt therefore exhibit higher levels of minimum principal stress compared to the lower σ_{min} of stiffer petroleum bearing sediments. Furthermore, petroleum reserves have also been retained due to clay-sealing along fault zones, thanks to the weakest product of weathering or hydrothermal alteration. At shallow depth the above minimum stress anisotropy is reversed, and the stiffer beds will tend to attract higher minimum stress than cap-rock shale or salt (Barton, 2006).

1.3 *Economic influence of joint behaviour*

Rock joints or natural fractures provide conductivity during water and petroleum production, by connecting the fluid-storing matrix to the wells. When fracing *oil-shales* for shale-gas production, the incipient (or very tightly closed) jointing may apparently shear and dilate enough to unlock the vast potential reserves in an otherwise impermeable looking rock mass. Micro-seismic activity in *geothermal* reservoirs, also suggests that jointing is mobilized, sometimes in unexpected ways when today's joint orientation is no longer consistent with today's rotated

2014

EAGE

Geomechanics and geophysical processes in naturally fractured reservoirs

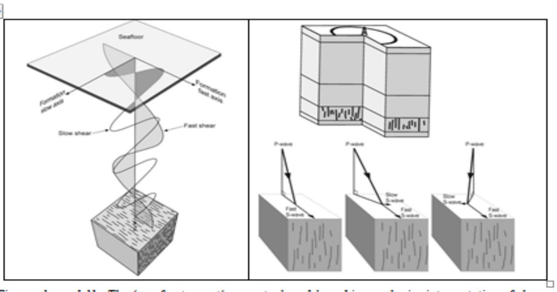
Nick Barton, NB&A, Oslo, Norway

Introduction

In the case of domal reservoirs, production may cause down-dip shearing on conjugate dipping fractures. This mechanism was deduced a long time ago from rock mechanics modeling, and from not previously seen slickensiding of fractures, in the case of the Ekofisk reservoir in the North Sea (Barton et al., 1988). There may also be changes in the stretching over-burden in the case of the compacting reservoir causing the over-burden to subside. This will cause temporal changes to the strength of shear-wave anisotropy and attenuation, due to intra-bed joint opening and shear. It is insufficient in each of the above cases to refer to 'stress or strain' effects, as if a continuum alone was reacting to the multiple effects of production in a multi-km^3 fractured reservoir, with a multi-km^3 overburden. The relatively 'early' consideration of discontinuous behaviour at Ekofisk is presented in this classic 'carbonates region' of the Middle East, in the hope of stimulating the modelling of fracture deformation, and coupled behaviour, which still seems to be rare, despite being needed for realistic 4D interpretation. The possibilities of making good use of geomechanics understanding has improved a lot since LOF monitoring/interrogation of reservoirs was slowly introduced in this last decade, starting in the North Sea.

Standard geophysics interpretation of shear wave polarization

From a remarkably broad range of geophysics literature, from petroleum reservoir exploration, and from geothermal reservoir interpretation, one finds the interpretation of shear wave polarization explained as if resulting from the assumed effect of a set of stress-parallel microcracks, or from the effect of one set of (reservoir) fractures. Two examples are shown in Figures 1a and 1b. As suggested elsewhere (Barton, 2006, 2013) this does not make a very convincing model for a producing petroleum reservoir, or for geothermal energy production. More than one set of fractures, and fractures preferably acted on by significant shear stresses, one would assume were needed.



Figures 1a and 1b. The 'one fracture set' conceptual model used in geophysics interpretation of shear wave polarization. This seems to be missing the geomechanics and production mechanisms needed to explain conductive fractures, and connectivity, at many kilometers depth. From Barkved et al., 2004, and Horne, 2003.

2013

Lessons learned using empirical methods applied in mining

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ABSTRACT: This paper is designed as a broad-brush resume of some methods developed by the writer, which have also found application in mining, even though the focus of the original developments was from civil engineering. Methods summarized will include estimation of shear strength for rock cuttings and benches, with possible application to open-pit slopes. Emphasis will be on the estimation of shear strength-displacement and dilation-displacement for rock joints, with allowance for block size. Rock dump stability assessment will also be touched on. Some adverse practices will be mentioned concerning direct shear testing. Estimation of the shear strength of rock masses will be included in the discussion. This will lead into the Q-system parameters, and how they can assist in shear strength estimation if one is forced by the scale of problem to perform continuum analyses. The Q-system's six parameters can assist in selection of support for permanent mine roadways and shafts. The six Q-parameters are also useful for statistical rock quality zonation of future mining prospects, based sometimes on the characterization of hundreds of kilometers of drill-core. The Q-system's first four parameters have been widely used in mining for stope categorization: stable, transitional, caving, and assessing the need for cable reinforcement. Some parallels between unsupported excavations in civil engineering and in mining engineering will be drawn, with emphasis on ESR, the modifier of span.

1 INTRODUCTION

1.1 *Shear strength of intact rock*

Although many still use linear Mohr-Coulomb, or non-linear Hoek-Brown, it is easily demonstrated that these will introduce inaccuracy if stress ranges are large. A new non-linear criterion, based on an old idea (critical state) has recently been developed, showing correct deviation from Mohr-Coulomb. A few tests at low confining pressures define the whole curved envelope. The critical confining pressure required for (weaker) rocks to reach maximum strength, where the strength envelope becomes horizontal, is found to be close to the uniaxial strength of the rock (Singh et al. 2011, Barton, 1976).

1.2 *Shear strength of jointed rock*

Here it is also found that many still use linear Mohr-Coulomb. Over a limited stress range, and with more planar joints this is defensible. Part of the reason for continued use of a nevertheless uncertain *cohesion intercept* is that multi-stage testing accentuates the apparent cohesion. Shear testing the same joint sample at successively increasing normal stress causes a potential clock-wise rotation of the strength envelope. The preferred method is based on index tests for JRC, using tilt tests (not subjective roughness profile matching), and Schmidt hammer tests for joint wall strength JCS. Scale effects caused by increasing block-size are allowed for using empiricism, not *a priori* assumptions. A useful check of the large-scale JRC is the 'a/L' method, measuring amplitude of roughness between straight-edge contact points, provided joint surfaces are well exposed by over-break, for instance in a bench-face

1.3 *Shear strength of rock masses*

Linear Mohr-Coulomb is still popular despite the existence of the also *a priori* GSI-based, modified Hoek-Brown criterion. A potential problem with these standard methods, in addition to the actually complex, process-and-strain-dependent reality, is that some failure of intact rock ('bridges') may be involved. This genuine cohesive strength is broken at smaller strain than the new fracture surfaces are mobilized. These new surfaces have high JRC and JCS and φ_b. The surrounding joint sets, with lower JRC, JCS and φ_r, may get their peak strength mobilized at still larger strain, followed by eventual clay-filled discontinuities or fault zones, if these are also involved. Since this is a process-and-strain related property, and also non-linear, why are we adding c +

2014

Anisotropy is Everywhere, to See, to Measure, and to Model

Nick Barton · Eda Quadros

Received: 29 May 2014 / Accepted: 18 July 2014

Abstract Anisotropy is everywhere. Isotropy is rare. Round stones are collectors' items, and any almost cubic blocks are photographed, as they are the exception. The reasons for rock masses to frequently exhibit impressive degrees of anisotropy, with properties varying with direction of observation and measurement, are clearly their varied geological origins. Origins may provide distinctive bedding cycles in sedimentary rocks, distinctive flows and flow-tops in basalts, foliation in gneisses, schistosity in schists and cleavage in slates, and faults through all the above. We can add igneous dykes, sills, weathered horizons, and dominant joint sets. Each of the above are rich potential or inevitable sources of velocity, modulus, strength and permeability anisotropy—and inhomogeneity. The historic and present-day stress anisotropy provides a wealth of effects concerning the preferentially oriented jointing, with its reduced roughness and greater continuity. High stress may also have induced oriented micro-cracks. All the above reinforce disbelief in the elastic-isotropic-continuum or intact-medium-based assumptions promoted by commercial software companies and used by so many for modelling rock masses. RQD and Q are frequently anisotropic as well, and Q is anisotropic not just because of RQD. The authors, therefore, question whether the a priori assumption of homogeneous-isotropic-elastic behaviour has any significant place in the scientific practice of realistic rock mechanics.

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Keywords Anisotropy · Anisotropic structure · Stress · Velocity · Modulus · Permeability

List of symbols

BB	Barton-Bandis constitutive model for rock joints
E_{mass}	Static modulus of deformation
E	Average physical aperture of a joint
e	Hydraulic aperture of a joint
EDZ	Excavation disturbed zone
J_a	Rating for joint alteration, discontinuity filling
JCS	Joint wall compression strength
J_n	Rating for number of joint sets
J_r	Rating for joint surface roughness
JRC	Joint roughness coefficient
J_w	Rating for water softening, inflow and pressure effects
K	Permeability (units m/s)
K_n	Normal stiffness of a joint
K_s	Shear stiffness of a joint
K_{int}	Intermediate principal permeability
K_{max}	Maximum principal permeability
K_{min}	Minimum principal permeability
k_o	Ratio of σ_h/σ_v
NGI	Norwegian Geotechnical Institute
Q	Rock mass quality rating (range 10^{-3} to 10^3)
Q_c	Rock mass quality rating (Q, or Q_o normalized by $\sigma_c/100$)
Q_o	Q calculated with RQD_o oriented in the loading or measurement direction
Q_{seis}	Seismic quality factor—the inverse of attenuation (used by geophysicists, normally with the P- and S-wave components 'Q_p' and 'Q_s')
RQD	Rock quality designation (% of core-pieces ≥10 cm in length)

2014

7th INTERNATIONAL SYMPOSIUM ON SPRAYED CONCRETE – Modern Use of Wet Mix Sprayed Concrete for Underground Support
– Sandefjord, Norway, 16. – 19. June 2014

Q-SYSTEM APPLICATION IN NMT AND NATM AND THE CONSEQUENCES OF OVERBREAK

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SUMMARY

The Q-system of rock mass classification for assisting in support and reinforcement selection for rock tunnels and caverns has now been in use for 40 years. During the last 20 years it has been used in order to assist in the choice of permanent single-shell fiber-reinforced S(fr) support and systematic corrosion protected rock bolt reinforcement. Twenty years ago the original S(mr) mesh reinforced recommendations were updated to fiber-reinforced shotcrete, in order to reflect the by then more than ten years of experience of wet process, robotically-applied S(fr) in Norway. This revolutionary product now has a 35 years track record. The Q-system is also used to select rib-reinforced shotcrete arches (RRS) which are superior to steel arches and lattice girders, because intimate contact with the tunnel arch and wall, and systematic bolting of these arches are integral and essential components of the method. The bolted RRS arches therefore help to prevent further deformation instead of allowing it as in NATM, which is a labour-intensive method which does not address these two problems adequately. In this paper some of the other differences between single-shell and double-shell tunnelling will be emphasised, including the frequent use of Q to select only the *temporary support and reinforcement* in double-shell tunnelling, using the 5Q and 1.5 ESR rule-of-thumb. Hong Kong road and metro authorities have applied this method in the last 25 years in hundreds of kilometres of tunnels and in station caverns. The B+S(fr) applied in such cases as the first stage of double-shell NATM, is considered as temporary support and reinforcement, prior to casting the final concrete lining with its drainage fleece and membrane. The temporary support is ignored in the final lining design. This of course is wasteful and adds to the cost. This paper also briefly addresses some of the useful Q-correlations to rock engineering parameters such as P-wave velocity, deformation modulus, and tunnel and cavern deformation. All are depth or stress-dependent.

INTRODUCTION

The first wet-process fiber-reinforced shotcrete applied in Norway was in a hydropower cavern in 1979 and in a main road tunnel in 1981. Mesh-reinforced shotcrete S(mr) ceased to be used by about 1983. The Q-system development in 1974 [1], which was first based on B+S(mr), was updated 'late' in Norway [2] by Grimstad and Barton, but obviously 'early' for many other countries. In Austria, B+*S(mr)*+lattice girders are seemingly still favoured as temporary support for transport tunnels. The writers have been surprised to see Austrian consultants continued recommendation of S(mr) in good quality but over-breaking rock in Asia. However the very strange and diametrically-incorrect instructions are given to accept the use of S(fr) when there is

2014

first break volume 32, September 2014 technical article

Non-linear behaviour for naturally fractured carbonates and frac-stimulated gas-shales

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Abstract
Gas-shales and naturally fractured reservoirs usually produce from several kilometers depth, with fracing-stimulation and eventual water-drive respectively. Due to porosity, the matrix is generally weaker than is typical for basement rocks. The potential pore pressure reduction of 10's of MPa during the early life of the fields, may therefore be a significant proportion of the strength of the matrix. Inevitable non-linear rock strength behaviour for the matrix should not then be ignored. It is therefore unrealistic to utilize a linear Mohr-Coulomb strength criterion as so frequently seen. The joints or natural fractures in the shales and carbonates, which are so important for production, will have producing fracture sets with different roughness and aperture, and few of them are planar enough to follow the frequently used linear Mohr-Coulomb behaviour. Non-linearity especially applies to the favourable shear strength-dilation-permeability coupling which is relevant for both NFR and gas-shales, and to the less desirable stress-closure-permeability coupling of a stress-sensitive reservoir. Non-linear constitutive modelling, partly based on the joint- or fracture-roughness coefficient (JRC) used widely in rock mechanics, also applies to the conversion from hydraulically interpreted theoretical smooth-wall apertures (e) to the larger and non-planar-non-smooth-wall physical apertures (E) through which the oil or gas actually flows to the wells. Simple index tests which can also be applied on joints or fractures recovered in occasional and inevitably expensive core, and which can also be estimated when mapping fractured pavement analogues, have been available in rock mechanics for several decades. They were already incorporated in coupled distinct element (jointed) non-linear modelling routines in 1985. However their implementation in petroleum industry geomechanics seems to be very rare judging by numerous workshops attended in the last seven to eight years on both sides of the Atlantic. Application of non-linear (non Mohr-Coulomb) rock mechanics, using recovered core from Ekofisk in 1986-1987 in order to model fracture shear-dilation coupling with simplified E and e tracking during compaction, may be the earliest example.

Introduction

In the EAGE workshop on *Naturally Fractured Reservoirs in Real Life*, held in Muscat, Oman in December 2013, Price and Wei (2013) from Shell reported on the conclusions from retrospective analyses of eight case studies of fractured reservoirs, mostly related to carbonate reservoirs in Oman. Through extensive fracture network modelling by a large team of company collaborators, the authors identified the most important factors which they considered necessary for *improved history matching*. A production and forecast time-scale of 10 to 40 years was considered in this extensive study by Shell.

Their list of important factors, including their own verbal additions given during their lecture, included the following: *fracture volume, fracture density, fracture clusters, permeability anisotropy due to stress*, and *aperture sensitivity to stress*. Most of these factors would seem to be amenable to realistic conceptual modelling, using rock mechanics principles and available non-linear methods. Such modelling would obviously need to be made at reduced scale at first, using coupled-process distinct element methods, such as the two-dimensional distinct element (jointed) model UDEC-BB (with non-linear Barton-Bandis joint behaviour), or the three-dimensional model 3DEC-MC (with less realistic linear Mohr-Coulomb joint behavior). The detailed behavioural trends thus revealed would subsequently need to be *up-scaled but not lost*, ready for potentially improved reservoir simulator modelling.

Geomechanics and fracture characterization is done differently in rock mechanics

Based on presentations made in the Muscat fractured reservoir workshop, and based on presentations made in several similar workshops and courses attended on both sides of the Atlantic in the last seven to eight years, the writer has gained the strong impression that *geomechanics*, complex enough as it is, seems to be mostly practiced in the petroleum industry without considering *non-linear* shear strength, dilation, stiffness of the different fracture sets. Much of reality is lost if this is true.

The desirable non-linear *shear-dilation-permeability* coupling and the less desirable and very non-linear fracture *aperture-closure* of a stress-sensitive reservoir, due to effective stress change during production, are also apparently not yet a part of open-source petroleum *geomechanics* literature. The aperture-closure would oppose the assumed benefits of major fracture sets 'always' being parallel to the major

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2014

Rock Mechanics for Natural Resources and Infrastructure
SBMR 2014 – ISRM Specialized Conference 09-13 September, Goiania, Brazil

Gas-Shale Fracturing and Fracture Mobilization in Shear: *Quo Vadis*?

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SUMMARY: The conversion of a highly impermeable medium like shale into numerous gas producing 'pay zones', using geomechanics steered, stress-and-structure oriented hydraulic fracturing, is a remarkable achievement. So remarkable in fact that shear mobilization of natural fractures has also to be invoked to explain both the continued though declining production and the sources of larger 'radius' microseismic activity well beyond the assumed ellipsoidally-shaped tensile-fractured and sand-propped 'central' zones. The microseismic activity is believed to be the remote-sensing sign of shearing *initiation* of a large number of the natural fractures. The assumed shearing, and the resulting gas drainage, cannot occur in the case of gas-shales unless the shale is of high enough modulus to sustain the shear induced dilation, which results in a coupling with enhanced fracture permeability. The pre-peak mobilization of roughness and permeability due to pre-peak dilation, combined with low in situ shear stiffness due to block-size related scale effects, is part of the rock mechanics reality behind critically stressed fractures, which are simplified as linear Mohr Coulomb events in petroleum geomechanics. In reality a more sophisticated and more favourable series of coupled processes are likely to be involved.

KEYWORDS: hydraulic fracturing, fracture shearing, dilation, permeability, coupling

1 INTRODUCTION

Shale gas is one of the unconventional sources of natural gas, which has remained trapped in shale, a sedimentary rock which originates from sedimentary deposits of clay, mud, silt and organic matter. The gas must pass through pore spaces that are 1,000 times smaller than in a conventional sandstone reservoir. The gas production, causing large pressure drop even in the first 250 days, depends on multi-stage hydraulic fracking from wells deviated to give long horizontal sections. The remarkable success, starting in the USA, has justified exploration and production. However the local cost to the environment, and the difficulty with sufficient water supply and disposal of fluids are remaining questions, especially in populated areas. Environmental concerns include gas migration and ground-water contamination due to the processes of well construction. However, communication between the fracked region and near-surface groundwater supplies appears to be impossible, due, usually, to thousands of intervening meters of rock. Some of the evidence for this containment, which of course is fundamental to general acceptance of this technology, will be reviewed in the next few pages.

2 FRACKING HORIZONTAL WELLS

Hydraulic fracturing is a well stimulation technique which has been employed in the oil and gas industry since 1947. There are two primary methods to produce shale gas: vertical multi-stage hydraulic fracturing,

2014

An Illustrated Guide to the Q-System following Forty years use in Tunnelling.

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ABSTRACT

This paper provides a well-illustrated guide to the workings of the Q-system, with many examples demonstrating its use. Not only rock exposure logging, but also core-logging, and tunnel-logging are illustrated with quantified examples. The Q-system was developed 40 years ago for describing rock mass quality in a quantitative way, using six important parameters and ratings of quality. These were first related to structural geology, in particular the number of joint sets, their roughness, whether there was clay-filling, followed by the effects of water and the stress/strength ratio. A logarithmic-like scale from about 0.001 to 1000 was the result. All the ratings of the key parameters are given in this guide, and include footnotes and a field-logging sheet and examples of its use. Linked to the Q-value and the span or height of the excavation in rock, and also reflecting the final purpose of the excavation, is an updated chart of recommended support and reinforcement for the arch and walls of underground excavations. Both tunnels and caverns are catered for, from roughly 3m to 60m span. Some 20 years ago the S(mr) support was updated by the same authors, replacing mesh reinforced shotcrete with fiber reinforced sprayed concrete or S(fr). The recommended PVC-sleeved (CT) bolts were more resistant to corrosion. The Q-system has always reflected single-shell B+S(fr) concepts of permanent support, as encompassed in the Norwegian Method of Tunnelling (NMT). During the 40 years of its use the Q-value has been shown to have empirical relationships to seismic velocity, deformation modulus, and tunnel or cavern deformation. It can also be used for helping to quantify the benefits of high-pressure pre-injection, and to estimate permeability. In addition, the Q-value has been extended for use in TBM prognosis, and a brief graphic review of this is given.

Key words: rock mass, classification, tunnels, drill-core, rock support, seismic velocity.

Introduction

Norway is a country with a small population, yet 3,500 km of hydro-power related tunneling, about 180 underground power houses, and some 1,500 km of road and rail tunnels. This has meant that *economic* tunnels, power-houses and also storage caverns, have always been needed, especially prior to the development of North Sea petroleum resources. The Q-system development in 1973 always reflected this, and single-shell tunnel support and reinforcement, meaning shotcrete and rock bolts as final support has been the norm, both before and since Q-system development. The first 200-plus case records from which Q was developed were 60% from Scandinavia, and already represented *fifty different rock types*, which is perhaps surprising for those who may focus on the quite frequent pre-Cambrian granites and gneisses. Norwegian and Swedish hydro power projects dominated these early

2014

FJELLSPRENGNINGSTEKNIKK
BERGMEKANIKK/GEOTEKNIKK 2014

FORTY YEARS WITH THE Q-SYSTEM IN NORWAY AND ABROAD
FØRTI ÅR MED Q-SYSTEMET I NORGE OG I UTLANDET

Nick Barton og Eystein Grimstad
Nick Barton & Associates og Geolog Eystein Grimstad

SUMMARY
This paper describes some of the lessons learned during four decades of application of the Q-system. It was first used in hydropower projects in Norway and in a water transfer project in Peru in 1974. Some years afterwards, application in Norwegian road tunnels followed. Personal application by the two main developers of the method in more than 30 countries, and widespread use by others in civil and mining engineering around the world, has provided rich experiences, stimulated numerous discussions and critique, and probably provided a simpler means of communication for geologists, for rock engineers, for mining engineers and also for lawyers in numerous court cases. In reality the Q-system is far more than six parameters, as the geology has to be understood before application can be optimal. A new combination of parameters, simply Jn/Jr, is found to have surprisingly useful properties for tunnels and mines.

SAMMENDRAG
Denne artikkelen beskriver en del av det man har lært, etter førti år med anvendelse av Q-systemet. Det ble først brukt i Norske vannkraftprosjekter, og i et vannoverføringsprosjekt i Peru i 1974. Noen år etter ble det anvendt i Norske veitunneler. Personlig anvendelse i flere enn 30 land av de som utviklet metoden, og utstrakt bruk av andre i alle typer undergrunns-anlegg, inkluder gruver over hele verden, har resultert i omfattende erfaringer, stimulert utallige diskusjoner og kritikk, og muligens ført til lettere kommunikasjon mellom geologer, bergingeniører, gruveingeniører og også mellom advokater i mange rettsaker. I realiteten har Q-systemet langt flere enn seks parametere, fordi geologien må bli forstått først for å oppnå optimal anvendelse. En ny kombinasjon av parameter Jn/Jr, viser seg å ha overraskende nyttige egenskaper for bruk i tunneler og gruver.

1. INTRODUCTION

Development of the Q-system during six hectic months in 1973 started as the result of a question from NVE (Statkraft) to NGI. The first author could not answer the question, so started developing a rock mass classification method, linked to support needs. RQD/Jn came first, with successively added parameters, and a lot of trial-and-error and empiricism using more than 200 case records. This finally enabled an answer to be given to the challenging question from Statkraft (NVE): Why all the different deformation magnitudes in Norwegian hydropower machine halls? So from the start not only rock mass quality, but excavation dimensions, purpose and rock reinforcement and support needs were integral parts of the method. The *number of joint sets*, which was suggested as an addition to RQD by Don Deere's Ph.D. student *Cecil (1970)*, has remained an important part of Q, but is remarkably absent from Bieniawski's RMR and is therefore also absent from GSI, which is the basis of the Hoek-Brown *non-empirical* failure criterion, used by so many optimistic continuum modellers. Since rock mass classification was '*not supposed to be possible, and can therefore never be developed*' (roughly the opinion expressed in NTH/NTNU Norwegian engineering

2014

Non-linear Shear Strength for Rock, Rock Joints, Rockfill and Interfaces

1. INTRODUCTION

Although intact rock is frequently represented by linear Mohr-Coulomb shear strength envelopes, the actual true behaviour, if taken over a wide range of confining stress, is extremely non-linear. Why is this important? Probably because the real stress across points of contact in both rockfill and rock joints, is approaching (or trying to exceed) the crushing strength, and if local confined stresses are equally high, strong non-linarity will be experienced. If we utilize the unconfined compression strength of the rockfill, or of the rock joint surfaces, suitably scaled-down due to size effects, we are part way towards a useful strength criterion. In fact, as we will see, we are 'one third' of the way, as the roughness (of particles and asperities) and a measure of the non-dilatant (residual) frictional strengths are also needed for what we will see are two very closely related strength criteria.

Figure 2 is a representation of the complete shear strength envelope for intact rock, as suggested by Barton, 1976 following a wide review of other researchers high-pressure triaxial data for intact rock, in particular the well-known and numerous studies of Byerlee and Mogi from the nineteen sixties. Both these researchers were concerned with the brittle-ductile transition. The present author suggested a simple 'definition' of the top (horizontal) part of the strength envelopes - in rock mechanics terminology called the 'critical state'. The complete shear strength envelopes of rock, and how much they deviate from linear Mohr-Coulomb has recently been quantified in a new criterion which may become known as the Singh-Singh criterion (Singh et al. 2011).

The horizontal part of the shear strength envelopes for a large group of silicate and carbonaceous rocks, suggested the following simple relation:

$$\sigma_{1\,max} = 3\,\sigma_{3\,critical} \qquad (1)$$

It will be noted that the uniaxial (unconfined) circle (#2) and the critical confining pressure circle (#4) are drawn as nearly tangent to one another. This potential simplicity has recently been confirmed by Singh et al. (2011), who found that the majority of rocks exhibited this tendency, i.e. $\sigma_{3\,critical} \approx \sigma_c$. This actually implies that if we reach a confining pressure (over the small area/volume in contact) approximately equal to UCS or σ_c, a local critical state may potentially be reached if (less confined) crushing has not already occurred. The maximum local rock strength will likely

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Q-SYSTEM - AN ILLUSTRATED GUIDE FOLLOWING FORTY YEARS IN TUNNELLING

N. Barton and E. Grimstad 2014

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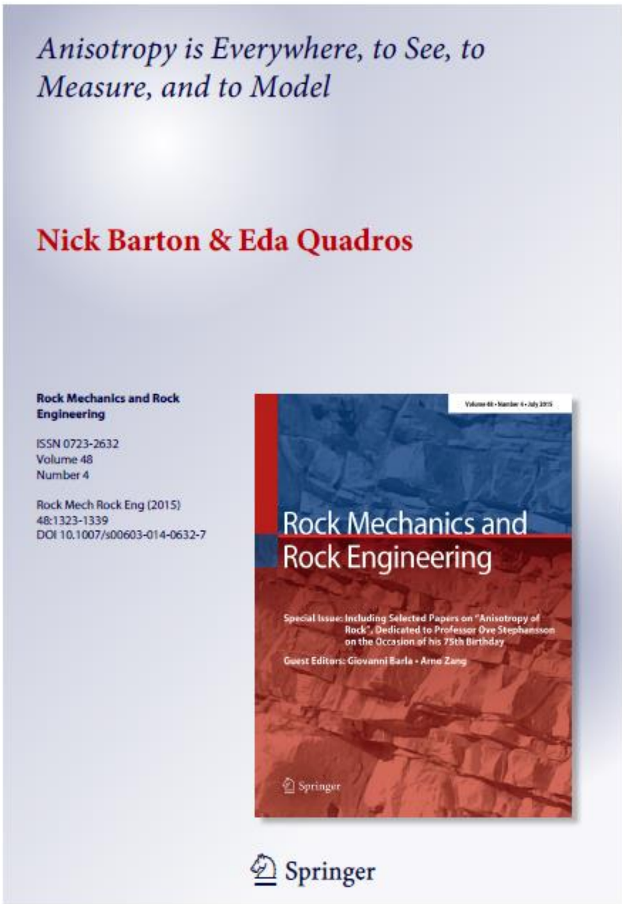

2015

EUROCK 2015 & 64th Geomechanics Colloquium. Schubert (ed.) © ÖGG

Introducing the Q-slope method and its intended use within civil and mining engineering projects

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ABSTRACT: The Q-system for characterizing rock exposures and drill-core, and for estimating single-shell support and reinforcement needs in tunnels, caverns and mine roadways has been widely used by engineering geologists and mining engineers. In the last ten years, a slightly modified Q-system called Q-slope was tested by the authors, for application in road cuttings, motorway cuttings, and benches in opencast mines. The purpose of Q-slope is to allow engineering geologists, rock engineers and mining engineers to rapidly assess the stability of excavated rock slopes in the field, and make optimal adjustments to slope angles as rock mass conditions become visible during construction of the road cuts or benches. Trials at several civil engineering and mining projects in Asia, Australia and Central America have shown that a simple correlation exists between Q-slope values and the long-term stable and unsupported slope angles. The new method includes Jr/Ja ratios for both sides of potential wedges, using relative orientation weightings.

1 INTRODUCTION

The Q-system (Barton et al. 1974 and Barton & Grimstad, 2014) for characterizing rock exposures, drill-core, and tunnels under construction, was developed from rock tunneling-related and rock cavern-related case records. Single-shell B+S(fr) tunnel support and reinforcement design assistance, and open stope design, utilizing Q' (the first four parameters) have been the principal focus of applications in civil and mining engineering. Correlations of Qc (Q normalized with UCS/100) with stress-dependent P-wave velocities and depth-dependent deformation moduli have also proved useful in site characterization and as input to numerical modelling. These approximate correlations remain with the new Q-slope value, which may also vary over six orders of magnitude, from approx. 0.001 to 1000. This large numerical range is an important reflection of the large variation of parameters such as deformation moduli and shear strength.

2015

Forty Years with the Q-system – Lessons and Developments

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ABSTRACT

This paper describes some of the lessons learned during four decades of application of the Q-system, both in Norway and abroad. Personal application in more than 30 countries, and widespread use by others in civil and mining engineering around the world, has provided rich experiences, stimulated numerous discussions and critique, and probably provided a simpler means of communication for geologists, for rock engineers, for mining engineers and also for lawyers in court cases. In reality the Q-system is far more than six parameters, as the geology has to be understood before application can be optimal. A new combination of parameters, simply Jn/Jr, is found to have surprisingly useful properties for tunnels and mines. The paper shows some comparisons between NMT (single-shell) and NATM (double-shell), emphasises the safety of RRS compared to lattice girders and S(fr) compared to S(mr). Latest Q-based dimensioning are given, plus some links between Q and useful parameters when modelling, specifically stress-dependent modulus, velocity, and tunnel or cavern deformation. During the 40 years of Q-application, the last 15 years has seen application of Q_{TBM} with added machine-rock interaction parameters. Recently Q_{slope} was developed, to guide the choice of maintenance-free slope angles.

1 INTRODUCTION

Development of the Q-system occurred during six hectic months in 1973 (not three years as stated in a recent NGI report), and it started specifically as the result of a question from NVE the Norwegian State Power Company (now Statkraft) to NGI. The challenging question: Why all the different deformation magnitudes in Norwegian hydropower caverns? The author could not adequately answer the question, so had to start developing a rock mass classification method, linked to support needs. Bieniawski (1973) developments were not yet known. So in the Q-system development, RQD/Jn came first, with successively added parameters, and a lot of trial-and-error and empiricism using more than 200 case records. Sixty percent of the first case records were from Norway and Sweden, and 50% concerned hydropower excavations, both caverns and tunnels. The development of Q finally enabled an answer to be provided to the question about powerhouse deformation magnitudes. Since some deep caverns and road tunnels were included, the onset of stress-fracturing was important, hence the sixth parameter SRF, involving the ratio rock uniaxial strength/stress. Grimstad and Barton (1993) added the experiences from 1,050 more case records, including extensive deep road tunneling in Norway, with eventually a depth as much as 1,400 m at Lærdal (and a world record length of 24.5 km). In this project there were three lorry-turning caverns of 30 m span at > 1,000 m depth (and 6 km intervals). These challenge even mining experiences.

From the start not only rock mass quality, but excavation dimensions, purpose and rock reinforcement and support needs were integral parts of the method. The *number of joint sets*, which was suggested as an addition to RQD by Don Deere's Ph.D. student Cecil (1970), has remained an important part of Q, but is remarkably absent from Bieniawski's RMR and is therefore also absent from GSI, which is the basis of the Hoek-Brown estimation of shear strength, used by so many optimistic continuum modellers. Since rock mass classification

2015

TBM PERFORMANCE, PROGNOSIS AND RISK CAUSED BY FAULTING

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Abstract.

World records for drill-and-blast tunnelling from Norwegian contractors, bear witness to numerous weeks of more than 100m, and an exceptional 5.8 km in 54 weeks, also from one face. Earlier hard-rock world records using high-powered TBM in Norway, but most frequently and more recently, the records with Robbins TBM through non-abrasive limestones in the USA, provide numbers in meters per day, per week, and per month, which are of course, even more remarkable. Unfortunately there are contrary and undesirable TBM records, which are occasionally recurring events so not records, which see TBM stopped for months or even years in fault zones, or permanently buried in mountains. The many orders of magnitude range of performance suggest the need for better investigations, better choice of TBM, and better facilities for improving the ground ahead of TBM, when probe-drilling indicates that this is essential. Control of water, and improved stand-up behaviour in significant weakness zones and faults may demand drainage, which can be unending, and pre-injection. Fortunately there are increasing signs that this is recognized by TBM manufacturers: more guide-holes for drilling pre-injection umbrellas are seen through front-shields nowadays. A little acknowledged fact is that when all hours are included, TBM will generally decelerate as tunnel length and time increases. This is usually seen after improved performance during the learning curve. Deceleration is also a general trend during world-record setting performances. This means that utilization U is equal to the ratio of actual advance rate and penetration rate, AR/PR, only for specified time intervals, because U is time-dependent. This is rarely quantified by designers, and is therefore a source of risk, by default. Another important item for correct prognosis is the recognition that *reduced* penetration rate PR can sometimes occur when thrust is *increased* by the TBM operator, due to exceptionally resistant rock mass formations. Each of the above, and PR sensitivity to a wide range of cutter forces, UCS and abrasiveness, are provided in the empirical Q_{TBM} method. This method explains variable progress in jointed rock, which is sometimes fast, and also quantifies the likely delays in untreated, or pre-injected, fault zones.

Keywords. TBM, penetration rate, advance rate, time-dependence, cutter force, Q.

1. Introduction

During the last 10-15 years, Norwegian contractors have led the world in the fastest drill-and-blast tunnelling rates, with 165m and even 176m in single 7x24 hour weeks. LNS and Veidekke have had consistent rates of more than 100m/week for several months in specific projects. At the Svea coalmine (one-face) access tunnel, in coal-measure rocks obviously requiring some bolting and shotcreting, LNS achieved 100m per week or more for 32 weeks, and used just 54 weeks to drill-and-blast 5.8 km. The tunnel had a 36 m² cross-section. This performance is actually better than many TBM project performances if one considers the whole year of tunneling, but does not appear so impressive in relation to TBM, if shorter time intervals are compared, as typically done with TBM.

2015

2015

Where is the Non-Linear Rock Mechanics in the Linear Geomechanics of Gas Shales?

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Abstract: Gas-shales usually produce from several kilometers depth, with fracing-stimulation, propping, and subsequent registration of desirable 'clouds' of microseismic activity in the surrounding rock mass. This is presumed to be the initiation of shearing on natural fractures. Further shear may be aseismic when cohesive bonds have already been broken. The shale matrix which must have at least some porosity, is weaker than is typical for basement rocks. The potential pore pressure reduction of tens of MPa during the early life of the fields, a life which is very short according to some operators, may therefore be a significant proportion of the strength of the matrix. Inevitable non-linear rock strength behaviour for the matrix should not then be ignored. Significant moduli are also required for good production. It seems unrealistic to utilize a linear Mohr-Coulomb strength criterion as so frequently seen in oil industry geomechanics. The joints or natural fractures in the shales, which are presumed to be so important for production towards the inner propped region closer to the horizontal well section, will have producing fracture sets with different roughness and aperture. How many of them are planar enough to follow the linear Mohr-Coulomb behaviour always shown, where only a traditional 'Byerlee friction coefficient' is used? Where is the expected non-linear rock mechanics in the multi-discipline teams doing geomechanics? It has not yet been seen in numerous workshops, nor in oil industry courses on both sides of the Atlantic.

Key words: Shale gas, non-linear, shear strength, fractures, dilation.

1 Introduction

It has been known for many decades that the mechanical behaviour of fractures (the naturally occurring joint sets in a rock mass) is in general non-linear. The non-linearity is largely due to fracture roughness. (Barton, 1973, 2014a). In the case of intact rock, non-linear shear strength envelopes would be the result of large changes of effective stress caused by gas production. Non-linearity with respect to the jointing applies to the favourable shear strength-dilation-permeability coupling, which starts pre-peak, before Byerlee (friction coefficient based) strength is reached. The latter simplification for frictional strength from Stanford University, adopted by the geomechanics teams of many oil companies, misses the finess of pre-peak and post-peak dilatant shear strength. Non-linearity applies to the dilation accompanying shear, and especially applies to the less desirable stress-closure-permeability coupling of a stress-sensitive reservoir. Non-linear coupled-process (MH) modelling, partly based on the joint- or fracture-roughness coefficient (JRC), has been applied in rock mechanics (and reservoir compaction modelling) for more than 30 years, in discrete fracture codes like UDEC-BB, usually coupled with a joint wall strength parameter JCS.

The joint or fracture roughness JRC, also applies to the conversion from the hydraulically interpreted theoretical smooth-wall apertures (e) to the larger and non-planar-non-smooth-wall physical apertures (E), through which the gas (or oil) actually has to flow to the wells, in the case of gas shales or naturally fractured reservoirs. Simple index tests for acquiring JRC and JCS for the different fracture sets can be applied on fractures recovered in occasional and inevitably expensive core, and can also be estimated when mapping (or drone-photographing) fractured pavement analogues.

The conversion of a highly impermeable medium like shale into numerous gas-producing 'pay zones', using principal-stress steered, stress-and-structure oriented hydraulic fracturing, is a remarkable achievement. So remarkable

2015

XV COLOMBIAN GEOTECHNICAL CONGRESS & II INTERNATIONAL SPECIALIZED CONFERENCE OF SOFT ROCKS.
CARTAGENA, COLOMBIA - OCTOBER 5th -7th 2016

Cavern and Tunnel Collapses due to Adverse Structural Geology

Colapsos en Túneles y Cavernas debido a Geología Estructural Adversa

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ABSTRACT: *Some remarkable cavern and tunnel failures are described in this keynote paper. As an independent consultant one occasionally has the privilege of observing some dramatic effects of adverse and usually completely unexpected structural geology, causing tunnel or cavern failures that can be dramatic. Very regrettably, the collapses are sometimes fatal for some unsuspecting tunnel or cavern workers. There is often an adverse design for the special circumstances. Failure is most frequent during construction, with only the temporary support to resist the unexpected challenges of adverse structural geology. In fact the three most serious cases shown have lattice girders or steel arches as one of the components of the temporary support. It is usually a surprise to read that this hardest of materials provides the softest of deformation resistance, because of the difficulty of making contact with the uneven and by now blasted rock surface, once the soil and saprolite has been passed in the early tens of meters of a typical tunnel. These partly free-standing girders or arches, and their footings, deform too much before fully resisting radial deformation, thereby potentially reducing the shear strength of the rock mass, which may not be bolted when there is deep weathering. Such measures (lattice girders and steel sets) should never be part of the Q-system, which is essentially for excavations in rock masses, even if of poor quality with clay and fault zones. A bolted, and intimately supporting, steel-and-fiber reinforced S(fr) arch is needed to reduce the risk of collapse. This can function well even when there is an excessively rough perimeter due to over-break.*

RESUMEN:

En esta charla especial se describen algunas colapsos notables de túneles y cavernas. Un consultor independiente de vez en cuando tiene el privilegio de observar algunos efectos extraordinarios resultantes de una geología estructural adversa y completamente inesperada, capaz de causar colapsos substanciales en túneles y cavernas. Muy lamentablemente esas colapsos son algunas veces fatales para algunos trabajadores no conscientes de la presencia del peligro. Hay con frecuencia un diseño adverso para una circunstancia especial. Los colapsos ocurren a menudo durante la construcción, cuando se emplea solamente un soporte temporal para resistir los retos inesperados de una geología estructural adversa. En realidad, los tres casos más serios presentados tienen vigas de celosía o arcos de acero como uno de los componentes del soporte temporal. Por lo general es una sorpresa saber que estos materiales tan duros proporcionan la peor resistencia a la deformación, por causa de la dificultad de tener contacto adecuado con la superficie irregular de la roca dejada por las voladuras y una vez se ha traspasado el suelo y el saprolito en las primeras decenas de metros en un túnel típico. Estas vigas de celosía o arcos y sus zapatas, actuando parcialmente libres, se deforman demasiado antes de que puedan resistir plenamenete la deformación radial, reduciendo así potencialmente la resistencia al cizallamiento del macizo rocoso, algunas veces no apto para el empleo de pernos cuando hay una alteración profunda. Tales complementos (vigas de celosía y arcos de acero) nunca debem ser parte del sistema Q, que debe ser esencialmente para excavaciones en macizos rocosos, aunque sean de mala calidad y con zonas de arcillas y de fallas. El uso de hormigón proyectado con pernos y reforzado con malla y fibra de acero S(fr) es necesario para reducir el riesgo de colapso. Esto tipo de suporte puede funcionar bien, incluso cuando hay una superficie muy rugosa por causa de exceso de excavación o de voladuras.

2016

ARMA 16-252

Non-linear shear strength descriptions are still needed in petroleum geomechanics, despite 50 years of linearity

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ABSTRACT: Despite utilizing multi-discipline teams in petroleum and service companies, and despite the remarkable abilities of these same companies to produce petroleum in many forms and in highly adverse environments, there appears to be a misalignment between the *geomechanics used*, and *rock mechanics*, the latter apparently *not used*, judging by workshop presentations. Is it possible that this is because *geomechanics* specialists are not aware of long-available non-linear description of shear strength, for both the fractures and matrix, in NFR and gas shales? Description of the non-linearity has been a goal of many in rock mechanics for the last 50 years, with a well-known start by Patton, 1966 reporting in the first ISRM congress held in Lisbon. His bi-linear strength envelope was an immediate improvement on Mohr-Coulomb linearity, and set the scene for others to improve upon this bi-linear description. The 1973 non-linear JRC- and JCS-based criterion of the present author was considered too complex by Byerlee, 1978, who favoured use of a friction coefficient. Shear strength represented in *geomechanics* remains linear, so is often unrealistic.

1. INTRODUCTION

Due to remarkable educational compartmentalization, there is widespread use of linear shear strength assumptions in the *petroleum geomechanics* taught in university courses. This is further applied by many oil companies and oil service companies, actually on both sides of the Atlantic, and indeed even in the Middle East. This is undoubtedly due in part to the application of Byerlee's well-known friction coefficients by Zoback and co-workers at the University of Stanford during the last several decades. The linear frictional strength assumptions are applied to signify probable critically stressed fractures, first in deep wells, later in naturally fractured petroleum reservoirs, and in the more recently exploited unconventional gas shales. Linear shear strength assumptions are also applied to the intact rock, which is usually described by simplified linear Mohr-Coulomb, despite many tens of MPa change of effective stress during the sometimes short lives of the reservoirs.

The commonly used Byerlee-type friction coefficient to signify that fractures may be critically stressed, with values quoted typically from 0.6 to 0.85, actually gives no insight into pre-peak phenomena, which include the beginnings of roughness and dilation mobilization, giving potential permeability enhancement in the case of gas shales and critically stressed fracture sets in NFR.

The more serious linearization of matrix strength, which is actually something with strong curvature when matrix porosity is present, is even more surprising, in view of the 30 to 60 MPa increase in effective stress which is often experienced during production. Linearization is also not consistent with the strong non-linearity demonstrated by tectonophysicists such as Mogi (and indeed Byerlee) back in the 1960's when applying high confining stresses to small intact triaxial rock samples.

Strength culminating in a maximum strength identified as a 'critical state' ($\sigma_{1\ max} = 3\sigma_{3\ crit}$) by Barton, 1976, led Singh et al. 2011 to demonstrate that the *horizontal critical state* of the strength envelope for a given rock matrix is reached when the confining pressure reaches the approximate *level of uniaxial compressive strength*. This simplification works well for the majority of rocks.

In contrast, shear strength suggested by linearity 'goes on forever'. Obviously it cannot, and the *highly stressed* 'island asperities' of even relatively planar shearing fractures, if with insufficient static deformation moduli as in the case of some gas shales, may be the reason for some very short production lives. Such were indicated in interviews with the industry, in Ghassemi and Suarez-Rivera, 2012 who mostly studied proppant-sustained hydraulic fractures at Schlumberger/TerraTek facilities.

2016

Application of the Q-slope method to highly weathered and saprolitic rocks in Far North Queensland

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ABSTRACT: The Q-slope method was developed to allow engineering geologists and geotechnical engineers to assess the stability of excavated rock slopes in the field and make potential adjustments to slope angles as rock mass conditions become visible during construction. Q-slope was developed over the last decade by modifying the Q-system for characterizing rock exposures and drill-core, and estimating single-shell support and reinforcement needs in tunnels, caverns and mining roadways. Q-slope features a new method of Jr/Ja ratios for both sides of potential wedges, using relative orientation weightings and also considers long-term exposure to various climatic and environmental conditions (e.g. tropical storms, ice-wedging effects). Q-slope is intended for reinforcement-free road or rail cuttings or individual benches in open cast mines.

Assessing slope stability in highly weathered rocks and saprolites (in-situ, soft, friable, weathered rock that retains the original rock's structure and fabric but with a lower bulk density) is considered complex since failure mechanisms often involve a combination of shearing and rotational sliding through a weak rock mass as well as sliding on relic geologic structures. The Q-slope method was applied to several highly weathered and saprolitic slopes in Far North Queensland and has shown that a simple correlation exists between Q-slope values and long-term stable and unsupported slope angles.

1 INTRODUCTION

1.1 *Original Q-system*

The original Q-system for characterizing rock exposures, drill core and tunnels under construction was developed from rock tunneling related and rock cavern related case records and has been used by engineers across the world for over 40 years (Barton et al. 1974 and Barton & Grimstad, 2014). Single-shell B+S(fr) tunnel support and reinforcement design assistance, and open stope design, utilizing Q' (the first four parameters) have also been the principal focus of application in civil and mining engineering.

1.2 *Q-slope overview*

The Q-slope method (Barton & Bar, 2015) is intended for use in reinforcement-free site access road cuts, road or rail cuttings or individual benches in open cast mines. It is not intended for assessing the stability of large slopes developed by several excavation stages over significant periods of time, such as inter-ramp or overall slopes in open cast mines.

Q-slope was developed from case records in six countries, spanning 17 rock types (igneous, sedimentary and metamorphic) and some saprolites for slope heights ranging from 5m to 30m.

Shear strength input is similar to the original Q-system, but more critical, as wedges are unconfined, and dilation is less important than around tunnels as there is usually no increase in normal stress or stiffness when shearing initiates. Filled discontinuities follow the same 'contact' scheme as before: a) rock-to-rock contact, b) rock-to-rock contact after shear displacement, c) no rock-to-rock contact due to thick clay fillings. Q-slope utilizes the same six parameters RQD, Jn, Jr, Ja, Jw and SRF. However, the frictional resistance pair Jr and Ja can apply, when needed, to the individual sides of potentially unstable wedges using simple orientation factors. The term Jw, which is now termed Jwice, takes into account an appropriately wider range of environmental conditions pertinent to rock slopes, which are exposed to the elements indefinitely. These conditions include the extremes of erosive intense rainfall, ice wedging, as may seasonally occur at opposite ends of the rock-type and regional spectrum. There are also slope-relevant SRF categories. For Q-system users, the formula for estimating Q-slope is two-thirds familiar (Barton & Bar, 2015):

$$Q_{slope} = \frac{RQD}{J_n} x \left(\frac{J_r}{J_a}\right)_0 x \frac{J_{wice}}{SRF_{slope}} \qquad (1)$$

2016

Fast or slow progress with TBM in ideal or faulted conditions

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ABSTRACT: Tunnel lengths of 5km, perhaps 10km, and even a world-record equaling 15km can be driven *in one year* by TBM. In contrast, the very best drill-and-blast record so far is 5.8km in 54 weeks, and usually it is closer to 50m in a week, rather than the 100m per week achieved by just a handful of contractors worldwide, in some portions of their drill-and-blast projects. A factor with TBM tunneling that does not seem to have been widely acknowledged, or used in planning is that there is a general *deceleration* as the tunnel gets longer. This is seen in open-gripper case records and in efficient double-shield, in each case following speed-up in the 'learning curve' period. Perhaps surprisingly, the same trends of *deceleration* are seen in the current world record TBM performances, with diameters all the way from 3m and beyond 12m. The *deceleration* from PR to best day, week, month, and 3-months is seen when total time is used, and advance rate is expressed in m/hr. It is found that the steepest deceleration occurs when the rock mass quality Q is low. This is confirmed by specific case records from Turkey. The time delay is quantifiable. Sometimes hybrid TBM *and* D&B is best.

1 INTRODUCTION

The authors have experiences from both fast and slow TBM excavations, and cases where drill-and-blast 'rescue' was required, although not originally planned. A longer tunnel is automatically a 'larger sample' of the rock mass, with more extremes likely to be encountered. Hard massive rock, faulted clay-bearing rock with water pressure, and high-mountain cover are three extremes that individually or collectively can cause serious delays. The longer, deeper tunnel is also unlikely to have been investigated as thoroughly, so surprises have almost to be expected.

2 SOME EXAMPLES FROM TURKEY

The geology of Turkey is complex with weak rocks and fault zones that tremendously decrease the performance of TBM. Strong deceleration and delays are seen in specific zones which often are related to low or extremely low Q-values, representing the inverse of tunnelling quality. Conventional (drill-and-blast) tunnelling and mechanical excavation methods are sometimes used together in the same project, as the tunnel is getting longer. The three following examples show clearly how TBM performance and machine utilization time are affected by low Q values. The benefit of sometimes using hybrid (drill and blast and TBM excavation) tunnelling methods are demonstrated by practical experiences.

2.1 Uluabat Hydropower Tunnel

The excavation of the headrace tunnel started in June 2006, with a 5.05 m diameter EPB-TBM. The tunnel was eventually finished in March 2010. During the tunnel excavation, the TBM jammed 18 times in different places, due to the highly squeezing characteristics of the ground. Rescue galleries were opened next to the TBM to free the shield, and a total of 192 days were spent on these operations (Bilgin & Algan 2012). One of the galleries opened to rescue the TBM is seen in Figure 1. The tunnel route from chainages 11+465km to 7+750 km (3.7 km) and from 6+000 km to 1+792km (4.2 km), consisted of the Karakaya formation of Triassic-aged meta-detritics such as fine grained meta-claystone, meta-sandstone, schists etc. The tunnel route between chainage 7+750 and 6+000 km (1.75 km) consisted of the Akcakoyun formation of Jurasic-aged limestone with crystallized calcite fillings. Table 1 lists rescues galleries with chainage, Q values and a brief description of the zones where the TBM was trapped. An average daily advance rate of 8.6 m/day was achieved, including all stoppages such as TBM standstills and hand mining. The best daily and weekly advance rates were found to be 28.8 m and 198.4 m, respectively. The best monthly advance rate was 583.2 m in February 2007, as shown in Figure 2. The breakdown

2016

Journal of Rock Mechanics and Geotechnical Engineering 9 (2017) 210–225

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

Risk of shear failure and extensional failure around over-stressed excavations in brittle rock

CrossMark

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ARTICLE INFO

Article history:
Received 13 June 2016
Received in revised form 10 November 2016
Accepted 14 November 2016
Available online 30 November 2016

Keywords:
Fracture propagation
Tension
Shear
Poisson's ratio
Break-out
Rock burst
Deep tunnels
FRACOD model

ABSTRACT

The authors investigate the failure modes surrounding over-stressed tunnels in rock. Three lines of investigation are employed: failure in over-stressed three-dimensional (3D) models of tunnels bored under 3D stress, failure modes in two-dimensional (2D) numerical simulations of 1000 m and 2000 m deep tunnels using FRACOD, both in intact rock and in rock masses with one or two joint sets, and finally, observations in TBM (tunnel boring machine) tunnels in hard and medium hard massive rocks. The reason for 'stress-induced' failure to initiate, when the assumed maximum tangential stress is approximately $(0.4\text{–}0.5)\sigma_c$ (UCS, uniaxial compressive strength) in massive rock, is now known to be due to exceedance of a critical extensional strain which is generated by a Poisson's ratio effect. However, because similar 'stress/strength' failure limits are found in mining, nuclear waste research excavations, and deep road tunnels in Norway, one is easily misled into thinking of compressive stress induced failure. Because of this, the empirical SRF (stress reduction factor in the Q-system) is set to accelerate as the estimated ratio $\sigma_{\theta max}/\sigma_c >> 0.4$. In mining, similar 'stress/strength' ratios are used to suggest depth of break-out. The reality behind the fracture initiation stress/strength ratio of '0.4' is actually because of combinations of familiar tensile and compressive strength ratios (such as 10) with Poisson's ratio (say 0.25). We exceed the extensional strain limits and start to see acoustic emission (AE) when tangential stress $\sigma_\theta \approx 0.4\sigma_c$, due to simple arithmetic. The combination of 2D theoretical FRACOD models and actual tunnelling suggests frequent initiation of failure by 'stable' extensional strain fracturing, but propagation in 'unstable' and therefore dynamic shearing. In the case of very deep tunnels (and 3D physical simulations), compressive stresses may be too high for extensional strain fracturing, and shearing will dominate, both ahead of the face and following the face. When shallower, the concept of 'extensional strain initiation but propagation' in shear is suggested. The various failure modes are richly illustrated, and the inability of conventional continuum modelling is emphasized, unless cohesion weakening and friction mobilization at different strain levels are used to reach a pseudo state of yield, but still considering a continuum.

1. Introduction

We will start by illustrating two very different failure modes, both of them being 'physical realities' but from very different environments. The first is from petroleum well-bore simulations in sandstones. With change of scale, a small deep tunnel in a weak but brittle rock can be envisaged. Failure is dominated by (log-spiral) shearing (Fig. 1). The second is a real case involving highly-stressed granite in an underground research laboratory (URL): the URL in Canada. Crack initiation is induced by tensile/extensional fracturing, but there is shearing, buckling, and a final characteristic notch (see Fig. 2). In the following investigations, both tensile (or extensional strain) initiation and progression in shear have their important roles to play. Tensile initiation may consist of critical strain-initiated extensional fracturing, which can explain several puzzling phenomena such as tensile fracturing in entirely compressive stress fields (e.g. Fairhurst and Cook, 1966).

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

http://dx.doi.org/10.1016/j.jrmge.2016.11.004
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2017

Rock Mech Rock Eng
DOI 10.1007/s00603-017-1305-0

CrossMark

ORIGINAL PAPER

The *Q*-Slope Method for Rock Slope Engineering

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Received: 8 July 2017 / Accepted: 25 August 2017

Abstract *Q*-slope is an empirical rock slope engineering method for assessing the stability of excavated rock slopes in the field. Intended for use in reinforcement-free road or railway cuttings or in opencast mines, *Q*-slope allows geotechnical engineers to make potential adjustments to slope angles as rock mass conditions become apparent during construction. Through case studies across Asia, Australia, Central America, and Europe, a simple correlation between *Q*-slope and long-term stable slopes was established. *Q*-slope is designed such that it suggests stable, maintenance-free bench-face slope angles of, for instance, 40°–45°, 60°–65°, and 80°–85° with respective *Q*-slope values of approximately 0.1, 1.0, and 10. *Q*-slope was developed by supplementing the Q-system which has been extensively used for characterizing rock exposures, drill-core, and tunnels under construction for the last 40 years. The *Q'* parameters (RQD, J_n, J_a, and J_r) remain unchanged in *Q*-slope. However, a new method for applying J_r/J_a ratios to both sides of potential wedges is used, with relative orientation weightings for each side. The term J_w, which is now termed J_{wice}, takes into account long-term exposure to various climatic and environmental conditions such as intense erosive rainfall and ice-wedging effects. Slope-relevant SRF categories for slope surface conditions, stress-strength ratios, and major discontinuities such as faults, weakness zones, or joint swarms have also been incorporated. This paper discusses the applicability of the *Q*-slope method to slopes ranging from less than 5 m to more than 250 m in height in both civil and mining engineering projects.

Keywords *Q*-slope · Rock slope engineering · Slope stability · Rock mass classification · Empirical method

List of symbols

RQD	Rock quality designation
J_n	Joint sets number
J_r	Joint roughness number
J_a	Joint alteration number
J_{wice}	Environmental and geological condition number
SRF_{slope}	Three strength reduction factors a, b, and c
SRF_a	Physical condition number
SRF_b	Stress and strength number
SRF_c	Major discontinuity number
O-factor	Orientation factor for the ratio J_r/J_a

1 Introduction

In both civil engineering and mining projects, it is practically impossible to assess the stability of rock slope cuttings and benches in real time, using analytical approaches such as kinematics, limit equilibrium, or FEM/DEM modeling. Excavation is usually too fast for this. The same limitation usually applies to tunneling, despite numerical modeler's wishes to the contrary. However, rock caverns of larger span are sufficiently 'stationary' for thorough and more necessary analysis, and the same applies to higher

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2017

Lessons learned from large failures: multiple causes include adverse design, geology, and support

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Large tunnel and rock cavern spans that have failed for 'geological' reasons, or because of design errors, are the main focus of this presentation. The effect of adverse and sometimes unexplored geology will be illustrated. We will need to recognize that pre-investigations might miss some important detail, despite an exceptional frequency of core drilling. Usually, this is of minor importance and does not mean that failure will occur. Being wise after an event, which is always much easier, one might generally question why cross-hole seismics is not performed more frequently to obtain between-borehole information. Prior to construction, the need for this in a particular, but at that time unknown location, cannot of course be foreseen.

It is apparent that really large failures seldom result from just one or two oversights, but are caused by a multitude of adverse factors working together. Small failures might be caused by designer or contractor short-cuts, or more likely by failure to log 'today's' conditions and react with more support. The really big failures are due to faulty design, and therefore inappropriate support, but sometimes just the multiple effects of some exceptionally adverse and unanticipated 'geology', aided by a remarkably adverse location, preventing the natural and needed arching. All or many of the following represent a rich assortment of possible reasons for failure underground: oversimplified design method assumptions that are erroneous, erroneous details of sub-surface topography, ignored details of surface topography, anisotropic low-strength jointing, the presence of unexpected discontinuities with adverse properties, and finally the use of deformable and weak temporary support like lattice girders.

It seems that a really massive failure may involve about five or more adverse factors. With sufficient factors involved, fatalities may be a regrettable consequence, besides the huge economic losses. Surprisingly, whether or not a tunnel or cavern reaches this point of ultimate collapse might not depend on real-time interpretation of the instrument readings. This is because reaction to a new and exceptional rate of deformation may be too late when too many unknown adverse factors are already combined. A state of 'guaranteed failure' may be reached *despite* instrumentation warnings.

The cavern and tunnel collapses that are generalized in the above paragraphs are specifically from the world of city metro, motorway ring-road tunnels, and hydropower caverns. They will be added to by reference to the largest open-pit slope failure to date, which showed the mechanism of progressive failure and an adverse pit shape, resulting in an absence of large-scale tangential stress. Failure was located in the 'unstressed' nose, and comprised 150 Mt of waste rock and ore. The tunnel and cavern failures include two 140 m tunnel collapses, two 35 000 m^3 progressive cavern collapses, and a 15 000 m^3 total collapse. A relative absence of sufficient tangential stress (arching stress) can be blamed in each case, and if the shear strength or designed support are in question, massive failure may result.

2017

BARTON-BANDIS CRITERION

Synopsis

The Barton-Bandis criterion is a series of rock-joint behaviour routines which, briefly stated, allow the *shear strength and normal stiffness* of rock joints to be estimated, graphed, and numerically modelled for instance in the computer code UDEC-BB. Coupled behaviour, with deformation and changes in conductivity are also included. A key aspect of the criterion is the quantitative characterization of the joint, joints, or joint sets in question, in order to provide three simple items of input data. These concern the joint-surface roughness (*JRC: joint roughness coefficient*), the joint-wall compressive strength (*JCS: joint compressive strength*), and an empirically-derived estimate of the *residual friction angle* (φ_r). These three parameters have typical ranges of values from: JRC = 0 to 20 (smooth-planar to very rough-undulating), JCS = 10 to 200 MPa (weak-weathered to strong, unweathered) and φ_r =20° to 35° (strongly-weathered to fresh-unweathered). Each of these parameters can be obtained from simple, inexpensive index tests, or can be estimated by those with experience.

The three parameters JRC, JCS and φ_r form the basis of the non-linear peak shear-strength equation of Barton, 1973 and Barton & Choubey, 1977. This is a *curved shear strength envelope* without cohesion (c). It will be contrasted to the linear Mohr-Coulomb 'c and φ' (with apparent cohesion) criterion later. To be strictly correct the original Barton equation utilised the basic friction angle φ_b of flat, unweathered rock surfaces (in 1973), while φ_r was substituted for φ_b following 130 direct shear tests on fresh and partly weathered rock joints (in 1977).

As well as peak and residual shear strength envelopes for laboratory-scale joint samples, Barton's cooperation with Bandis (from 1978) resulted in corrections (reductions) of JRC and JCS to allow for the *scale effect* and reduced strength as rock-block size is increased. The laboratory-scale parameters, written as JRC_0 and JCS_0 for laboratory-size samples of length L_0 (typically 50mm to 250mm), are written as JRC_n and JCS_n for *in situ* rock block lengths of L_n (typically 250mm to 2500mm, or even larger in massive rock).

Bandis is also responsible for utilizing JRC and JCS in empirical equations to describe *normal closure and normal stiffness*. Normal stiffness (Kn) has units of MPa/mm, and might range from 20 to 200 MPa/mm. The Barton-Bandis (B-B) criterion includes the related modelling of *physical joint aperture E* (typically varying from 1mm down to 50μm, or 0.05mm) as a result of the normal loading (or unloading). B-B also includes the theoretically equivalent smooth-wall *hydraulic aperture e*, (typically 1mm down to 5μm, or 0.005mm). Usually E > e, and the two are empirically inter-related, using the small-scale joint roughness JRC_0.

Finally the stiffness in the direction of shearing has also to be addressed. It is called *peak shear stiffness (Ks)*. It has typical values of 0.1 MPa to 10 MPa/mm, i.e. 1/10th to 1/100th of normal stiffness. The concept of *mobilized roughness* ($JRC_{mobilized}$) developed by Barton, 1982, allows both the peak shear-stiffness and the *peak dilation angle* (the effective aperture increase with shearing) to be calculated. The full suite of Barton-Bandis joint behaviour figures includes *shear stress-displacement-dilation, stress-closure, and the change of estimated conductivity* in each case. Examples of these will be given, following diagrams illustrating joint index testing.

2017

Innov. Infrastruct. Solut. (2017) 2:52
DOI 10.1007/s41062-017-0071-x

CrossMark

TECHNICAL PAPER

Minimizing the use of concrete in tunnels and caverns: comparing NATM and NMT

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Received: 4 May 2017 / Accepted: 2 June 2017

Abstract For many decades, a tunnelling method has been in use which effectively minimizes the use of concrete, which should be one of the goals in our CO_2-producing planet. We call the method NMT (Norwegian Method of Tunnelling) and emphasize its 'single-shell' characteristics, to distinguish it clearly from double-shell NATM (the so-called New Austrian Tunnelling Method), which is recommended to have (ASG, NATM: the Austrian practice of conventional tunnelling 2010): shotcrete, mesh, lattice girders, rock bolts (if in-rock), drainage fleece, membrane, and the final load bearing and often steel-reinforced concrete lining, including the invert when in poor rock conditions. This tunnelling method is inevitably several times more expensive, uses many times the volume of concrete, takes longer to build, and requires at least a ten times larger labour force than single-shell NMT. The single-shell tunnels for road or rail or hydropower or water transfer, or for large caverns for storage of oil or food, or for hydropower machine and transformer halls, can be made stable by judicious application of a well-used (>2000 case record based) the so-called *Q*-system of rock mass quality estimation. The latter encompasses a rock mass quality scale from 0.001 (equivalent to a serious fault zone, where we also may need a local concrete lining) to 1000 (equivalent to massive unjointed rock) where careful blasting will remove the need even for shotcrete. In general, rock masses where we need tunnels or caverns will lie closer to 'mid-range' (i.e. closer to $Q = 1$ which is described as 'poor quality'). Here we would need combinations of corrosion-protected rock bolts and high quality fibre-reinforced shotcrete, with stainless steel or polypropylene fibres. We may also need systematic high-pressure pre-injection of micro-cement and micro-silica, which may add 20% to the (low) starting cost of the NMT excavation. Written as $B + S(fr)$ in short-hand, NMT has rock bolt c/c spacing in metres and shotcrete thickness in centimetres, as specified by the range of *Q* values and excavation dimensions. The details are also affected by the planned use. For instance, at our record-breaking Olympic cavern of 60 m span (for housing 5400 spectators or later concert goers), $B = 2.5$ m c/c + $S(fr)$ 10 cm were (and remain 25 years later) the stabilizing and permanent measures of support and reinforcement. Deformation monitoring and distinct element (jointed rock) numerical verification showed 7–8 mm of maximum deformation in the arch. The moderate *Q* value range of quality of 2–30 (poor/fair/good) and RQD = 60–90 indicated a well-jointed gneiss, which had only moderate UCS = 90 MPa compressive strength.

keywords Tunnels · NMT · NATM · Overbreak · Shotcrete · Concrete · Collapse

This paper was selected from GeoMEast 2017—Sustainable Civil Infrastructures: Innovative Infrastructure Geotechnology.

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Introduction

In view of the above abstracted summary, and as a valid challenge, it would be interesting to know how mid-European, specifically Austrian NATM (double-shell) designers would have tackled the design of such a large cavern, and what thickness of concrete would have been

2017

ARMA 17-686

ARMA american rock mechanics association

Extension failure mechanisms explain failure initiation in deep tunnels *and* critical heights of cliff faces and near-vertical mountain walls

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ABSTRACT: Brittle rock can fail due to extension strain over-coming the tensile limit, even when all stresses are compressive. Two related topics can thereby be addressed. The first is extension strain-induced failure in deep tunnels and mines, with fracture initiation in tension but subsequent propagation in shear. The familiar ratio of maximum tangential stress σ_θ / UCS of 0.4 ±0.1 signifying rock fracture initiation, and the initiation of acoustic emission, are each explained by the ratio of tensile and compressive strength and by Poisson's ratio, using a classic elastic equation and simple arithmetic. The maximum critical tangential stress σ_θ can actually be expressed as the ratio of tensile strength and Poisson's ratio σ_t/ν, hence the ratio 0.4±0.1. The second related topic is the *limited* heights of mountain walls, seen from the perspective of rock climbing. Limited in this case is a relative term: 1,000m to 1,350m of almost vertical meters challenge all rock climbers. This 'big-wall' range of heights is not exceeded anywhere in the world. Although strong granites are often involved, large-scale tensile strengths are apparently no larger than about 5 to 10MPa, due to weakening from temperature-cycling. At the other end of the spectrum, cliff houses in Cappadocia tuffs in Turkey may become exposed at intervals, due to extension failure of 15-20m high cliffs, with *in situ* tensile strengths of only 0.1MPa. Mountain heights limited to 8-9km can be explained by non-linear critical state rock mechanics: maximum shear strength is the limitation.

1. INTRODUCTION TO TUNNEL FRACTURES

Why do tunnels in massive rock start to exhibit fracture initiation when the ratio of the assumed maximum tangential stress σ_θ reaches approximately 0.4 x UCS? Why does acoustic emission in a laboratory compression test initiate at a similar ratio of principal stress and uniaxial compressive strength? Why do vertical cliffs in the ultra-weak Cappadocia tuffs in central Turkey tend to fail and expose underground dwellings or churches at intervals of a few decades or centuries, when their heights are even as little as 10 to 20 m? Why do the highest vertical mountain faces in the world of rock climbing range from 'no more' than 1,200 to 1,300m in height? Why did the first TBM tunnel in the world (1880) fail at its haunches when it curved beneath only a 70m high chalk cliff next to the Channel Tunnel, which was built 110 years later?

In the world of tunnelling we have come to expect *increased depth of break-out* when σ_θ /UCS > 0.4 (± 0.1), following a Canadian initiative (Martin et al. 1998). This is shown in Figure 1. Since 1993 we have used a rapidly accelerating value of the stress reduction factor SRF in the widely used single-shell Q-system tunnel-support recommendations, when σ_θ /UCS exceeds 0.4-0.5, as shown in Table 1. (Barton et al. 1974 showed SRF deliberately accelerated when the simpler ratio of σ_c/σ_1 reduced to below 5). Why do we need to reach a peak tangential stress of *only* 0.3-0.5 x UCS? Is this due to a scale effect, or due to an incorrect assumption?

Table 1. The 'accelerating' value of SRF when the ratio σ_θ/UCS ≥ 0.4, from the Grimstad and Barton, 1993 analysis of deep (600 to 1,400 m) road tunnels in Norway. Pre-1974 cases had suggested these limits: σ_c/σ_1 <5 or σ_t/σ_1< 0.33 required higher SRF and more robust tunnel support, reduced bolt c/c.

b) Competent rock, rock stress problems		σ_c/σ_1	σ_θ/σ_c	SRF
H	Low stress, near surface, open joints.	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition.	200-10	0.01-0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > 1 hour in **massive** rock.	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in **massive** rock.	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in **massive** rock.	< 2	> 1	200-400

2017

CHARACTERISATION AND MODELLING OF THE SHEAR STRENGTH, STIFFNESS AND HYDRAULIC BEHAVIOUR OF ROCK JOINTS FOR ENGINEERING PURPOSES

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1 INTRODUCTION

The term 'characterisation' will be used to describe methods of collection and interpretation of the physical attributes of the joints and other discontinuities, in other words those which control their mechanical and hydraulic properties, and the behaviour of jointed rock as an engineering medium. Rock discontinuities vary widely in terms of their origin (joints, bedding, foliation, faults/shears, etc.) and associated physical characteristics. They can be very undulating, rough or extremely planar and smooth, tightly interlocked or open, filled with soft, soil-type inclusions or healed with hard materials. Therefore, when loaded in compression or shear, they exhibit large differences in the normal and shear deformability and strength, resulting in surface separation and therefore permeability. Such variability calls for innovative, objective and practical methods of joint characterisation for engineering purposes. The output must be quantitative and meaningful and the cost kept at reasonable levels. The practical methods to be described will be biased in the direction of quantifying the non-linear shear, deformation and permeability behaviour of joints, based on the Barton-Bandis (BB) rock engineering modelling concepts. The term 'modelling' will be used to introduce the basic stress-displacement-dilation behaviour of joints in shear, and the basic stress-closure behaviour when joints are compressed by increased normal stress. These are the basic elements of the (non-linear) behaviour, which are used when modelling the two- or three-dimensional behaviour of a jointed rock mass. They are the basic BB (Barton-Bandis) components of any UDEC-BB distinct element numerical model (used commercially and for research since 1985). The BB approach can also be used to determine improved MC (Mohr-Coulomb) strength components for a 3DEC-MC three-dimensional distinct element numerical model. In other words for acquiring input at the appropriate levels of effective stress, prior to BB introduction into 3DEC, believed to be a project underway. Due to space limitations, constant stiffness BB behaviour of rock joints is given elsewhere.

Keywords: joint characterization, roughness, wall-strength, peak strength, shear stiffness, normal stiffness, physical and hydraulic apertures (Quantification of parameters: JRC, JCS, φ_r, K_s, K_n, E and e)

2017

Proceedings of the World Tunnel Congress 2017 – Surface challenges – Underground solutions. Bergen, Norway.

Tunneling through Either Intact, or Jointed, or Faulted Rock

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ABSTRACT: Rock masses obviously represent the most variable of engineering materials, and the three categories listed in the title represent most of the range of rock quality. We need to include stress magnitudes, water pressure and permeability to be more complete, but will ignore swelling pressures. This keynote paper will address some new findings about tunneling through these three categories. For instance, the critical tangential stress experienced when fracturing of intact rock initiates in deep tunnels is caused by the ratio of tensile strength/Poisson's σ_t/ν. This explains the critical ratio $\sigma_\theta/\sigma_c \approx 0.4$ The extensional strain-induced fracturing will mostly propagate in shearing mode at higher stress levels, and this can be the source of rock bursts. The presence of jointing helps to dissipate the latter. Contrary to the experiences with drill-and-blast tunneling, TBM experience difficulties at both ends of the spectrum (intact, jointed, faulted). Partly for this reason, deceleration from day to week to month to one year is a common experience, also shown by the remarkable world records of TBM performance. A simple formula explains the delays of TBM tunnels in fault zones.

1 THE ROCK THAT BEARS THE LOAD

When we excavate a tunnel through intact, jointed or faulted rock masses, with Q-values potentially ranging from 1000 to 0.001, how important is the capacity of the support and reinforcement of the tunnel periphery, compared to the capacity of the surrounding 'cylinder' of rock mass to take load? The redistributed stresses, and the slightly deforming and adjusting rock blocks in the surrounding 'cylinder' (with dimensions which may be up to several tunnel diameters in thickness), account for a huge majority of the load-bearing abilities, except when very close to the surface.

The shotcrete, rock bolts, and occasional concrete of single-shell NMT (Q-system-based) tunnels, are selected merely to retain the load bearing abilities of the all-important surrounding rock mass. Naturally we can assist this process with sufficiently high-pressure pre-injection, if using stable (non-shrinking and non-bleeding) microcement suspensions. It is believed that most of the six Q-parameters are effectively improved by correctly carried-out high-pressure pre-injection. We know of velocity increases and permeability tensor rotations and magnitude reductions, even as a result of quite conservative grouting in dam abutments (Barton, 2012a). Hydraulic (e) and physical apertures (E) must be differentiated.

Even the thickest concrete lining can hardly compete with the hundreds or thousands of tons of load that are arched around each running meter of a tunnel. For instance, a 5MPa vertical stress at 200m depth, which may be concentrated to more than 10MPa in the tunnel walls, arch or invert (depending on the stress anisotropy) causes variation from 1,000 to 500 tons/m^2 in the nearest meter-thick-meter-wide, naturally load-bearing 'rock-mass-ribs' which surround the excavation. In the first 10m of the surrounding rock 'cylinder' an estimated load-in-the-arch of 5,000 to 10,000 tons per running meter of tunnel, obviously far exceeds the load-bearing abilities of shotcrete, rock bolts or concrete. At 1,000m depth the load-bearing capacity of the natural rock arch is even more essential.

The 'softest' support of all, the lattice girders used in NATM tunneling, have little to contribute in hard rock with marked over-break, because good contact with the tunnel perimeter is difficult. Why do we seldom see over-break and its volumetric (and stress-distribution) consequences in drawings and numerical models of concrete-lined NATM tunnels?

In both single-shell NMT and double-shell NATM philosophies, we are attempting to *help the rock to help itself*. Clearly there are potentially big cost differences depending on how we do this, but these will not be addressed

2017

Limited heights of cliffs, mountain walls and mountains using rock mechanics

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Abstract

Intact brittle rock can fail in tension even when all principal stresses are compressive. This is due to lateral expansion and extension strain when near to a free surface, caused by Poisson's ratio. Tensile strength and Poisson's ratio are the fracture-initiating parameters around deep tunnels, not the increasing mobilization of compressive strength, commonly beyond 0.4 x UCS. In a related discovery, the limiting height of vertical cliffs and near-vertical mountain walls can also be explained using extension strain theory. The range of limiting heights of approximately 20m for cliffs in porous tuff to record 1,300m high mountain walls in granite are thereby explained. The world's highest mountains are limited to 8 to 9km. This is due to non-linear critical state rock mechanics. Maximum shear strength is the weakest link when stress levels are ultra-high, while tensile strength is the weakest link behind cliffs and ultra-steep mountain walls. Sheeting joints can also be explained by extension strain theory.

Keywords: cliffs; mountains; sheeting joints; extension strain; tensile strength; shear strength

1 Introduction

1.1 The lessons from deep tunnels

The starting point for the ultra-simple cliff-height and mountain wall-height equation which is introduced in this article is the observed and recently modelled fracturing behavior of deep tunnels in massive rock. Fracturing may be initiated by extensional strain overcoming the tensile limit, even when all stresses are compressive. This is possible due to the lateral expansion caused by Poisson's ratio. A small-scale example of this is the acoustic emission that occurs due to micro-fracture initiation when testing intact rock cylinders in traditional uniaxial compression, where Poisson's ratio is also at work. The commonly used parameter obtained from such tests is σ_c, the unconfined compression strength (commonly written as UCS). This might be 120MPa for granite but only 1MPa for weak porous tuff, the medium once used by Christian cliff-dwellers in Cappadocia, Turkey. The tuffs are used today for

2017

Journal of Rock Mechanics and Geotechnical Engineering 9 (2017) 671–682

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

Nonlinear shear behavior of rock joints using a linearized implementation of the Barton–Bandis model

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ARTICLE INFO

Article history:
Received 1 October 2016
Received in revised form
28 December 2016
Accepted 4 January 2017
Available online 10 July 2017

Keywords:
Rock joints
Joint shear behavior
Friction and dilation
Barton–Bandis (B–B) model
Equivalent Mohr–Coulomb (M–C) parameters

ABSTRACT

Experiments on rock joint behaviors have shown that joint surface roughness is mobilized under shearing, inducing dilation and resulting in nonlinear joint shear strength and shear stress vs. shear displacement behaviors. The Barton–Bandis (B–B) joint model provides the most realistic prediction for the nonlinear shear behavior of rock joints. The B–B model accounts for asperity roughness and strength through the joint roughness coefficient (*JRC*) and joint wall compressive strength (*JCS*) parameters. Nevertheless, many computer codes for rock engineering analysis still use the constant shear strength parameters from the linear Mohr–Coulomb (M–C) model, which is only appropriate for smooth and non-dilatant joints. This limitation prevents fractured rock models from capturing the nonlinearity of joint shear behavior. To bridge the B–B and the M–C models, this paper aims to provide a linearized implementation of the B–B model using a tangential technique to obtain the equivalent M–C parameters that can satisfy the nonlinear shear behavior of rock joints. These equivalent parameters, namely the equivalent peak cohesion, friction angle, and dilation angle, are then converted into their mobilized forms to account for the mobilization and degradation of *JRC* under shearing. The conversion is done by expressing *JRC* in the equivalent peak parameters as functions of joint shear displacement using proposed hyperbolic and logarithmic functions at the pre- and post-peak regions of shear displacement, respectively. Likewise, the pre- and post-peak joint shear stiffnesses are derived so that a complete shear stress-shear displacement relationship can be established. Verifications of the linearized implementation of the B–B model show that the shear stress-shear displacement curves, the dilation behavior, and the shear strength envelopes of rock joints are consistent with available experimental and numerical results.

1. Introduction

Rock masses contain various types of discontinuities, such as bedding planes, joints, shear zones, and faults, which distinguish them from other materials. Joints are commonly observed and widespread in rocks. Unless they are healed and mineralized, joints have weakening effects on the strength of rock masses because they do not have tensile resistance and their shear strength is usually much smaller compared to the intact rock matrix (Priest, 1993; Singhal and Gupta, 2010). In a fractured rock mass, the shear behavior of rock joints is particularly important because it dominantly controls the deformability, strength, and hence the stability of the rock mass. For example, the conditions for slip on major pervasive features in fractured rock masses, such as block sliding from a slope or block falling in an underground excavation, are controlled not only by the shear strength of the particular joint but also by its dilation along asperities under shearing (Goodman, 1976, 1989; Barton and Hansteen, 1979; Barton, 1982). This dilation is caused by the mobilization of joint surface roughness, and it will cause nonlinearity in the shear strength as well as strain hardening and strain softening in the shear behavior of rock joints. Therefore, it is essential for fractured rock models to take into account this nonlinearity so that the realistic response of fractured rock masses can be accurately predicted, leading to the economical and reliable design and analysis of excavations and structures in rocks.

Extensive laboratory experiments on rock joint behavior from the early 1960s to date have shown that the shear behavior of rock

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

http://dx.doi.org/10.1016/j.jrmge.2017.01.006

2017

International Progress Report

IPR-04-07

Äspö Hard Rock Laboratory

Äspö Pillar Stability Experiment

Q-logging of the TASQ tunnel at Äspö

For rock quality assessment and for development of preliminary model parameters

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August 2003

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Abstract

Statistical logging of individual Q-parameters has been performed along 55m of the APSE tunnel (ch. 24 to 80m) and for the full length of the KA 3376 B01 core, which runs in the left wall of the tunnel. The overall picture is of increased jointing towards the end of the tunnel and towards the end of the borehole. The best values of Q_{mean} and $Q_{most\ frequent}$ and relative block size RQD/J_n were registered in the chainage 45 to 53m. Since the target area, with completed invert, is from about ch. 60 to 75m, the Q_{mean} and $Q_{most\ frequent}$ averages of 20 and 40 from this 15m of tunnel have been used to make preliminary estimates of seismic velocity (5.8 to 6.0 km/s) and E_{mass} (59 to 66 GPa). Estimates have also been made of tunnel convergence, which corresponds quite well to measured convergences. Rock mass strengths and cohesive and frictional strengths have also been empirically estimated, based on the Q-logging.

2003 (Omission: Inserted here to avoid displacing all 'boxes')

2018 'Letter' to Research Gate: Response to authors who suggested RQD could 'rest in peace' By Nick Ryland Barton

The respected developers of RMR (Dr. Dick Bieniawski) and RQD (Dr. Don Deere) passed away on either side of the 2017/2018 New Year. They will be remembered both with thanks and initial sadness. There is no doubt that rock mechanics and rock engineering users of their methods form only a very small portion of the 14 million users of Research Gate with its remarkable 100,000,000 research items. Nevertheless, perhaps most of our millions of colleagues will also have driven through a tunnel or drunk water from a reservoir where one of these rock mass classification methods were used in the drill-core related investigations for the subterraneous constructions, in infinitely diverse rock masses, in our numerous countries. So, the respected professors' passing, and their heritage is important to mark, also in 'the pages' of RG. RQD in particular has had enormous application in civil engineering.

As in all scientific and engineering fields there is competition and there are strong opinions. This is how each subject develops. Better arguments may be needed, or better methods eventually replace the old. Related exactly to these opinions it has recently been suggested by a small hand-full of co-authors that RQD, which was developed in 1964, should be replaced, or should rest in peace. This is an opinion that is hardly likely to be shared by many of the tens of thousands of engineering geologists who have used this method and will no doubt be using it in the future as well. Fifty years is a respectable track record in any field. (cont).

2018

TBM Tunnelling under Difficult Conditions: Too Massive, Too Faulted, Too Wet, Too Deep

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ABSTRACT: It is common knowledge that TBM are remarkable machines. It nevertheless takes mental effort to accept that they can have world record 1 day, 1 week and 1 month tunnel advance as high as 172m, 703m and 2163m. However, the best monthly project averages for the usually smaller 3m to 6m diameter machines which have delivered these incredible records are 'only' 1.1 to 1.3 km, not anywhere near 2km. In other words, tunnel length and time take a toll, due both to geology, hydrogeology, and machine-related delays. Unfortunately, the other side of the coin has seen more than a few TBM that remain buried in mountains forever (needing drill-and-blast completion), or they are delayed for many months or even years. This huge range of performance demands a lot from models of TBM prognosis: the range of tunnel advance may vary over four orders of magnitude from 0.001m/hr (i.e. stuck in fault zone) to an occasional 10m/hr penetration rate. Because of the huge range of an empirical parameter Q_{TBM} this range is possible to encompass, and is founded on the 0.001-1000 Q-value range, plus particular emphasis on comparing cutter thrust and rock mass strength, 1 to 100MPa.

1 INTRODUCTION – WORLD RECORDS

TBM prognosis models must be capable of explaining less than 1m/hr penetration rate (PR) in hard massive rock, perhaps short-term 10m/hr penetration rate (PR) in softer jointed rock, but only 0.01 m/hr average advance rate (AR) when severely delayed in fault zones. The implied eighty meters during one year, struggling to get through a faulted zone is clearly close to the limit of acceptance. Values an order of magnitude lower than this mean virtual burial. So we see in these figures a ten thousand-fold variation (fastest 10m/hr, slowest 0.001m/hr) that needs a geo-technical, quantifiable explanation.

In this lecture we will examine the adverse effects of massive hard rock, faulted rock with and without the complications of wet tunnels requiring delaying pre-injection, and finally TBM tunnels that are actually too deep in relation to the strength of the rock, and therefore suffer rock bursts.

Thanks to some detailed TBM world record advance rate statistics provided by Robbins on the internet, it was possible to derive the present (2015) record data shown in Figure 1. The 3 to 6m diameter class shown with the smallest 'cubes' is *the mean of three sets* of data given for 3-4m, 4-5m and 5-6m TBM, based on assumed 24 hours, 168 hours and 720 hours. The 6 to10m diameter class shown with the larger 'cubes' is *the mean of four sets* of data for 6-7m, 7-8m, 8-9m and 9-10m TBM. This collective averaging helps to see trends more clearly.

In Figure 1, day, week and month records (given in meters) are converted to the form AR (m/hr) by dividing by 24, 168 and 720 hours. Data from 8 countries are represented, but chiefly USA and China. The record *mean monthly* data plots at AR = 1.7 m/hr for the 3m to 6m class, and at AR = 1.1 m/hr for the 6m to 10m class. These results are shown with the two small circles. The larger crossed-circle to the right represents 54 weeks for 5.8 km at the Svea Mine Access Tunnel, achieved during the LNS drill-and-blast world record. This was driven in coal-measure rocks and obviously required some shotcreting and rock bolting, due to varied Q-values. Slowest progress was made through a near-surface zone of permafrost.

2 CASE RECORDS SHOW DECELERATION

There is an all too common habit of reporting utilization (U) of TBM without specifying the time period involved. An estimated average daily utilization is especially an insufficient form of prognosis. Since stand-stills are naturally excluded, the client may get an optimistic view of likely performance. Utilization is estimated from the classic and most used TBM

2018

Periodica Polytechnica Civil Engineering

Rock Slope Design using Q-slope and Geophysical Survey Data

OnlineFirst (2018) paper 12287
https://doi.org/10.3311/PPci.12287

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RESEARCH ARTICLE

Received 22 March 2018; Accepted 08 April 2018

Abstract
The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of excavated rock slopes in the field. It enables rock engineers and engineering geologists to make potential adjustments to slope angles as rock mass conditions become apparent during the construction of reinforcement-free road or railway cuttings and in open cast mines. Q-slope was developed by supplementing the Q-system which has been extensively used for characterizing rock exposures, drill core and underground mines and tunnels under construction for over 40 years. The Q' parameters (RQD, J_n, J_r and J_a) have remained unchanged in Q-slope, although a new method for applying J_r/J_a ratios to both sides of a potential wedge is used, with relative orientation weightings for each side. The term J_w has been replaced with the more comprehensive term J_{wice}, which takes into account long-term exposure to various climatic and environmental conditions such as intense erosive rainfall and ice-wedging effects. SRF categories have been developed for slope surface conditions, stress-strength ratios and major discontinuities such as faults, weakness zones or joint swarms. Through case studies across Europe, Australia, Asia, and Central America, a simple relationship between Q-slope and long-term stable slope angles was established. The Q-slope method is designed such that it suggests stable, maintenance-free, bench face slope angles of, for instance, 40–45°, 60–65° and 80–85° with respective Q-slope values of approximately 0.1, 1.0 and 10. Q-slope has also been found to be compatible with P-wave velocity and acoustic and optical televiewer data obtained from borehole and surface-based geophysical surveys to determine appropriate rock slope angles.

Keywords
Q-slope, rock slopes, borehole geophysics, slope stability

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1 Introduction

Assessing the stability of rock slope cuttings and benches in real-time, as excavations progress and ground conditions become apparent, using analytical approaches such as kinematics, limit equilibrium or finite and discrete element models is practically impossible in both civil and mining engineering projects. The rate of excavation is too fast for this. The same limitation usually applies to tunneling, although large underground openings (e.g. caverns) are sufficiently stationary for thorough and more necessary analysis, and the same applies to high rock slopes.

Several empirical methods for assisting rock engineering design have been developed in the last 50 years and are used for a variety of applications by rock engineers and engineering geologists, primarily for tunneling and support of underground excavations. In the case of rock slopes, some empirical methods predict support, reinforcement and performance of excavated slopes. However, aside from Q-slope, no empirical rock engineering methods provide guidance in relation to appropriate, long-term stable slope angles in which reinforcement and support is deliberately absent. Such slopes actually dominate the demand by a huge margin.

2 Q-System

The Q-system for characterizing rock exposures, drill core and tunnels under construction was developed from tunneling-related and cavern-related case records [1] [2]. Single shell *B + S(fr)* tunnel support and reinforcement design assistance, and open stope design, utilizing *Q'* (the first four parameters: *RQD*, J_n, J_r & J_a) have been the principal focus of applications in civil and mining engineering. Correlations of Q_c (*Q* normalized with UCS/100) with stress-dependent P-wave velocities and depth-dependent deformation moduli have also proved useful in site characterization and as input to numerical modelling. These approximations remain with the Q-slope value, which may also vary over six orders of magnitude from approximately 0.001 to 1000. This large numerical range is a reflection of the large variation of parameters such as deformation moduli and shear strength.

2018

Rock Mechanics and Rock Engineering
https://doi.org/10.1007/s00603-018-1558-2

ORIGINAL PAPER

CrossMark

Extension Strain and Rock Strength Limits for Deep Tunnels, Cliffs, Mountain Walls and the Highest Mountains

Nick Barton[1] · Baotang Shen[2]

Received: 13 March 2018 / Accepted: 24 July 2018

Abstract
Brittle rock can fail in tension even when all principal stresses are compressive. The culprit is Poisson's ratio, but marked stress anisotropy due to a neighbouring free surface, and due to a raised principal tangential stress is also necessary. Extension strain-induced failure causes fracture initiation in tension. Propagation in unstable shear may occur if the tunnels or mine openings are deep enough, and if they are located in hard, brittle, sparsely jointed rock. Both in laboratory uniaxial compression test samples with strength σ_c and in deep tunnels, extension fracturing and acoustic emission begin when the principal applied or induced stress reaches the magnitude of tensile strength divided by Poisson's ratio σ_t/ν. The traditionally expected fracture initiation when the principal or maximum tangential stress σ_1 or $\sigma_\theta = 0.4 \pm 0.1 \times \sigma_c$ can actually be explained with arithmetic. Using related logic, cliffs and the near-vertical mountain walls frequented by rock climbers, may have erosional or glacial origin, but extension strain limits their height, including vertical walls of sheeting joints and long continuous fractures. Shear failure seems to be reserved for occasional major rock avalanches. Equations with soil mechanics origin involving Coulomb parameters c and φ and density that may apply to vertical cuts in soil, give greatly exaggerated heights for rock cliffs and mountain walls since rock is brittle and favours failure in tension. Tensile strength, Poisson's ratio and density are suggested for estimating the maximum heights of rock cliffs and mountain walls, not compression strength and density. However, overall mountain heights are limited by critical state maximum shear strength, or by the slightly lower brittle–ductile transition strength.

Keywords Extension strain · Tensile strength · Poisson's ratio · Shear strength · Fracturing · Tunnels · Cliffs · Mountain walls · Mountains

Abbreviations

σ_c	Uniaxial compression strength (of rock)	SRF	Stress reduction factor (from *Q*-value)
q_c	Unconfined compression strength (of soil)	R_f	Depth of failure + excavation radius (a)
σ_t	Uniaxial tensile strength (of rock)	FRACOD	Fracture mechanics numerical code
ν	Poisson's ratio	DDM	Displacement discontinuity method
σ_h	Minor horizontal principal stress	NGI	Norwegian Geotechnical Institute
σ_H	Major horizontal principal stress	Q	Rock mass quality
σ_v	Vertical principal stress	ε_3	Lateral extension strain (radial)
σ_1	Major principal stress	ε_t	Critical extensional strain
σ_3	Minor principal stress	E	Young's modulus
k_0	Ratio of σ_h/σ_v	$E' = E/(1-\nu^2)$	For plane strain
K_0	Ratio of σ_H/σ_v	H_c	Critical height of vertical cutting in soil
σ_θ	Maximum tangential stress (also σ_{max})	C	Cohesion of soil (or intact rock)
		φ	Friction angle of soil (or intact rock)
		γ	Density of soil (or intact rock)
		JRC	Joint roughness coefficient
		JCS	Joint wall compression strength
		R	Equivalent roughness of broken rock, screes
		S	Equivalent strength of broken rock, screes

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2018

Geomechanics and Engineering, *Vol. 16, No. 1 (2018) 35-47*
DOI: https://doi.org/10.12989/gae.2018.16.1.035

Rock fracturing mechanisms around underground openings

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(Received August 17, 2017, Revised February 26, 2018, Accepted March 13, 2018)

Abstract. This paper investigates the mechanisms of tunnel spalling and massive tunnel failures using fracture mechanics principles. The study starts with examining the fracture propagation due to tensile and shear failure mechanisms. It was found that, fundamentally, in rock masses with high compressive stresses, tensile fracture propagation is often a stable process which leads to a gradual failure. Shear fracture propagation tends to be an unstable process. Several real case observations of spalling failures and massive shear failures in boreholes, tunnels and underground roadways are shown in the paper. A number of numerical models were used to investigate the fracture mechanisms and extents in the roof/wall of a deep tunnel and in an underground coal mine roadway. The modelling was done using a unique fracture mechanics code FRACOD which simulates explicitly the fracture initiation and propagation process. The study has demonstrated that both tensile and shear fracturing may occur in the vicinity of an underground opening. Shallow spalling in the tunnel wall is believed to be caused by tensile fracturing from extensional strain although no tensile stress exists there. Massive large scale failure however is most likely to be caused by shear fracturing under high compressive stresses. The observation that tunnel spalling often starts when the hoop stress reaches 0.4*UCS has been explained in this paper by using the extension strain criterion. At this uniaxial compressive stress level, the lateral extensional strain is equivalent to the critical strain under uniaxial tension. Scale effect on UCS commonly believed by many is unlikely the dominant factor in this phenomenon.

Keywords: tunnel spalling; fracture propagation; extension strain criterion; shear fracturing; failure mechanism; FRACOD

1. Introduction

Rock masses are increasingly employed as the host medium in a vast array of human activities. Facilities like storage caverns, petroleum wells, water and transport tunnels, and underground power stations are located in a variety of rock types and suffer extra challenges when at significant depth. Excavation stability is imperative for all such constructions, in both the short and long term. The understanding of fracturing of rock masses has become a necessity for deep rock excavations in brittle rocks. Small-scale breakouts around single wells in petroleum engineering help to indicate principal stress direction and the degree of stress anisotropy. Large-scale stress-or-strain induced fracturing in tunnels can lead to massive tunnel failure which not only increases the time and cost of tunnel excavation and maintenance, but also imposes serious safety threats to personnel, and occasionally leads to fatalities.

Failure of brittle rock is often associated with explicit fracturing events. The mechanisms of rock fracturing around an actual underground excavation are often complex and have been constantly debated amongst researchers. Tunnel spalling is the most commonly observed fracturing phenomenon in highly stressed brittle rock, and most researchers believe it is caused by tensile fracturing (Andersson 2007, Martin and Chandler 1994). However, researchers have been struggling to explain convincingly why tensile fracturing occurs in the tunnel wall where no tensile stress exists. Also difficult to explain is that the spalling tends to start when the maximum estimated hoop or tangential stress reaches approximately 0.4*UCS (Uniaxial Compressive Strength) (Martin *et al.* 1999). Some researchers tend to believe this may be a logical scale effect on UCS. However, this phenomenon not only occurs in large scale tunnels but also in laboratory scale samples (Martin 1997), making the scale effect theory inadequate.

Large scale massive failures have been observed in tunnels and boreholes under very high stresses (Barton 2006). This is believed to be caused predominately by shear fracturing. Fracturing around boreholes drilled at various angles into a highly-stressed brittle medium in the laboratory (not a thick-walled cylinder test) was consistently caused by the log-spiral shear mechanism (Addis *et al.* 1990).

Diederichs (2003) and Diederichs *et al.* (2004) carried out detailed studies on the mechanisms of rock fracturing in hard rocks, and believed that, depending on the stress state, failure could be caused by shear (high confining stress), spalling (low confining stress) or tension (tensile stress), see

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ISSN: 2005-307X (Print), 2092-6219 (Online)

2018

Q-SLOPE: AN EMPIRICAL ROCK SLOPE ENGINEERING APPROACH IN AUSTRALIA

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ABSTRACT

The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of excavated rock slopes in the field. Q-slope allows geotechnical engineers and engineering geologists to make potential adjustments to slope angles as rock mass conditions become apparent during the construction of reinforcement-free road or railway cuttings and in open pit mines. Through case studies across Australia, the Americas, Asia and Europe, a simple correlation between Q-slope and long-term stable slopes was established. The Q-slope method is designed such that it suggests stable, maintenance-free, bench face slope angles of, for instance, 40-45°, 60-65° and 80-85° with respective Q-slope values of approximately 0.1, 1.0 and 10.

Q-slope was developed by supplementing the Q-system which has been extensively used for characterizing rock exposures, drill core and underground mines and tunnels under construction for the last 40 years. The Q' parameters (RQD, J_n, J_r and J_a) have remained unchanged in Q-slope, although a new method for applying J_r/J_a ratios to both sides of a potential wedge is used, with relative orientation weightings for each side. The term J_w has been replaced with the more comprehensive term J_{wice}, which takes into account long-term exposures to various climates and environments. SRF categories have been developed for slope surface conditions, stress-strength ratios and major discontinuities such as faults, weakness zones or joint swarms.

This paper discusses civil and mining engineering applications of the Q-slope method in Australia for a variety of ground conditions from very weak to strong rocks, blocky to massive, isotropic rock masses to laminated, heterogeneous, highly anisotropic rock masses. A case study is also presented to illustrate the compatibility of Q-slope with P-wave velocity and acoustic and optical televiewer data obtained from borehole geophysical surveys to determine appropriate rock slope angles.

NOMENCLATURE

RQD – rock quality designation	SRF_{slope} – largest of three strength reduction factors: a, b and c
J_n – joint sets number	SRF_a – physical condition number
J_r – joint roughness number	SRF_b – stress and strength number
J_a – joint alteration number	SRF_c – major discontinuity number
J_{wice} – environmental & geological condition number	O-factor – orientation factor for the ratio J_r/J_a

1 INTRODUCTION

In both civil and mining engineering projects, it is practically impossible to assess the stability of rock slope cuttings and benches in real-time using analytical approaches such as kinematics, limit equilibrium or finite and distinct element modelling. Excavation is usually too fast for this. Furthermore, in Australia, the cost of engineering services and labour are too high to facilitate such detailed slope design guidance and reconciliation during excavation. The same limitation usually applies to tunnelling, although caverns and large underground openings are sufficiently stationary for thorough and more necessary analysis, and the same applies to high rock slopes.

The purpose of Q-slope is to allow engineering geologists and geotechnical engineers to assess the stability of excavated rock slopes in the field, and make potential adjustments to slope angles as rock mass conditions become visible during construction (Barton & Bar, 2015). Key areas of Q-slope application are from the surface and downwards: bench face angle decisions in open pit mines, and for numerous slope cuttings to reach remote project sites in mountainous terrain through varying geological conditions. In many rock slope problems, the engineer needs to quickly decide whether the slope will be excavated at angles of 45 to 90° or shallower than 45°. The use of Q-slope during excavation can help to reduce

2018

XIX Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica
Geotecnia e Desenvolvimento Urbano
COBRAMSEG 2018 – 28 de Agosto a 01 de Setembro, Salvador, Bahia, Brasil

Limited heights of vertical cliffs and mountain walls linked to fracturing in deep tunnels - Q-slope application if jointed slopes

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ABSTRACT: Intact brittle rock can fail in tension even when all principal stresses are compressive. This is due to lateral expansion and extension strain when near to a free surface, caused by Poisson's ratio. Exceeding tensile strength due to stress anisotropy and Poisson's ratio are the fracture-initiating conditions around deep tunnels, not the increasing mobilization of compressive strength, commonly beyond 0.4 x UCS. In a related discovery, the limiting height of vertical cliffs and near-vertical mountain walls can also be explained using extension strain theory. The range of limiting heights of approximately 20m for cliffs in porous tuff to record 1,300m high mountain walls in granite are thereby explained. Tensile strength is the weakest link behind cliffs and ultra-steep mountain walls. Sheeting joints can also be explained by extension strain theory. Maximum shear strength is the weakest link when stress levels are ultra-high, or when there is jointing and maximum slope angles is the issue. Here one can use Q-slope. The world's highest mountains are limited to 8 to 9km. This is due to non-linear critical state rock mechanics. It is not due to UCS.

KEY WORDS: Deep tunnels, Cliffs, Mountains; Extension strain; Tensile strength; Shear strength

INTRODUCTION

The lessons from fracturing in deep tunnels is the starting point for the ultra-simple cliff-height and mountain wall-height equation which is introduced in this article. The observed and recently modelled fracturing behavior of deep tunnels in massive rock indicates that fracturing may be initiated by extensional strain over-coming the tensile limit, even when all stresses are compressive. This is possible due to the lateral expansion caused by Poisson's ratio. A small-scale example of this is the acoustic emission that occurs due to micro-fracture initiation when testing intact rock cylinders in traditional uniaxial compression, where Poisson's ratio is also at work. The commonly used parameter obtained from such tests is σ_c, the unconfined compression strength (commonly written as UCS). This might be 150MPa for granite but only 1.5MPa for weak porous tuff, the medium once used by Christian cliff-dwellers in Cappadocia, Turkey. The tuffs are so weak that there have been many historic cliff failures, which expose old dwellings and Christian churches at irregular intervals. The most basic strength parameter σ_c has traditionally been compared with the estimated maximum tangential ('arching') stress, to investigate if a deep tunnel will suffer fracturing or rock-burst and need more support like sprayed concrete and rock bolts. A newly excavated tunnel results in a big contrast between the maximum tangential ('arching') stress (σ_θ) and the almost unloaded radial stress (σ_r). For elastic isotropic materials and a circular tunnel, the theoretical maximum tangential stress is three times the major principal stress (σ_1) minus the minimum principal stress (σ_2) acting in the same plane, at right angles to the tunnel. At 1,000m depth we

2018

Journal of Rock Mechanics and Geotechnical Engineering 10 (2018) 645–652

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

An approximate nonlinear modified Mohr-Coulomb shear strength criterion with critical state for intact rocks

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ARTICLE INFO

Article history:
Received 18 October 2017
Received in revised form
26 April 2018
Accepted 28 April 2018
Available online 30 May 2018

Keywords:
Shear strength
Modified Mohr-Coulomb criterion
Critical state
Intact rock

ABSTRACT

In this paper, the Mohr-Coulomb shear strength criterion is modified by mobilising the cohesion and internal friction angle with normal stress, in order to capture the nonlinearity and critical state concept for intact rocks reported in the literature. The mathematical expression for the strength is the same as the classical form, but the terms of cohesion and internal friction angle depend on the normal stress now, leading to a nonlinear relationship between the strength and normal stress. It covers both the tension and compression regions with different expressions for cohesion and internal friction angle. The strengths from the two regions join continuously at the transition of zero normal stress. The part in the compression region approximately satisfies the conditions of critical state, where the maximum shear strength is reached. Due to the nonlinearity, the classical simple relationship between the parameters of cohesion, internal friction angle and uniaxial compressive strength from the linear Mohr-Coulomb criterion does not hold anymore. The equation for determining one of the three parameters in terms of the other two is supplied. This equation is nonlinear and thus a nonlinear equation solver is needed. For simplicity, the classical linear relationship is used as a local approximation. The approximate modified Mohr-Coulomb criterion has been implemented in a fracture mechanics based numerical code FRACOD, and an example case of deep tunnel failure is presented to demonstrate the difference between the original and modified Mohr-Coulomb criteria. It is shown that the nonlinear modified Mohr-Coulomb criterion predicts somewhat deeper and more intensive fracturing regions in the surrounding rock mass than the original linear Mohr-Coulomb criterion. A more comprehensive piecewise nonlinear shear strength criterion is also included in Appendix B for those readers who are interested. It covers the tensile, compressive, brittle-ductile behaviour transition and the critical state, and gives smooth transitions.

1. Introduction

The traditional Mohr-Coulomb shear strength criterion considers the strength to be linearly depending on the normal stress on the shear plane. It has been widely reported that the shear strength of many rocks actually follows a nonlinear relationship with the normal compressive stress, especially at extremely high confining pressure (e.g. Barton, 1976), and even at relatively low confining pressure (e.g. Mogi, 1974), if the rock is weak. The shear strength envelopes in the τ-σ_n plane are concave towards the normal compressive stress axis, where τ is the shear strength and σ_n is the normal stress on the shear plane. Many nonlinear shear strength criteria exist in the literature, including the Barton criterion (Barton, 1976, 2013) and Hoek-Brown criterion (Hoek and Brown, 1980a, b, Hoek and Brown 1988). Barton (2013) summarised the nonlinear shear strengths for intact rocks, fractured rocks, jointed rocks and rockfills.

Mogi (1966) compiled a large body of triaxial experimental data for rocks from a variety of sources. Fig. 1 (from Barton (1976)) reproduces Mogi's test data for dry carbonate rocks and shows the variation of shear strength with the confining pressure. It can be seen that for most rock samples, the increase in the shear strength reduces and becomes negligibly small beyond a certain confining

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

https://doi.org/10.1016/j.jrmge.2018.04.002

2018

Thermal over-closure of rougher joint sets – consequences for HLW disposal strategies and HTM modelling.

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1. Introduction

The writer's first experience of (ambient temperature) over-closure of rough fractures was during Ph.D. studies at Imperial College, fifty years ago, when model rock slopes (in 40,000 block tension-fracture models) excavated in 'green-field' situations would not fail at the expected slope angles. Conventional 1:1, and over-closed 4:1 and 8:1 direct shear tests (with a prior normal stress higher than in the following DST of the same rough fractures) showed successively steeper shear strength envelopes [1]. Subsequently, while at NGI, a four-cavern 20,000 blocks model, also pre UDEC, demonstrated over-closure / hysteresis since deformation was not reversed in pillars when successive caverns were excavated. [2]. The rough fracture sets were exhibiting some tensile strength and higher shear strength due to prior-to-excavation higher normal 'tectonic' $\sigma_h > \sigma_v$ boundary stresses. (see Fig. 1).

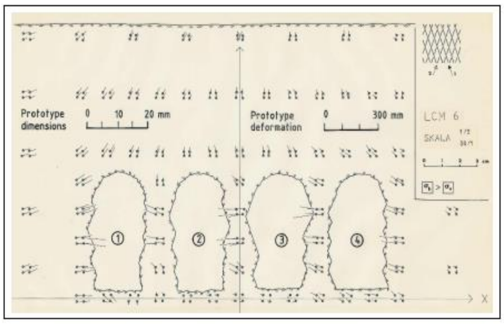

Figure 1. Caverns excavated in chronologic order 1 through 4, showing hysteresis (no reversal of deformation vectors) due to over-closure of the rough fractures, which were under higher normal stress prior to excavation.

Rough joints in igneous and metamorphic rocks can *over-close* even due to temperature increase alone, due to better fit, as conditions closer to their formation temperature are reached. Mineral-constituent thermal expansion coefficients are to blame. As a result, the *rock mass* deformation moduli, the mass thermal expansion coefficients, seismic velocities (each likely to be anisotropic), and the physical and hydraulic apertures of individual joint sets may each be affected. The initial cause is lowered normal stiffness of the roughest set of joints due to the thermal over-closure. An important side-effect: direct shear strength is increased due to the reduced physical apertures.

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Barton-Bandis Criterion

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Definition

A series of rock-joint behavior routines which, briefly stated, allow the *shear strength and normal stiffness* of rock joints to be estimated, graphed, and numerically modelled, for instance, in the computer code UDEC-BB. Coupled behavior with deformation and changes in conductivity is also included (Barton 2016).

A key aspect of the criterion is the quantitative characterization of the joint, joints, or joint sets in question, in order to provide three simple items of input data. These concern the joint-surface roughness (*JRC: joint roughness coefficient*), the joint-wall compressive strength (*JCS: joint compressive strength*), and an empirically derived estimate of the *residual friction angle* (φ_r). These three parameters have typical ranges of values from: JRC = 0 to 20 (smooth-planar to very rough-undulating), JCS = 10 to 200 MPa (weak-weathered to strong, unweathered) and φ_r = 20° to 35° (strongly weathered to fresh-unweathered). Each of these parameters can be obtained from simple, inexpensive index tests or can be estimated by those with experience.

The three parameters JRC, JCS, and φ_r form the basis of the nonlinear peak shear-strength equation of Barton (1973) and Barton and Choubey (1977). This is a *curved shear strength envelope* without cohesion (c). It will be contrasted to the linear Mohr-Coulomb "c and φ" (with apparent cohesion) criterion later. To be strictly correct the original Barton equation utilized the basic friction angle φ_b of flat, unweathered rock surfaces (in 1973), while φ_r was substituted for φ_b following 130 direct shear tests on fresh and partly weathered rock joints (in 1977).

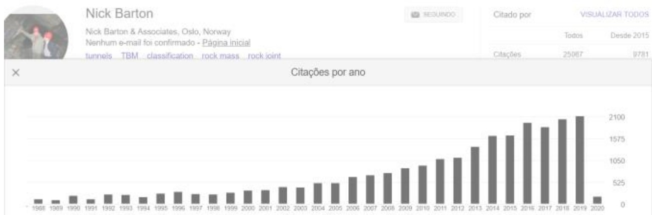
As well as peak and residual shear strength envelopes for laboratory-scale joint samples, Barton's cooperation with Bandis (from 1978) resulted in corrections (reductions) of JRC and JCS to allow for the *scale effect* and reduced strength as rock-block size is increased (Banton and Bandis 1982). The laboratory-scale parameters, written as JRC_0 and JCS_0 for laboratory-size samples of length L_0 (typically 50–250 mm), are written as JRC_n and JCS_n for in situ rock block lengths of L_n (typically 250–2500 mm, or even larger in massive rock).

Bandis is also responsible for utilizing JRC and JCS in empirical equations to describe *normal closure and normal stiffness*. Normal stiffness (Kn) has units of MPa/mm and might range from 20 to 200 MPa/mm. The Barton-Bandis (B-B) criterion includes the related modelling of *physical joint aperture E* (typically varying from 1 mm down to 50 µm, or 0.05 mm) as a result of the normal loading (or unloading). B-B also includes the theoretically equivalent smooth-wall *hydraulic aperture e* (typically 1 mm down to 5 µm, or 0.005 mm). Usually E > e, and the two are empirically inter-related, using the small-scale joint roughness JRC_0.

Finally the stiffness in the direction of shearing has also to be addressed. It is called *peak shear stiffness (Ks)*. It has typical values of 0.1 MPa to 10 MPa/mm, i.e., 1/10th to 1/100th of normal stiffness. The concept of *mobilized roughness* ($JRC_{mobilized}$) developed by Barton (1982) allows both the peak shear-stiffness and the *peak dilation angle* (giving an effective aperture increase with shearing) to be calculated. The full suite of Barton-Bandis joint behavior figures includes *shear stress-displacement-dilation, stress-closure, and the change of estimated conductivity* in each case. Examples of these will be given, following diagrams illustrating joint index testing (Figs. 1, 2, 3, 4, 5, 6, 7, and 8) (Barton and Bandis 2017).

P. T. Bobrowsky, B. Marker (eds.), *Encyclopedia of Engineering Geology*,
https://doi.org/10.1007/978-3-319-12127-7_25-1

2018



Engineering classification of rock masses for the design of tunnel support N Barton, R Lien, J Lunde Rock mechanics 6 (4), 189-236	3506	1974
The shear strength of rock joints in theory and practice N Barton, V Choubey Rock mechanics 10 (1-2), 1-54	2726	1977
Fundamentals of rock joint deformation SC Bandis, AC Lumsden, NR Barton International Journal of Rock Mechanics and Mining Sciences & Geomechanics ...	1750	1983
Strength, deformation and conductivity coupling of rock joints N Barton, S Bandis, K Bakhtar International journal of rock mechanics and mining sciences & geomechanics ...	1551	1985
Review of a new shear-strength criterion for rock joints N Barton Engineering geology 7 (4), 287-332	1491	1973
The shear strength of rock and rock joints N Barton International Journal of rock mechanics and mining sciences & Geomechanics ...	949	1976
Some new Q-value correlations to assist in site characterisation and tunnel design N Barton International journal of rock mechanics and mining sciences 39 (2), 185-216	903	2002
Experimental studies of scale effects on the shear behaviour of rock joints S Bandis, AC Lumsden, NR Barton International journal of rock mechanics and mining sciences & geomechanics ...	612	1981
Updating the Q-system for NMT E Grimstad Proceedings of the International Symposium on Sprayed Concrete-Modern use of ...	563	1993
Review of predictive capabilities ofJRC-JCS model in engineering practice N Barton, S Bandis Rock Joints, Proc int symp on rock joints, Loen, Norway (eds N. Barton and O ...	534 *	1990
Rock quality, seismic velocity, attenuation and anisotropy N Barton CRC press	489	2006
TBM tunnelling in jointed and faulted rock NR Barton Crc Press	456	2000
An improved model for hydromechanical coupling during shearing of rock joints R Olsson, N Barton International Journal of Rock Mechanics and Mining Sciences 38 (3), 317-329	446	2001
Effects of block size on the shear behavior of jointed rock N Barton, S Bandis The 23rd US symposium on rock mechanics (USRMS)	372	1982
Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and some solutions N Barton Journal of Rock Mechanics and Geotechnical Engineering 5 (4), 249-261	252	2013
A model study of rock-joint deformation NR Barton International Journal of Rock Mechanics and Mining Sciences & Geomechanics ...	209	1972
Modelling rock joint behavior from in situ block tests: implications for nuclear waste repository design N Barton Office of Nuclear Waste Isolation, Battelle Project Management Division	194	1982
A relationship between joint roughness and joint shear strength N BARTON Rock Fracture-Proc, Int. Symp. on Rock Mechanics, Nancy, France, paper1-8	191	1971
A review of the shear strength of filled discontinuities in rock N Barton Norwegian Geotechnical Institute Publication	186	1974
The influence of joint properties in modelling jointed rock masses N Barton 8th ISRM Congress	184	1995
Application of Q-system in design decisions concerning dimensions and appropriate support for underground installations N Barton, F Løset, R Lien, J Lunde Subsurface space, 553-561	181	1981
Shear strength of rockfill N Barton, B Kjaernsli Journal of Geotechnical and Geoenvironmental Engineering 107 (ASCE 16374)	178	1981
Joint conductivity variation due to normal and shear deformation A Makurat Proc. Int. Symp. Rock Joints, 1990, 535-540	159	1990
The disturbed zone around tunnels in jointed rock masses B Shen, N Barton International Journal of Rock Mechanics and Mining Sciences 34 (1), 117-126	146	1997
Deformation phenomena in jointed rock NR Barton Geotechnique 36 (2), 147-167	144	1986
Suggested methods for the quantitative description of discontinuities in rock masses N Barton ISRM, International Journal of Rock Mechanics and Mining Sciences ...	144	1978
Joint aperture and roughness in the prediction of flow and groutability of rock masses N Barton, EF de Quadros International Journal of Rock Mechanics and Mining Sciences 34 (3-4), 252 ...	137	1997
Technical Note: Some Effects of Scale on the Shear Strength of Joints N Barton, S Bandis International Journal of Rock Mechanics and Mining Sciences 17, 69-73	133	1980
TBM perfomance estimation in rock using QTBM N Barton Tunnel and Tunnelling International, 30-34	128	1999
The Q-system following twenty years of application in NMT support selection N Barton, E Grimstad International Journal of Rock Mechanics and Mining Sciences and Geomechanics ...	126	1995
Recent experiences with the Q-system of tunnel support design N Barton	122	1978
Aperture measurements and flow experiments using transparent replicas of rock joints E Hakami, N Barton Rock Joints, 383-390	121	1990

Note concerning total citations: Eight articles by other authors with 'N Barton' in list of authors (c/o Google citations) which are excluded here, means approx. 1800 spurious citations from other 'NB' authors. Cannot remove! (In 2020 listing these successfully removed.Total citations approx. 23,900).

Rock Mechanics and Rock Engineering
https://doi.org/10.1007/s00603-019-01902-9

TECHNICAL NOTE

Graphic Examples of a Logical Nonlinear Strength Criterion for Intact Rock

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Received: 12 February 2019 / Accepted: 29 June 2019

Keywords Shear strength criterion · Critical state · Intact rock · Principal stresses

List of Symbols

- c Apparent cohesion (MPa)
- c_0 Apparent cohesion when $\sigma_n = 0$ (MPa)
- c' Derivative of c with respect to σ_n
- m Material parameter in Hoek–Brown criterion
- σ_1 Major principal stress (MPa)
- σ_3 Minor principal stress (MPa)
- σ_c Uniaxial compressive strength (MPa)
- σ_n Normal stress on potential shearing plane (MPa)
- σ_t Uniaxial tensile strength (MPa)
- ϕ Apparent internal friction angle
- ϕ_0 Apparent internal friction angle when $\sigma_n = 0$
- ϕ' Derivative of ϕ with respect to σ_n
- ϕ^* Inclination angle of the tangent slope of the strength envelope

1 Introduction

It is well-known that the dependence of shear strength of rocks on the normal stress acting on the potential shear plane is nonlinear, and that rocks are weaker than predicted by the traditional linear Coulomb shear strength criterion, especially when the normal stress is large. There are many nonlinear shear strength criteria in the literature, such as the Barton criterion (Barton 1976, 2006, 2013) and Hoek–Brown criterion (Hoek and Brown 1980a, b, 1988). Barton (2013) presented a summary of the nonlinear shear strengths for intact rocks, fractured rocks, jointed rocks and rockfills. As with most of these laboratory-tested geologic media, shear strength envelopes in the τ–σ_n plane are convex in relation to the normal compression stress axis, where τ is the shear strength and σ_n is the normal stress on the shear plane.

Based on many high-pressure triaxial experiments in the literature, including Mogi (1966) and Byerlee (1968), Barton (1976) proposed a critical state concept for rocks. At the critical state, the tangent of the shear strength envelope approaches horizontal in the τ–σ_n plane (Barton 2006, 2013). For confining pressure greater than the critical value, which is close to the unconfined compressive strength, the shear strength will not increase anymore. The value of peak shear strength is half of the normal compressive stress. The proposed non-linearity and reasons for such opinions are indicated in Fig. 1.

Singh et al. (2011) incorporated the critical state concept into a modified triaxial strength criterion for intact rocks expressed in a relation of σ_1–σ_3 and σ_3 and, with a large set of data, showed that the critical confining pressure is approximately equal to the UCS, which agrees with Barton's suggestion from 1976. Recently, Shen et al. (2018) further investigated the critical state concept for rocks and proposed a simple modified nonlinear shear strength criterion, which is of the classical Coulomb criterion form, but with the cohesion and internal frictional angle depending on the normal stress. It covers both compression and tensile regimes of failure. In this technical note, we first show graphs of the nonlinear shear strength using four sets of parameters, and then develop an approximate conversion of the shear strength criterion proposed by Shen et al. (2018) into one in terms of $\sigma_1 \sim \sigma_3$. It is noted that the critical state concept for rock failure has also been employed by others, for example, Carroll (1991) and Baud et al. (2006), as reviewed by Wong and Baud (2012), with a different form from the one we proposed.

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2019

ISRM 14th International Congress of Rock Mechanics – Foz do Iguassu, Brazil, 2019 - Rock Mechanics for natural resources and infrastructure development

The Q-Slope Method for Rock Slope Engineering in Faulted Rocks and Fault Zones

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1 INTRODUCTION

The Q-system for characterizing rock exposures and drill-core, and for estimating single-shell support and reinforcement needs in tunnels, caverns and mine roadways has been widely used by engineering geologists and mining engineers (Barton et al. 1974). In the last ten years, a modified Q-system called Q-slope was tested by the authors (Barton & Bar, 2015; Bar & Barton, 2017), for application in road cuttings, motorway cuttings, and in open cast mines. The purpose of Q-slope is to allow engineering geologists, rock engineers and mining engineers to rapidly assess the stability of excavated rock slopes in the field, and make optimal adjustments to slope angles as rock mass conditions become visible during construction of the road cuts or benches. Trials at several civil engineering and mining projects in Asia, Australia, Central America and Europe have shown that a simple correlation exists between Q-slope values and the long-term stable and unsupported slope angles. The new method includes Jr/Ja ratios for both sides of potential wedges, using relative orientation weightings. Slope-relevant strength reduction factors (SRF) are also applied. This paper focuses on the application of Q-slope when dealing slopes with major discontinuities, faulted rocks or fault zones.

2 METHODOLOGY

The relationship between Q-slope and long-term stable slope angles is now supported through over 500 case studies. This paper investigates existing and additional case studies pertaining to major discontinuities, faulted rocks and fault zones in which complex failure mechanisms can be expected. In these cases, the significance of SRFc (strength reduction factor) for major discontinuities becomes apparent.

3 RESULTS AND CONCLUSION

The case studies presented in this paper help to illustrate the ease and logic of applying Q-slope to rock slope engineering problems in the field, as geological and geotechnical conditions become apparent with the progression of excavations. The ability to rapidly assess and act on deviations from expected ground conditions in faulted rocks and fault zones makes Q-slope a powerful tool for engineering geologists and rock engineers in the field.

Notwithstanding the above, it is not our intention to promote Q-slope as a replacement for more detailed and rigorous slope stability analysis in situations where these are warranted or when time permits.

KEYWORDS: Q-slope, Q-system, rock slopes, slope stability, empirical method.

2019

Highest Mountains Suggest Strong Curvature of Shear Strength Envelopes for Rock

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ABSTRACT: The apparent 8 to 9km height limit of mountains will be addressed using critical state shear strength arguments, since confined compression strength is too high to explain these 'limited' heights. Modified Mohr Coulomb criteria have been derived based on critical state mechanics for rocks. These criteria are utilised to obtain estimates of maximum shear strength which actually is more likely to govern the height limit of mountains.

1 INTRODUCTION

There are fifteen mountains in the world with heights in the rarified range of 8 to 9km. The highest is Everest at approximately 8,848m. An extract from a Wikipedia photograph is shown in Figure 1. Since we are concerned with the ultimate strength of rock one can pose the question: why are the highest mountains no higher than 9km? Have mountains ever been higher than this during the earth's history? Since plate tectonics have been at work for a very long time, and contrary glacial processes also, one can perhaps assume that the extensive 'empirical evidence' that we see today is also a reflection of what has been in the recent and distant past. The strength of rock has little reason to have changed either, although it could be higher 'today' if the geothermal gradient had declined significantly.

In a well-known article written by Terzaghi (1962) near the end of his career: '*Stability of steep slopes on hard unweathered rock*', a simple formulation of critical slope height was suggested: $H = q/\gamma$, where the uniaxial strength of rock and the vertical stress caused by its density are compared. The assumed vertical stress is estimated to be γH (or $\gamma H/100$ if using familiar MPa units as in rock mechanics). One can also use units kN/m^2 and kN/m^3 for the rock strength and density. Concerning the height of steep slopes, as opposed to mountains, Terzaghi suggested that the reason this formula over-predicted heights must be due to the presence of jointing. In fact, an explanation of cliff and mountain-wall heights has recently been developed as $H = \sigma_t/\gamma\nu$, involving the tensile strength of intact rock, density, and Poisson's ratio.

The even simpler 'Terzaghi' formula or its equivalent has been observed in use in internet 'chat sites' ('What rock strength is the highest mountain limited by?'). The UCS/density formula (unfortunately) appears to produce a 'realistic' height for the highest mountains, when using the uniaxial strength of strong rocks such as 250MPa. However, at 9km depth, the rock mechanics reality concerning compression strength will be the 2 to 3 times higher polyaxially *confined* compression strength, and we do not accept 20-25km high mountains as being possible on earth, with our strong gravity. We need another explanation for the height 'limit' of apparently, about 9km.

Formation of high mountains due to colliding of plates in tectonically active region can be explained as shown in Fig. 2 (Nedoma, 1997). It was suggested that due to tectonic forces the lower plate bends downwards and releases horizontal tectonic stress. The horizontal stress becomes minor principal stress near the thrust. There is subsidence and normal faulting in the lower plate. The upper plate bends upwards and introduces compression and higher tectonic stress. The horizontal stress becomes major principal stress. This region experiences continuous uplifting in the form of mountains. The boundary of colliding plates experiences thrust faulting. The bottom part of the upper plate also bends upwards, due to which tangential stresses are released. Decrease in confining stress results in reduction of melting temperature of the rock and the rock melts. This molten rock may come out in the form of volcanoes.

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Chapter 18
Rock Mass Classification of Chalk Marl in the UK Channel Tunnels Using *Q*

Nick Barton and Colin Warren

18.1 Introduction

The Channel Tunnel was driven in chalk marl with the prior expectation by the designers of quite ideal tunnelling conditions on the UK side. This expectation was partly the result of little emphasis on the implications of joint structure. As a result of the difficulties and initial delays caused by overbreak in some of the UK sub-sea TBM drives, the first author was requested to assess the rock quality in existing tunnels in chalk marl. The work was performed during 1990 and 1991 under contract to GeoEngineering who were conducting a major review for Eurotunnel. The assessment was made using the *Q*-system of rock mass classification (Barton et al. 1974) which was also being used by TransManche Link (TML) in the Marine Service and Running Tunnels. The first author's classification of the grey chalk at Shakespeare Cliffs and of the chalk marl in the Beaumont and Terlingham Tunnels was performed prior to any data being provided on conditions in the Marine Service Tunnel (MST) or in the Marine Running Tunnel (MRT). The PB series of core logs and photographs for marine drill core PB1 to PB8 was also classified without prior knowledge of MST or MRT conditions. The extensive MST and MTR *Q*-logging by TML was subsequently made available by the second author of this paper, who was Eurotunnel's chief geologist. The comparison of multiple parties' *Q*-logging was satisfactorily close.

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RESEARCH GATE Q/A 2019:

Has biased academic and commercial marketing hidden basic problems of description and modelling of rock masses in the last several decades?

In 2003 the writer posed some questions for the ISRM journal editor. Many seem in need of repeating. A different format and some additions will be tried here.

1. Why can we monitor the progressive failure of slopes, and pillars, and over-stressed volumes underground?

2. Is it because the strength of rock masses is described by linear Mohr-Coulomb or non-linear Hoek-Brown/GSI?

3. Do such models of 'c plus sigma n tan phi' (linear or non-linear) realistically describe where shear failure is occurring around an over-stressed opening (e.g. the classic URL mine-by)?

4. Is the development of GSI ('to replace RMR') the first time we can inspect rock masses? (Recent Canadian university authors - clearly with journal reviewer's and co-author acceptance - described GSI as follows: 'After decades of relying on empirical classification systems to assess rockmass quality and ground support prescriptions, a rockmass characterization system that depends on direct geological field observations was created: the Geological Strength Index GSI').

5. Do we / did we perform 'direct geological field observation' when using the Q-system and RMR (in the last 45 years)?

6. Is GSI more 'geological' or 'observational' than RMR or Q?

7. Do any other serious scientific professions combine picture recognition and multiple opaque equations to estimate their key parameters?

8. What happens to the H-B c, φ, 'compressive' strength, and deformation modulus if there was one more joint set and this had clay filling?

9. We can monitor the progressive failure of over-stressed slopes, pillars, mine-volumes because rock masses *do not fail* by exceeding the addition of cohesional and frictional strength.

10. We can model where shear failure is occurring *by not adding* cohesion and friction, but rather by degrading cohesion and mobilizing frictional strength, up to peak and down towards residual.

11. Rock masses reach ultimate failure after exceeding the strength of (maybe) four components, each mobilised at different shear strains or displacements.

12. The components are (probably but not always) failure of intact rock (clearly includes stock-work and welded veins: they reduce the representative UCS), shearing of the new fractures, shearing of appropriately oriented joints, and maybe shearing of the lower resistance filled discontinuities (which often form one side of a large instability).

13. If one was able to be present without getting killed it might be heard as CCSS: crack, crunch, scrape, swoosh. (One may smile, but this is seriously meant).

14. It is more than 50 years since Müller, 1966 (and Rocha) regretted that we did not know how to formulate the shear strength of rock masses. Müller suggested, as here, and as done by several colleagues in the last two decades, that after cohesion was broken friction remained. *'The deformation resistance of the material bridges takes effect at much smaller deformations than the joint friction: this joint friction makes partly up for lost strength'.*

15. We should *not* be adding c and $\sigma_n \tan \varphi$.

16. Recently the writer has demonstrated that cliffs or mountain walls in massive rock do not have heights limited by Coulomb (c and φ). These parameters over-estimate heights by factors of 3 to 6 times (lower and upper-bound soil-based solutions for vertical cuts). But tensile strength, Poisson's ratio (and density) give correct results – from 10m to 1,000m.

Apparently Leonardo da Vinci (1452-1519) once gave advice that was distinctly helpful to one starting out in a relatively undeveloped field: 'If you find from your own experience that something is a fact and it contradicts what some authority has written, then you must abandon the authority and base your reasoning on your own findings'. Let's start over and make progress in the next 50 years.

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UNDERGROUND NUCLEAR POWER PLANTS CONSIDERING ROCK ENGINEERING PRECEDENT

By Nick Barton

ABSTRACT

Various rock engineering developments over the last decades have made the use of very large engineered rock caverns feasible as a method for developing underground nuclear power plants. Early site investigations in Norway in 1971 for potential UNPP, were followed by pre-UDEC physical models in 1976 and 1977 with tens of thousands of blocks formed by joint-simulating sets of intersecting tension fractures. The objective was the simulation of 50m spans in jointed rock. This was followed a decade later by Norwegian construction of the 62m span cavern for the winter Olympics in 1994. The Gjøvik cavern measures 62 x 24 x 90m and was constructed in 7 months. It is supported with systematic rock bolts and just 10cm of fiber-reinforced shotcrete. This cavern, despite its moderate rock quality Q from 2 to 30, RQD from 60 to 90, remains by far the largest engineered span for public use. However, the large span is dwarfed in another direction by the 80 to 90 m heights of a very few of the world's hydropower caverns. These are all located in China. Underground siting of nuclear power plants of a variety of potential sizes, presents obvious safety enhancement in relation to the earthquake, terrorist, and tsunami risks of surface plants. Rock engineering is clearly not one of the limitations for UNPP.

Keywords: caverns, rock quality, deformation, numerical modelling, nuclear power,

INTRODUCTION

By chance the author's first on-site job in a 50 years career was related to *the planned underground siting of a full-scale nuclear power plant*. This job was assigned a few months after arriving in Norway to work in the Norwegian Geotechnical Institute (NGI, Oslo). The year was 1971, and the first potential site identified by the Norwegian State Power Board (today called Statkraft) was at Brenntangen, in good quality gneiss, on the east side of the Oslo fjord. Some of the field testing performed was described by Di Biagio and Myrvoll, 1972 and more briefly by Barton, 1972, in an international conference in Stuttgart: *Percolation through Fissured Rock*. Norway already had two small underground research plants in Kjeller, and in Halden. These had been used in European reactor research and in medical research studies. They have since been decommissioned after approximately 60 years of operation.

The first task at Brenntangen was to extract details from borehole permeability measurements in an inclined hole that was part of the initial site characterization of this potential location. This was followed by other investigations, including tracer tests. The hope was to find less jointing and lower permeability as depth increased, which would help with decisions of how deep to site the largest caverns, including the need for a 50m span reactor cavern, if this site option was to be chosen.

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Deep Tunnels, Cliffs, Mountain Walls and Mountains: An Exploration of Failure Modes in Rock and Rock masses

Duboki tuneli, klifovi, litice i planine: istraživanje modela loma u stijenama i stijenskim masama

Nick BARTON[1]

Abstract

In relation to soil, rock is usually extremely strong, with a compression strength that will seldom be mobilized, even in deep tunnels. Intact rock may also have cohesion that is so high that it makes mountain avalanches rare events. Frictional strength tends to be high as well, due to the big contribution of dilation unless the rock has high porosity. The weakest link of the intact rock is of course it's tensile strength. It is realized now that Poisson's ratio also plays a major role in failure, as even rock under 3D compression can fail in tension due to the mechanism of extensional strain in the direction of a free surface. This is an important morphological property. Naturally if the rock is jointed, there are usually massive changes in strength and stability and slope height, in relation to slopes in intact rock. Failure may be progressive in nature, involving several components. In this paper all these aspects will be explored utilizing deep tunnels, and then the maximum heights of cliffs and mountain walls. The apparent 8 to 9km height limit of mountains will also be addressed using critical state shear strength arguments, since confined compression strength is too high.

Keywords: Extensional strain, shear strength criteria, deep tunnels, mountain walls, mountain heights

Sažetak

U odnosu na tlo, stijene su uobičajeno iznimno velike čvrstoće, s tlačnom čvrstoćom koja se rijetko dostiže, čak i u dubokim tunelima. Intaktna stijena također može imati i koheziju koja je toliko visoka da se planinske lavine događaju rijetko. I trenje teži visokim vrijednostima zbog velikog doprinosa dilatancije, osim u slučaju stijena visoke poroznosti. Najslabija karika intaktne stijene je, naravno, njezina vlačna čvrstoća. Prema novijim saznanjima, i Poissonov omjer ima važnu ulogu u pojavi loma, jer se čak i u stijeni u uvjetima 3D kompresije može dogoditi vlačni lom uslijed mehanizma razvoja vlačnih deformacija u smjeru slobodne površine. To je važno morfološko svojstvo. Naravno, ako su u stijeni prisutni diskontinuiteti, uglavnom postoje znatne promjene čvrstoće, stabilnosti i visine padine u odnosu na padine u intaktnoj stijeni. Lom po prirodi može biti progresivan, uključujući nekoliko komponenti. U ovom radu će se istražiti svi ovi aspekti koristeći duboke tunele, a potom i najveće visine klifova i litica. Očigledno ograničenje visine planine na 8 do 9 km također će se obrazložiti koristeći argumente kritičnog stanja posmične čvrstoće, budući da je tlačna čvrstoća u uvjetima spriječenog širenja previsoka.

Ključne riječi: Vlačna relativna deformacija, kriterij posmične čvrstoće, duboki tuneli, litice, visine planina

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Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

Understanding the need for pre-injection from permeability measurements: What is the connection?

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ARTICLE INFO

Article history:
Received 7 August 2018
Received in revised form
5 December 2018
Accepted 24 December 2018
Available online xxx

Keywords:
Pre-grouting
Micro-cement
Hydraulic apertures
Physical joint apertures
Joint roughness
Particle size
Anisotropic permeability

ABSTRACT

Pre-grouting ahead of tunnels has three main functions: to control water inflow into the tunnel, to limit groundwater drawdown above the tunnel, and to make tunnelling progress more predictable since rock mass quality is effectively improved. It helps to avoid settlement damage caused by consolidation of clay deposits beneath built-up areas, since towns tend to be built where terrain is more flat, due to the clay deposits. There are so many instances of settlement damage that the profession needs to take note of the need for high-pressure pre-grouting, to use micro-cements and micro-silica additives. The use of high-pressure injection may cause joint jacking, but this is local in extent when the rapid pressure decay away from an injection hole is understood. This effect is variable and depends on the geometrical parameters of the joints. This pressure-decay advantage must not be violated by maintaining high pressure when grout flow from the injection hole has ceased. The latter can cause damage to the grouting already achieved. Simplified methods of estimating mean hydraulic apertures (*e*) from Lugeon testing are described, and from more sophisticated three-dimensional (3D) permeability measurement. The estimation of the larger mean physical joint apertures (*E*) is based on the joint roughness coefficient (JRC). Comparison is then made with the empirical aperture-particle size criterion $E > 4d_{95}$, where d_{95} represents almost the largest cement particle size. Depending on joint set orientations and on the available micro-cements, the decision must be made of which range of pre-injection pressure should be aimed for, using successive reductions of the water-cement ratio *w/c*. More simple estimation of permeability, also with depth dependence, can be made with the empirical link between a modified rock mass quality Q and permeability, which is termed Q_{H2O}. The value of this parameter can be based on core-logging or in-tunnel face logging. The 3D before-and-after-grouting permeability measurements have been used to justify the quantification of rock mass quality Q-parameter improvement, and the consequent increases in expected P-wave velocity and deformation modulus, for application in dam foundation treatment and its monitoring.

1. Introduction

As is well known to all working in rock mechanics, the behaviour of rock masses, whether shear strength or deformability or the shear strength and permeability of the component rock joints, has been a life-long interest and pre-occupation of Professor Ted Brown, in between his remarkably active academic and professional career, and his strong involvement in mining and dam engineering in particular.

In this paper, the writers have technically related interests. By focussing on pre-grouting, we simultaneously address practical ways of *improving the properties* of rock masses where strength, permeability and deformability are considered to be inadequate for problem-free drill-and-blast and tunnel boring machine (TBM) tunnelling, and presently inadequate for the desired range of properties in dam foundations where relative impermeabilization (somewhat lowered Lugeon values), increased modulus, reduced uplift pressures, and stable abutments are the principal goals.

With a long-standing rule for injection pressure gradients of approximately 0.23 bar/m depth (1 bar = 0.1 MPa) for dam foundation grouting in the USA, but usually higher elsewhere (Quadros and Abrahão, 2008), it is clear that there will be reactions when

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

https://doi.org/10.1016/j.jrmge.2018.12.008

Please cite this article as: Barton N, Quadros E, Understanding the need for pre-injection from permeability measurements: What is the connection?, Journal of Rock Mechanics and Geotechnical Engineering, https://doi.org/10.1016/j.jrmge.2018.12.008

2019

© International Society for Rock Mechanics and Rock Engineering
Norwegian Group for Rock Mechanics
ISBN: 978-82-8208-072-9
C.C. Li, H. Ødegaard, A.H. Høien, J. Macias (Eds.)

ISRM International Symposium
Eurock 2020 – Hard Rock Engineering
Trondheim, Norway, 14-19 June

Q-Slope addressing ice wedging and freeze-thaw effects in Arctic and Alpine environments

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Abstract

Q-slope is an empirical rock slope engineering method for assessing the stability of excavated rock slopes in the field. Intended for use in reinforcement-free road or railway cuttings or in open cast mines, Q-slope allows rock engineers to make potential adjustments to slope angles as rock mass conditions become apparent during excavation.

Q-slope was developed over the last decade by modifying some of the Q-parameters so that rock-cuttings and bench faces could be characterized. Drill core and seismic velocity can still be used as supportive input. The original Q-value has traditionally been used for estimating single-shell support and reinforcement needs in tunnels, caverns and mine roadways and access ramps. A simple correlation between Q-slope and long-term stable slopes was suggested five years ago. Through over 500 additional case studies from Asia, Australia, the Americas and in Europe, Q-slope has been confirmed as giving stable, maintenance-free rock-cuttings and bench-face slope angles of for instance 40-45°, 60-65° and 80-85° with respective Q-slope values of approximately 0.1, 1.0 and 10.

Assessing rock slope stability in arctic and alpine environments brings its own challenges both during the peak of winter when ice building in joints can result in wedging or jacking, and in the pre- and post-winter seasons when cyclic freeze-thaw effects often degrade the quality of the rock mass. Modelling such processes using numerical techniques is possible to some extent; however, it is impractical as a routine application.

This paper discusses the use of the Q-slope method as a means of appraising rock slope stability in environments susceptible to ice wedging and freeze-thaw effects.

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perfectly horizontal. This does not happen when the direction of the vertical principal stress is parallel to the axis of the borehole. In one borehole in the field, a 300 meter borehole, for example, you may run something like 6 tests to get a good value of the stresses. Now, if the results of the impressions are such that the fractures are always within a very small range of directions, let's say within plus or minus 20° or so, as happened in the cases cited in the paper, we feel very strongly that this is a vertical fracture that represents vertical and horizontal principal stresses.

In the case of Helms Creek Power House in California we were required to run tests in an inclined hole in addition to the tests in the vertical hole. The tests in an inclined hole at 30° to the vertical confirmed, as is stated in the paper, that one of the principal stresses was parallel to the vertical borehole axis. I can also report that the Los Alamos Scientific Laboratories have conducted probably the most elaborate type of hydraulic fracturing in conjunction with a geothermal energy project. They were able, at a depth of, I think, 6,000 ft. below surface, to determine that, although the direction of the well at that depth was something like 4° off the vertical, the direction of the hydraulic fracture was perfectly vertical.

Lindner

First off, I'd like to thank Mr. Pacher for his comments; we appreciate them. I would like to add a few points. Exploration is generally performed in stages. However, in each stage, even in the final stage, there is uncertainty. There is uncertainty in the knowledge of the geology below the surface. There is uncertainty when we predict the cost and time of the project, in the manpower and equipment and how they will perform in that environment. With a computer model, with the tunnel cost model that we have developed, we can evaluate these uncertainties to come up with a time-cost spread for the project. The time-cost spread quantitatively illustrates the uncertainty. In other words, the project time has a range, and the project cost has a range. You can see that for a specific case study on Fig. 4 of our paper. Exploration has the purpose of reducing the uncertainty in your time-cost projection.

However, in using exploration tools and performing exploration, there is additional uncertainty. Even the knowledge we get directly out of a borehole is open to question. This will again generate a degree of uncertainty about our knowledge of what is down there. The method proposed in our paper uses common probabilistic techniques, I emphasize the word 'common', to analyze these uncertainties, using both statistical input and subjective input from the geologist on the job, because he probably has a better feeling of what is down there from his total experience with the site than the bare data retrieved from a few boreholes can tell us. The computer method gives you a quantitative assessment of what is down there. It can show you, and this perhaps is the major point of the paper, how to compare the cost of exploration with the reduction in the time-cost uncertainty for the project.

Panek

The design of a tunnel lining is greatl dependent on the assumptions that one is forced t use, both with respect to the characteristics c some proposed liner, and with respect to th characteristics of the rock that one envisions t have supported. Much of the confusion with line design is brought on by designers having preconceived idea of what kind of a liner they ar going to use. The load that is brought on the line in a large measure is caused by the liner itsel not just the rock. So we made an attempt to fir out, free of any a priori choice of liner desigr what one might be able to do in the way c innovative tunnel support by simply asking th question, what kind of distribution of pressur against the wall is required to prevent the mos simple kind of failure? This situation we conside to be well represented by the triaxial test t determine Mohr's envelope.

SESSION 3 - GENERAL DISCUSSION

Question by Gerdeen (for Barton, Lien and Lunde)

The authors are to be congratulated on a ver interesting and potentially useful paper.

Our work has been mainly with roof bolting ar so my question deals with this method of support Would you explain the difference between categorie 10 and 11 in Table 8 where the quality indices fo block size are the same, but where different type of bolting are recommended? It seems to me, yo favor tensioned mechanical bolts over untensione grouted bolts for larger spans. The question i why? Are there not cases where tensioning may caus additional damage? Does your analysis apply as wel to flat horizontal roofs or only to arches?

Reply by Barton

The two support categories referred to in Mr Gerdeen's comments were categories 10 and 11, ar you can see from Fig. 3 that between these tw categories there is no change in the rock quality The rock quality ranges from 40-100 in each case That's pretty good rock. Some people would say it' the best rock you ever get, but that's not ou Scandinavian experience. So we've just got difference in span and/or in the use of th excavation. You can say the EST (excavation suppor ratio) which describes the type of excavation make the effective span for category 10 less than tha for category 11.

I am in full agreement with your point on th application of tension bolts as opposed to groute bolts. If this paper were expressing purely th opinion of we three authors in NGI, we would have, think, in most cases have recommended just groute bolts. We have analysed approximately 200 cas records, or 200 usable case records, from which i appears that the general practice of some years ag 5 years ago let's say for an average constructic completion, was the use of tensioned bolts in thes

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ESTIMATION OF SUPPORT REQUIREMENTS FOR UNDERGROUND EXCAVATIONS

ESTIMATION DES SOUTÈNMENTS NÉCESSAIRES POUR LES EXCAVATIONS SOUTERRAINES

ABSCHÄTZUNG DES NÖTIGEN FELSAUSBAUES IM HOHLRAMBAU

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An analysis of some 200 case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to excavation stability. The rock mass quality Q is a function of six parameters,each of which has a rating of importance,which can be estimated from surface mapping and can be updated during subsequent excavation. The six parameters are as follows: the RQD index,the number of joint sets,the roughness of the weakest joints,the degree of alteration or filling along the weakest joints,and two further parameters which account for the rock load and water inflow. In combination these parameters represent the rock block size,the interblock shear strength,and the active stress. Analysis of the rock mass quality and corresponding support practice has shown that suitable permanent support can be estimated for the whole spectrum of rock qualities. Support measures include various combinations of shotcrete,bolting,and cast concrete arches together,with the appropriate bolt spacings and lengths,and the requisite thickness of shotcrete or concrete.

Une analyse de données provenant de quelque 200 cavitées creusées a permis d'établir une relation utile entre, d'une part,l'envergure et le type de soutènements et,d'autre part,la qualité Q des masses rocheuses,en ce qui concerne la stabilité. La qualité Q de la roche est une fonction de six paramètres dont chacun,dans des échelles données,s'est vu attribuer un coefficient pondéré déterminé qu'on peut estimer en se basant sur des observations faites en travaillant à ciel ouvert et qui pourra être ajusté et mis à jour au cours de l'avancement des travaux. Ces paramètres sont:l'indice RQD,le nombre de systèmes de fissuration,la rugosité(celle du plus faible plan de fissuration),le degré d'altération (caractéristiques de ce dont les fissures sont remplies),et,en outre,deux paramètres qui tiennent compte du niveau de tension et de l'afflux d'eau. Dans leur ensemble,ces paramètres représentent l'influence qu'exercent la grandeur des pierres,la résistance au cisaillement existant sur les surfaces de contact entre les pierres,et les tensions actives. Des analyses de la qualité,accompagnée d'une prise en considération de la pratique de soutènement utilisée,ont permis de démontrer qu'il est possible d'estimer un soutènement approprié pour toute la variété de qualités de roche. Les mesures de sûreté englobent différentes combinaisons de béton projeté,de boulonnage et d'arcs en béton coulés,accompagnées de l'indication de la distance appropriée entre boulons,de la longueur de ces derniers et de l'épaisseur à respecter tant pour le béton projeté que pour le béton coulé.

Eine Untersuchung von Daten aus etwa 200 fertiggestellten Tunnelbauten ergab einen nutzbaren Zusammenhang zwischen Umfang und Typ des permanenten Verbaues und der Gebirgsqualität Q. Die Gebirgsqualität Q ist eine Funktion von sechs Parametern,die aus Oberflächenbeobachtungen und nach skalierten Gewichten bestimmte Leitziffern erwerfet werden.Die Werte können während des Bauvortriebes justiert werden. Die sechs Parameter sind: RQD-Leitziffer,Anzahl der Kluftsysteme,Rauhigkeit (für schwächste oder ungünstigste Spaltebene),Umwandlungsgrad (Charakter der Risse oder Füllung längs der schwächsten Spalten) und des weiteren zwei Parameter,dei Spannungsniveau und Wasserzufluss berücksichtigen. Wenn man diese Parameter koordiniert,vertreten sie den Einfluss der Körnung,der Scherfestigkeit an den Anschlussflächen zwischen den Felsblöcken und der einwirkenden Spannungen. Analysen der Gebirgsqualität und der entsprechenden Sicherungsmassnahmen haben erwiesen,dass es möglich ist, einen angemessenen Ausbau fürs ganze Spektrum der Gebirgsqualität zu veranschlagen. Die Sicherungsmassnahmen umfassen verschiedene Kombinationen von Nägeln, Ankern, Spritzbeton und Ortsbetongewölben sowie auch Angaben über Ankerabstände und erforderliche Stärke des Spritz- oder Gussbetons.

INTRODUCTION

Two important factors for the stability of underground excavations are their location and orientation relative to unfavourable geological conditions. Both factors are weighed to minimise difficult rock conditions for the case of large span openings of limited length. However there is little opportunity to choose the orientation of tunnels,and generally only the location can be changed significantly. The amount of support required will be strongly dependent on orientation if poor rock conditions are encountered.

Estimates of support are required at three stages in a project: for the feasibility studies,for the detailed planning,and finally during excavation itself. In view of the economic importance of support costs it is vital that the support estimates are as accurate as possible for all three stages. The accuracy will depend partly on the success of the geological investigations,and partly on the success of extrapolating past experiences of support performance to new rock mass environments.
When beginning this work of support estimation a literature survey directed towards related excavations

2020

Feedback from: **Nick Barton, Independent Consultant**

Dear TunnelTalk,
It is always interesting to read of tunnel failures, from which we all learn. I have two points in response.

In his interesting and well-illustrated review, Brox has suggested using "other than rock mass classification methods" for application of final support and linings. There is nothing wrong with this suggestion except that it ignores the contribution of such methods to thousands of kilometres of hydropower tunnels and considerable cost savings for owners. If mistakes are made in application of the methods due to oversight, especially in the case of faulted rock, then lessons need to be learned by those involved.

Incidentally the Ituango case, the failure erosion cone is much larger than described and is a special case of optimism in diverting water with a peak velocity of up to 10m/sec around a remarkably tight bend, both of which are entirely different to the typical 1.5-2.5 m/sec velocities in the case of hydropower tunnels. Design for velocities of 1.5-2.5 m/sec have been and are the basis of Q-system case records. With Eda Quadros, I have prepared a paper for Eurock 2020 titled ***Some lessons from single-shell Q-supported headrace and pressure tunnels*** that may not now be presented due to Covid-19 postponements but may become available in published proceedings of Eurock 2020 planned for 15-19 June 2020 in Trondheim, Norway.

Secondly, Brox recommends independent checking of waterway tunnel designs. I agree that this could, in principle, be valuable. I have reservations however based on what is available outside of the use of more careful rock mass characterization and use of empirical methods, like the Q-system which probably has the most relevant database. It should not be forgotten that there are thousands of kilometres of such waterways, and many hundreds (actually thousands) of economic projects as a result of the single-shell type of support, which, as pointed out by many, needs to consider the intended use of the tunnel.

As indicated above, if a water velocity, as in the case of a river diversion, is chosen by a designer that is well outside the database (for example 10m/sec as compared with a conventional 2m/sec velocity), then one is asking for potential trouble, if the tunnel support, also of the invert, is not dimensioned accordingly.

2020

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

A review of mechanical over-closure and thermal over-closure of rock joints: Potential consequences for coupled modelling of nuclear waste disposal and geothermal energy development

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ARTICLE INFO

Keywords:
Joints
Closure
Thermal effects
Permeability
Velocity
Modulus

ABSTRACT

Rough joints can *over-close* due to a prior higher stress, or due to temperature increase alone. There is better fit of their opposing walls causing increased friction and even tensile strength. Well-controlled laboratory HTM tests, *in situ* HTM block tests, and large-scale heated rock mass tests, lasting several years at Stripa, Climax and Yucca Mountain, have produced likely evidence for this coupled response, which is different from pressure solution. Rock mass deformation moduli, thermal expansion coefficients, hydraulic apertures, shear strength, and seismic velocities can each be affected. In the cooling phase of an HLW repository, and in a geothermal project, rougher joints may be thermally over-closed, and cooling causing contraction effects may be focused where joints are more planar, causing shear and fluid capture.

1. Introduction to over-closure using physical models and rough tension fractures

The writer's first experience of over-closure of rough fractures was during (ambient temperature) research at Imperial College, fifty years ago. Several dry, two-dimensional-slice, model 'rock slopes' excavated in 40,000 block tension-fracture models, with both horizontal and vertical stress, would not fail at the expected slope angles. Fig. 1 illustrates two of these models. The 'proving-ring-and-dial-gauge' loading was applied at both boundaries. It was found that the rough fractures could be *over-closed* and remain over-closed by the previous application of the higher normal stress acting prior to any slope excavation. Direct shear tests on over-closed tension fractures also showed higher strength if *previously loaded* to a higher normal stress than applied in the subsequent direct shear test.

Conventional 1.1, and over-closed 4:1 and 8:1 direct shear tests, with a prior normal stress higher than the normal stress applied in the following DST of the same rough fracture, showed successively steeper shear strength envelopes, and thus explained the reluctance to fail. (Barton, 1971).

Mistaken application of a higher-than-planned normal stress in a direct shear box test of the shear strength of a rock joint in the Engineering Geology department at Imperial College, reportedly resulted in the need to wedge open the joint (pers. comm. Dr. Mike DeFreitas, Imperial College, 1969) as the sample could not be sheared at the correct (lower) normal stress. One may also relate the true story of a tilt test on a tension fracture that we made for a rock mechanics course in a university with poor rock mechanics facilities some 20 years ago. The newly fractured block weighing some 2 kg, with the two halves placed together by self-weight, was slowly rotated by the author, thereby increasing the tilt angle (or dip) of the fracture. Surprisingly, a vertical dip angle was reached – most joint samples slide at 50° to 80° when their JRC value is in the typical range of 5 to 15 (Barton and Choubey, 1977; Barton and Bandis, 2017). This particularly rough tension fracture, with a JRC perhaps even higher than 25, tolerated continued tilting to a dip angle of 180° (i.e. pure over-hang). It now exhibited tensile strength due to presumed *frictional interlock*. This mechanism, repeated at much lower roughness, sets the scene for a new way to look at joint closure. A significant difference to 'asperity shortening' models will be seen.

Some years after the writer had moved from Imperial College to NGI in Oslo, a study for underground nuclear power plants was performed (Barton, 1972, 2019), using the same physical modelling tension fracture technique. This was performed just prior to Peter Cundall's development of the distinct (jointed) element UDEC. A 'demonstration' four-cavern model with 20,000 blocks was also performed at this time, which is illustrated in Fig. 2. Fracture *over-closure* was now demonstrated in more detail. The photogrammetrically recorded deformations across the face of the '2D' model were not reversed in the pillars when successive caverns were excavated. (Barton and Hansteen, 1979). The

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https://doi.org/10.1016/j.tust.2020.103379

2020

TunnelTalk *Direct by Design*

TunnelTECH

Rock mechanics and nuclear waste disposal

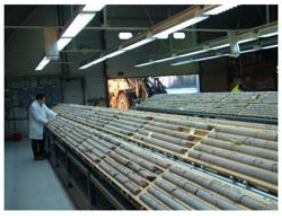
09 Jul 2020

Nick Barton, Independent Consultant

The study of rock mass behaviour and rock mass quality is essential in the site investigation and selection of suitable geological hosts for underground nuclear waste disposal facilities. **Nick Barton** discusses the topic based on his career in rock mechanics and his engagement, with colleagues and geotechnical companies, in research and characterization studies for planned nuclear waste repository siting in the USA, Canada, the UK and Sweden.

With a long rock mechanics background, there has been the opportunity to gain some detailed insight into various international nuclear waste related studies. These have been studies with a geological disposal facility focus, and have involved strong crystalline and strong volcanic rocks, and specifically with higher strength rocks with reference to the UK-studies reported by Marsh, Williams and Lawrence in the recent *TunnelTalk* focus on nuclear waste disposal.

Personal involvement in studies started in 1980 and ended about ten years ago but with a degree of time extension due to the presently running, post-Fukushima ISRM commission that concerns siting of nuclear power plants underground. This is chaired by Professor Sakurai, a past-president of ISRM, and follows an interest in underground siting of nuclear power plants in large caverns of more than 50 years ago.[1]



Many kilometres of cores add to the site investigation for suitable underground nuclear waste repositories

In the 1980s, the Office of Nuclear Waste Isolation (ONWI) in the USA funded a study by the geotechnical company TerraTek, to test several instruments in parallel to assess the in-situ performance of instrumentation when subjected to the heat generated by decaying nuclear waste in an underground repository. One task was to interpret the hydraulic behaviour of the first fully coupled hydro-thermo-mechanical in-situ block test of jointed rock (Fig 1). The test was performed on a heavily instrumented and flat-jack loaded, heated and flow-tested $8m^3$ of quartz monzonite in the Colorado School of Mines experimental mine in the USA.

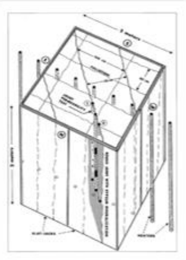


Fig 1. Mean jointing trends of the heated block test[2]

Strain and deformation gauges used for the HTM in-situ block test

ONWI subsequently funded the completion of a joint constitutive model, which was finalized in 1982 and used in a two-volume report with Bakhtar for the Atomic Energy of Canada company and the Canada Centre for Mineral and Energy Technology. This concerned potential model application in fractured parts of the underground research laboratory in Manitoba granite. The model for joint behaviour, which became known as the BB (Barton-Bandis) model, was subsequently incorporated in the numerical code UDEC, UDEC-BB, by Itasca and NGI in 1985.

2020

Leakage mitigation in tunnelling, with emphasis on karst

We read at intervals in excellent journals, or publish or lecture about what can go seriously wrong in our challenging underground media. It may be the occasional, easily explained, locally massive failures in single-shell tunnels, or the occasional, easily explained, locally massive failures in double-shell tunnels (or caverns under construction). The former are using the far more frequently used shotcrete and rock bolt methods (tens of thousands of kilometers of mine access, mine roadway, hydropower, economic road and rail each year), as compared to the less frequent shotcrete, bolts, lattice girders, smoothing layer, drainage fleece, membrane, cast concrete tunnels used in our more expensive transport tunnels. We could identify the methods used with various combinations of N's and T's and M's, and perhaps an occasional Q or an R - their geographic origins now masked by more variable use in many, many countries.

Instead of reading or hearing of a hydropower head-loss, or mine accident, or near-miss when driving, it is almost refreshing, though no less serious, to view the muddied train that exited the Lötschberg deep base tunnel recently. A challenging problem perhaps caused by an unknown thickness of immediately surrounding rock in karstic terrain, despite 400m depth, or somehow a serious increase in water pressure (quite feasible in karst caverns) or deformation-caused cracking, and then leakage – the list of possibilities emphasises the hydrogeological risks in our chosen profession, especially in the case of karst. It could on the other hand have been caused by measures taken during construction, that did not, with the benefit of hind-sight, provide a long-term solution.

But there are also risks with Choice of Method. The leakage of water, and mud, serves to warn us of the questionable long-life performance of an easily clogged drainage fleece, if it has to tackle a fluid different from water, or suffer gradual mineralization deposits. We then run in to the problem of the many kilometers of membrane welds per kilometer. (This can be a challenging 12 to 15km of welds/km in a big double-track rail tunnel with a perimeter exceeding 30m). We also have the radial joints in successive concrete pours, and their actual absence when using sufficiently thick longitudinally sprayed shotcrete There are no radial joints with S(fr) and it has extremely low permeability, in the panel-sprayed range of 10E-10 to 10E-12 m/s. This is 40 years-old data. The first Ph.D on S(fr) in Norway from Opsahl, was at the very end of the 1970's, with reporting in 1980, 1981.

So what are the best choices? Frankly it is not felt that the style of 'belts-and-braces' incorporated in double-shell is the right answer – even though when functioning well, it can be, though is expensive and time-consuming. What about another choice of 'belts-and-braces', namely one that automatically includes drilling through the crucial rock surrounding and ahead of the tunnel - clear advantages especially in karstic terrain. So an easily guessed single-shell but with rock mass improvement: systematic high-pressure pre-injection (but with pressure suited to the rock and rock mass). This b-and-b remains a very cheap solution. There are literally millions of kilometers of drill-holes used for the world's large dams: grout curtains are permanent and headed for a 100 years (plus) life. Pre-grouting umbrellas take 24 to 30 hours, and an almost guaranteed 20 hours plus per week full-face tunnel construction – as indeed for high-speed rail tunnels of > 100m2. Grouting is known, and 3D-measured, to cause rotation of the 3D permeability tensors, and their magnitude reduction, and significant modulus and velocity increase. In fact the six Q-parameters may be effectively improved. The risk during construction is reduced dramatically by drilling 1 to 1.5km of holes every 15 to 20 meters of tunnel advance. Eventual damp spots in the shotcrete signal where a local post-injection hole is needed. This is not the case with the double-shell solution.

Nick Barton, NB&A, May 2020

2020

TunnelTalk *Direct by Design*

TunnelTECH

Single-shell lining design and advantages

Feb 2020

Nick Barton

In taking a broad look at tunnel lining options, cost, construction time, and the CO_2 implications of concrete consumption are each of significant concern. For many decades a method has been in use that effectively minimizes cost and time and the use of concrete. The method is known as NMT, the Norwegian Method of Tunnelling. It is based on application of the Norwegian Q-system to design the rock support and emphasizes a single-shell shotcrete final and permanent lining. Its characteristics distinguish it from double-shell options that include final load-bearing and often steel-reinforced concrete linings as the permanent finish. Such a method is more expensive, uses more concrete, can take longer to build, and usually requires a larger labour force.

A common problem in underground excavation is the incidents of profile over excavation and drill+blast overbreak. Significant over break is inevitable if the Q-system parameters show a ratio of joint sets (Jn) to surface roughness (Jr) equal to or more than 6 (Fig 2). For example, in a situation of three joint sets and planar joints where the ratio is Jn 9 and Jr 1.5, over-break is common and increases the volume of concrete required to complete a double shell lining. When using the NMT single-shell S(fr) lining option, over-break is less of an issue. The area of the excavation perimeter is greater with over-break, but the over-break is not and should not be filled. The rock mass, assisted by systematic bolting and shotcrete, takes the major load.

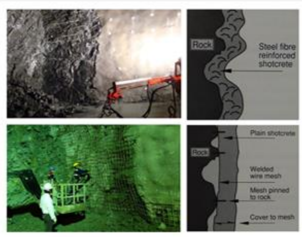


Fig 1. Top: Single shell shotcrete final lining with fibre reinforced shotcrete and pre-excavation grouting to control water inflow consists of an initial 5cm layer of S(fr) and permanent CT-bolts each applied close to the face Bottom: Wire mesh reinforcement demonstrates the difficulties caused by overbreak
(Credit: Right images courtesy Vandewalle[4])

Single-shell designs provided the final, permanent tunnel linings for road, rail, hydropower, water transfer, mine access and mine roadway developments as well as for large caverns for storage of oil or food, for hydropower machine and transformer halls.

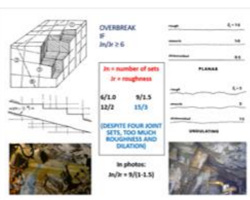


Fig 2. Inevitable drill+blast over-break particularly with clay-bearing joint sets

In each case, NMT based tunnels and caverns are made stable via application of the Q-system of rock mass quality estimation that encompasses a rock mass quality scale from 0.001 - equivalent to a serious fault zone, where NMT may also need a local concrete lining - to 1000 which is equivalent to massive unjointed rock where careful blasting can remove the need for shotcrete, but in practice is applied perhaps with one layer of S(fr).

In typical rock masses where tunnels and caverns are excavated, rock mass quality will lie on either side of mid-range or closer to Q = 1 which is described as poor quality with an associated mid-range bell-shaped distribution of RQD. These conditions would need combinations of corrosion-protected rock bolts and high-quality fibre-reinforced shotcrete, with stainless-steel or polypropylene fibres. Systematic high-pressure pre-injection grouting using micro-cement and micro-silica may be required to control water inflow to create essentially dry tunnels with limited inflow of approximately 1-2 liters/minute/100 linear metres of tunnel which can be dried out by the ventilation air in ventilated facilities. Systematic

2020

Some rock mechanics tasks in nuclear waste disposal projects

Nick Barton, NB&A, Oslo

During a long rock mechanics career the undersigned has had the opportunity to gain some insight into various national and international nuclear waste related studies. Personal and company involvement has been in the USA (several projects), Canada, Sweden (several projects) and a major site characterization in the UK.

A first field task when joining TerraTek in Salt Lake City for four years in 1980 was to assist and later help interpret the hydraulic parts of the first fully coupled HTM *in situ* block test, which was performed on a heavily instrumented and flat-jack loaded, heated and flow tested 8m³ of tough-to-core high strength quartz monzonite in the CSM mine in Colorado. This was funded by ONWI – the Office of Nuclear Waste Isolation. They contracted TerraTek to test many instruments in parallel, so that more confidence could be gained about the performance of instrumentation subjected to heating, used in various experiments. A glimpse of the principles is shown in Fig. 1.

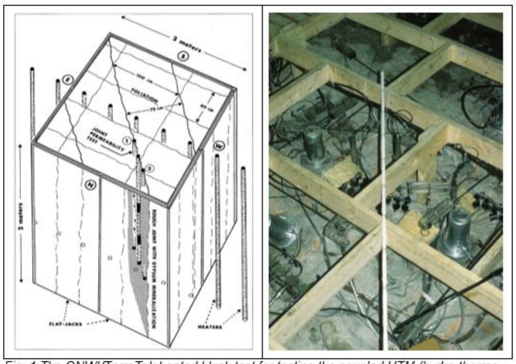


Fig. 1 The ONWI/TerraTek heated block test for testing the coupled HTM (hydro-thermo-mechanical) behaviour of a jointed crystalline rock. Mean jointing trends are shown. A range of strain and deformation gauges are seen. (Hardin et al. 1981, Barton, 1982).

ONWI subsequently funded the development of the Barton-Bandis joint constitutive model, which was finalized in Barton,1982, with the strong software-savvy help of Khosrow Bakhtar, and was immediately used to help illustrate a two-volume report for AECL and CANMET in Canada concerning its potential application in fractured parts of the Underground Research Laboratory in Manitoba granite. The BB model for joint

2020

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Norwegian Group for Rock Mechanics
ISBN: 978-82-8208-072-9
C.C. Li, H. Ødegaard, A.H. Høien, J. Macias (Eds.)

ISRM International Symposium
Eurock 2020 – Hard Rock Engineering
Trondheim, Norway, 14-19 June

Unconventional exploration of failure modes in rock and rock masses

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Abstract

This paper deals with the exploration of failure modes in rock and rock masses, starting with extension failure in deep tunnels, followed by analysis of the limited heights of cliffs, mountain walls and mountains. Here, tensile failure applies to the cliffs and mountain walls, since cohesive strength is too high, and shear strength applies to the maximum mountain heights since confined compression strength is too high. In each case it is the weakest link that applies, as in morphological processes.

The actual strength of rock masses is neither Mohr-Coulomb nor Hoek-Brown nor friction coefficient based, although the latter may be useful for describing the more linear residual strength of faults. We should not be adding '$c + \sigma_n \tan \varphi$' since these components are not mobilized in unison. Intact rock has a cohesive strength that is so high that it makes mountain avalanches rare events. Frictional strength tends to be high as well, due to the big additional contribution of fracture dilation. The weakest link of the intact rock is of course the tensile strength, and this is proved by cliff height limits in a wide range of rock types with heights varying by a factor of 100 depending mostly on tensile strength.

Thanks to recent work by Baotang Shen it is now known that Poisson's ratio plays a major role in initial failure, as even rock under 3D compression can fail in tension due to the mechanism of extensional strain in the direction of a nearby free surface. This is an important morphological property. At higher stress levels, the extensional fractures may propagate in shear.

A simple new cliff formula is demonstrated based on tensile strength, density and Poisson's ratio. Naturally if the rock is jointed, there are usually massive changes in strength and stability and slope height, in relation to slopes in intact rock. Failure may be progressive in nature, involving several components of strength which are mobilized at different shear displacements or strains. The stability of the famous Prekestolen in SW Norway will be assessed from a new viewpoint, considering several components of strength and including potential extension strain-based failure at its base. It's factor of safety may be different from that obtained by conventional shear strength analysis.

The apparent 8 to 9km height limit of mountains, of course lower than this below the immediate peaks, will be addressed using critical state shear strength arguments, since confined compression strength is too high. The strongly non-linear nature of shear strength is emphasised throughout. Non-linearity is stronger than Hoek-Brown. Maximum shear strength is numerically similar to UCS and this has probably confused popular analysis of height limits which have been based on UCS.

2020

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Trondheim, Norway, 14-19 June

Some Lessons from single-shell Q-supported headrace and pressure tunnels

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Abstract

Development of the Q-system has meant engagement in water transfer tunnels, hydropower headrace and pressure tunnels in many countries since 1974. The support requirements of single-shell tunnels, were initially dominated by Norwegian and Swedish hydropower projects. The Q-system data base was greatly expanded later, by Grimstad's incorporation of steel fiber reinforced shotcrete S(fr).

The economic advantages of single-shell tunnels for hydropower has made this form of water 'conveyance' very attractive in relation to more expensive concrete lined alternatives. There are tens of thousands of kilometres of single-shell or nominally 'unlined' tunnels, and all need sound design. Interesting controversies arise in occasional hearings and court cases. One side may demand concrete-lined tunnels, the other defends 'nominally-unlined,' with Q-system based support and reinforcement where needed. Once the question 'what about rocks in the turbines?' was even heard.

Empirical *a posteriori* experience, related to the theoretical laminar-flow paraboloidal 3D velocity distribution, and a glance at the Hjulström-Sundborg river-erosion diagram, should convince the wise designer that flow velocities need to be limited to about 1.5 to 2.5m/s so that no fallen rock blocks ever reach the 'rock trap', which will likely contain silt and sand and perhaps floating pumice, when a tunnel system is emptied for inspection and maintenance. Too high flow velocities in lightly supported river diversion tunnels, with too thin shotcrete, have on occasion had dramatic consequences.

Remembering the *a posteriori* origin of the Q-system it is wise for numerical modellers to think twice before proposing 'longer rock bolts'. Claims about deep 'plastic' zones when analysis methods are full of *a priori* assumptions and alarming opaque equations devoid of joint sets, inevitably fail to convince. Unavoidable overbreak caused by high Jn/Jr ratios, and full-scale roughness losses, will also be briefly addressed. Minimum rock stress greater than water pressure is of course fundamental as well.

Keywords

Headrace tunnels, flow velocities, shotcrete, roughness, head-losses

2020

nick barton

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Nick Barton & Associates
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rock mechanics   tunnelling   rock classification   TBM

TITTEL	SITERT AV	ÅR
Engineering classification of rock masses for the design of tunnel support N Barton, R Lien, J Lunde Rock mechanics 6 (4), 189-236	3821	1974
The shear strength of rock joints in theory and practice N Barton, V Choubey Rock mechanics 10 (1-2), 1-54	3011	1977
Fundamentals of rock joint deformation SC Bandis, AC Lumsden, NR Barton International Journal of Rock Mechanics and Mining Sciences & Geomechanics ...	1943	1983
Strength, deformation and conductivity coupling of rock joints N Barton, S Bandis, K Bakhtar International journal of rock mechanics and mining sciences & geomechanics ...	1746	1985
Review of a new shear-strength criterion for rock joints N Barton Engineering geology 7 (4), 287-332	1649	1973
The shear strength of rock and rock joints N Barton International Journal of rock mechanics and mining sciences & Geomechanics ...	1031	1976
Some new Q-value correlations to assist in site characterisation and tunnel design N Barton International journal of rock mechanics and mining sciences 39 (2), 185-216	992	2002
Experimental studies of scale effects on the shear behaviour of rock joints S Bandis, AC Lumsden, NR Barton International journal of rock mechanics and mining sciences & geomechanics ...	672	1981
Rock joints N Barton, O Stephansson Proceedings of International Symposium On rock joints, Loen, Norway, 4-6	575	1990
Rock quality, seismic velocity, attenuation and anisotropy N Barton CRC press	542	2006
An improved model for hydromechanical coupling during shearing of rock joints R Olsson, N Barton International Journal of Rock Mechanics and Mining Sciences 38 (3), 317-329	501	2001
TBM tunnelling in jointed and faulted rock NR Barton Crc Press	486	2000
Effects of block size on the shear behavior of jointed rock N Barton, S Bandis The 23rd US symposium on rock mechanics (USRMS)	380	1982
Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and some solutions N Barton Journal of Rock Mechanics and Geotechnical Engineering 5 (4), 249-261	301	2013

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SITERT AV	ÅR
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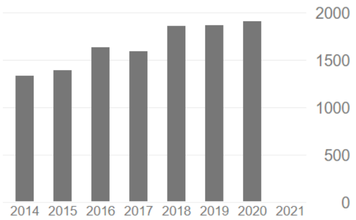

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