FRONT PAGES OF PUBLICATIONS – CONTINUED FROM 1971 TO 2020 FILE

Details related to 'the ten' JRC profiles and further work with the Barton-Bandis criterion – why JRC, JCS and φ_r.

by Nick Barton, NB&A, Oslo. 2021.

This highly illustrated article, with minimal text, is basically an abstract followed by many figures and figure texts. It ends with a reference list that goes beyond Barton and Choubey, 1977 – which is where many published articles 'stop' in relation to work performed on the BB criterion – which has been part of UDEC-BB since 1985. There are by now more than 60 prolile-related equations in the literature, and hundreds of articles, all addressing JRC. Many do not reference the source of JRC anymore, assuming it is an 'established parameter'. It is however liberally criticised, with justification of why 'the current research' was funded and reported. This article is designed to try to put to rest some misconceptions and errors made by many who see 'the ten JRC profiles' and assume (correctly) that they represent a far too subjective method for estimating peak shear strength. In fact, the ten selected profiles, with suggested ranges of JRC like 8 to 10, 14 to 16 were just to illustrate the range of surfaces tested. We characterized and tested130 natural rock joints, from seven different rock types. There are 390 other roughness profiles, since three per sample. The main focus was the accuracy of the peak shear strength prediction. We used gravity tilt or (horizontal) pull tests at mostly < 0.001MPa normal stress for comparison to the DST tests on the same samples at normal stresses of approx. 0.1 to 1.5MPa, so up to one thousand times higher stress. Tilt, push and DST are 'real' 3D behaviour, 2D profile predictions are not. Some of the latter developments have been erroneously based on the assumption that we used 1mm diameter 'brush' profilers commonly found in hardware stores. Some 'creative' authors even drew stepped profiles imagining steps in ours (there are none) and misleading the profession to assuming 100 z-coordinates per 100mm long sample. This has spawned incorrect science and conclusions. The reality was an unusually precise Leschhorn gauge with 3 or 4 'shims' (blades) per mm. (See Appendix and Figure 3). A significantly stepped fine-pencil trace was not possible. Those not reading past our 1977 article miss scale effects and coupled behaviour, which of course depends on normal stiffness and apertures, both physical and hydraulic. The following figures give some indication of where JRC, JCS and φ_r have been used in the years following 1973/1977.

2021

UNDERGROUND NUCLEAR POWER PLANTS by ISRM Prof. Sakurai Committee.

From Nick Barton

Chapter 16. Conclusions

With suitable siting, and suitable engineering geological site description and design, the rock engineering construction costs of 10m, 20m, 30m, or 50m span (and of course much longer) caverns can be reliably estimated and their stability guaranteed by application of modern rock design and construction techniques. Note that the volumetric cost reduces with increased size due to a favourable surface/volume relationship. This has been verified many times in storage projects. Rock support within the connecting tunnels and UNPP caverns should not include concrete linings if there is potential for earthquakes as that historically invites cracking during seismic loading and is unnecessarily expensive. Concrete linings do not increase long-term stability. Even extremely adverse structural geology, such as dipping sedimentary rock with bedding planes filled with sheared clay ('bedding-plane faults') have also been engineered on occasion and resisted major earthquakes successfully without any reported damage due to the appropriate bolting, anchoring and fibre-reinforced shotcrete cavern support (Barton, 1996, 2021. Refer to the M7.8 Chi Chi earthquake, 9km deep with nearby epicentre).

As opposed to the typical surface nuclear power plant, one that is sited underground is secure from physical damage caused by hurricanes, tsunamis, earthquakes, and missile attacks or aeroplane accidents or terrorist hijacks of aircraft as in '9/11'. Concerning precedent for using rock caverns, the foremost in complexity are probably the 800 or more underground hydroelectric stations, which require three parallel caverns of large volume. The machine halls housing a typical line of multiple generators have reached several hundreds of meters length, spans of more than 30m and heights in excess of 80m. As much as 8,000 megawatts have been generated in single facilities, and with mirror image plants on either side of the river, 16,000 Mw have been produced at Baihetan. Mirror image UNPP could share cooling water facilities, and be much more economic as a result, if desired.

2021

FJELLSPRENGNINGSTEKNIKK
BERGMEKANIKK/GEOTEKNIKK 2021

NEW IDEAS ABOUT FAILURE MODES IN ROCK MASSES – FROM TUNNELS TO PREKESTOLEN TO EL CAPITAN TO EVEREST

Nye ideer om bruddmekkanismer i begmasser – fra tunneler til Prekestolen til El Capitan til Everest

Nick Barton, PhD, NB&A, Høvik, Norway

SUMMARY

This paper deals with the exploration of failure modes in rock and rock masses. Failure in tension initially applies in deep tunnels, and extension failure also applies to cliffs and mountain walls. In each case a free surface is present. However, 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. In deep tunnels in massive rock it has been common practice, also in the Q-system, to compare an estimate of the maximum tangential stress with the uniaxial strength of the intact rock. When this ratio reaches approximately 0.4 rock failure and acoustic emission initiate. An alternative and more realistic interpretation involves the numerically equivalent ratio of tensile strength and Poisson's ratio derived very simply by Baotang Shen when formulating his FRACOD code. The present author has applied this to explain the limited height of cliffs in weak rock and mountain walls in strong rock, a range of heights exceeding 10 to 1,000m. In each case an ultra-simple term involving tensile strength, density and Poisson's ratio is used. If the rock is jointed, there are usually massive changes in strength and stability and slope height, in relation to slopes in intact rock. 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 failure at its base. The factor of safety may be different from that obtained by conventional shear strength analysis. The Mohr-Coulomb criterion gives unrealistic solutions to cliff and mountain wall heights due to too high cohesive strength for intact rock.

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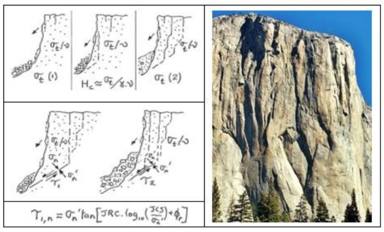


2021

TECHNICAL DETAILS OF SINGLE-SHELL NMT TUNNELS

N.R.Barton
NB&A, Oslo, Norway
E. Quadros
BGTech, São Paulo, Brazil

Abstract: Selected aspects of NMT are described in some detail. Tunnelling in jointed rock that may be clay-bearing and faulted is assumed, with a typical wide range of Q of at least 100 down to 0.01, or roughly RMR = 80 down to 20, but not needing double-shell NATM. Selected aspects to be discussed will be the three principle EDZ, two of them representing the load-bearing cylinder of rock where redistribution of principal stresses and joint deformation occurs, the third the disturbance due to blasting, which is much narrower. So-called 'plastic' behaviour via GSI, H-B, RS2 modelling is rejected since based on too many assumptions and complex page-wide equations. Case records suggest that combinations of bolting and fiber-reinforced shotcrete can provide stable tunnels at reasonable cost, but if some aspects are neglected, like under-dimensioned shotcrete thickness, lack of washing prior to shotcreting, and failure to record the presence of clay, then surprises can occur. Two important further conventions need to be adhered to. The Q-system based B+S(fr) reinforcement and support recommendation was never designed to accommodate or rely on lattice girders, which are far too 'soft' since unbolted and unevenly loaded. Single-shell Q-based tunnel design was also never intended to allow the passage of water at high velocities, such as 10m/s river diversion compared to the case-record expected 2m/s of typical headrace and pressure tunnels. When rock mass quality is compromised by fracture zones, or if permeability is too high and inflow from the surrounding rock mass needs prevention for ensuring both dry in-tunnel and stable external environments, then systematic pre-injection may be demanded. Injection of suitable stable grouts at high pressure improves the rock mass quality Q, and over-design of unadjusted Q-based support is then apparent. P-wave velocities, and deformation moduli are also improved by pre-grouting, as verified in formal dam-site studies in Brazil and Iran. In reality millions of kilometers of grout holes beneath the world's largest dams are giving the same evidence. Suitable stable grouts with their extensional viscosity must not be disqualified with filter-pumps. High injection pressures are needed, but do not hold pressure when flow ceases. Wet shotcrete, leaking bolt holes, and the need for post-injection indicate failed technology, if the objective was to pre-inject in one round only and prevent environmental damage.

1 INTRODUCTION

The frequent assumption of those who feel they know best is that the Q-system only applies to typical hard jointed rocks. We actually make wider use of Q in NMT: the Norwegian Method of (single-shell) Tunnelling. The original case records included 50 different rock types in the initial two hundred or so cases analysed, with deliberate choice of challenging cases such as clay-bearing and sheared rock masses so that significant amounts of support were included. If a more limited range of application of Q had been suggested that would have been the result, since Q is an *a posteriori* empirical method.

Development of the Q-system has meant engagement in numerous tunnel and cavern projects in Norway and abroad since 1975, including experiences in water transfer tunnels, hydropower headrace and pressure tunnels in many countries. Significantly, the Q-system data base and applicability was greatly expanded in 1993, by Grimstad's incorporation of steel fiber reinforced shotcrete S(fr) and by the development of corrosion-protected sleeved (CT) bolts. Both have added to the reliability of B+S(fr) single-shell permanent support. The Q-system has been successfully used in rocks with UCS as low as 4 to 7MPa (significantly jointed chalk marl in shallower parts of the Channel Tunnel: Barton and Warren, 2019) and UCS up to at least 300MPa for some granites, gneisses and quartzites.

2021

9TH CONGRESS & EXHIBITION, Asian Mining. February 15-18, 2022. Kolkata, India

TUNNELS, CAVERNS AND SLOPES IN DISCONTINUA – A CRITICAL ASSESSMENT OF CONTINUUM ANALYSES, GSI, HOEK-BROWN AND MOHR-COULOMB, WITH FOCUS ON DISCONTINUUM ANALYSES AND GEOLOGY

Nick Barton, NB&A, Oslo, Norway (www.nickbarton.com)

Abstract

Rock masses are by definition assemblies of rock blocks separated by joint sets and less frequent faults. Over the years quite accurate methods have been developed for numerical modelling of these assemblies, both in 2D (UDEC-MC, UDEC-BB) and in 3D (3DEC-MC). We have used them for studying how tunnels, caverns and slopes perform when excavated in these challenging media. Empirical characterization methods have also been developed which can assist in such activities as tunnel and cavern support, and stope dimensioning. These can complement the numerical modelling. Clearly, open pit slopes in jointed rock are not the same as model slopes in unjointed model materials. We are readily able to observe the differences between real failures and modelled failures. Two key problems seem to be the over-simplicity of GSI and the black-box complexity of Hoek-Brown et al equations. Related codes using M-C parameters derived from H-B seem also to be affected. A return to joint and rock mass characterization for discontinuum models is needed if we are to return closer to reality. We made good progress in rock engineering many decades ago, until too many chose GSI and H-B, the easy way to lose sight of real behaviour since no 'geology'.

Introduction

In this lecture the author will be showing studies with UDEC, 3DEC, FLAC and FLAC3D, in illustrating both discontinuum and continuum analyses for tunnels, caverns and open-pit slopes. The use of the first four parameters of Q for assisting in stope dimensioning will also be briefly addressed – and just two for overbreak.

Having been around for a long time, also as a student colleague of Cundall before he developed his remarkable computer codes, it perhaps is permitted to illustrate briefly what we could achieve with physical models of fractured media *before Cundall's codes became available*, both from thesis times in 1971 and from just prior to Cundall's UDEC release (Barton, 1971, Barton and Hansteen, 1979).

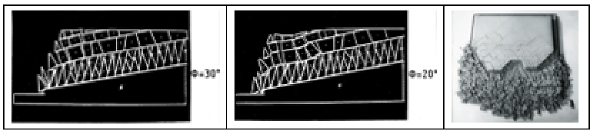


Figure 1 The contrasting flexibility of the intelligent computer code µDEC: two of four results of varying angle Φ from Cundall, Voegele and Fairhurst, 1975, and the 'fixed-fracture-sets' fractured 2D models developed some years earlier by the author in 1968. Coming just before UDEC such 2D 'slab models' with 4,000, 1,000 and 250 blocks also assisted in scale effect understanding. The smallest block sizes gave unexpected 'linear' stress-strain behaviour.

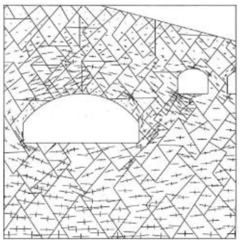
2022

Chapter 16

Engineering geology, rock mechanics and rock engineering for the design and construction of underground (UNPP) rock caverns

16.1 Introduction

The objective of this chapter is to assist in the understanding of some basic rock mechanics and rock engineering principles which are needed following the site investigation for a future underground nuclear power plant development. We will also refer to some prior rock engineering experiences selected from Norway, Taiwan and China which illustrate the confidence with which we have utilized the underground for the construction of large caverns, especially in the last three decades. A variety of rock mass qualities will be referred to, not just the jointed pre-Cambrian granitic gneiss for the widest 60m span, but also far from ideal volcanic extrusive columnar basalts for huge pairs of twin hydropower caverns, and caverns in challenging interbedded sandstone with faulted (sheared) clay inter-beds. The siting needs in these particular cases vary very widely: a convenient city-outskirts hillside, a major river canyon dam site for hydropower, and a far from ideal underground rock cavern site but with the advantages of an existing top-reservoir for pumped hydro. Due to the huge range of sites utilized in the past decades, we have learned how to safely engineer the necessary cavern complexes in geologic locations that may not always be ideal from a rock mass quality viewpoint. Important developments have occurred and been applied during at least the last six decades, that make use of the underground something approaching a routine exercise for numerous countries. This is because of the expertise and long experience of hundreds of site investigation, design, consulting and contracting companies operating in the many countries regularly making these underground developments, mostly since the nineteen sixties and seventies.



2022

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On the selection of joint constitutive models for geomechanics simulation of fractured rocks

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ABSTRACT

Fractures such as faults and joints often dominate the mechanical strength and deformation of rock masses. It is thus of central importance to adopt an appropriate joint constitutive model in geomechanics simulations so that the behaviour of fractures can be realistically represented. Over the past decades, various joint constitutive models have been proposed from theoretical/experimental perspectives and implemented into different geomechanics solvers. However, numerical modelling researchers are often confronted and even confused with the question about which joint model to use in their simulations, especially when a compromise needs to be reached between the realism (or complexity) of the selected constitutive model and the difficulty in the numerical implementation. In this Short Communication, we review some of the popular joint constitutive laws that have been used for geomechanics simulations and present a discussion on their suitability and limitations, aiming to provide a guidance for the joint constitutive model selection for computer simulations. We also point out a few unrealistic features of some widely used joint constitutive models with corresponding corrections recommended.

1. Introduction

With the rapid advances in computing technologies, an increasing number of geomechanics models have been developed to simulate the complex processes and phenomena in fractured rocks based on a variety of numerical methods (e.g. finite element method, discrete element method, finite-discrete element method, finite difference method, among others) (Jing, 2003). Due to the enhanced recognition of the important role of fractures in controlling the bulk behaviour and frequent anisotropy of rock masses (Barton and Quadros, 2015), many computational tools have nowadays been equipped with the functionality of explicitly modelling discrete fracture networks (DFNs) in their geomechanics computations (Lei et al., 2017). An early example of this was Cundall's UDEC (Universal Distinct Element Code) (Cundall and Hart, 1985) with the nonlinear Barton-Bandis' joint model as a subroutine from 1985. The DFN concept represents an important step towards a more accurate (or at least more realistic) simulation of fractured rocks, where a representative elementary volume may not exist (Bonnet et al., 2001) so that the conventional continuum models building upon a homogenisation paradigm might not be applicable. The development of DFN-based geomechanics models is faced by two core questions: (i) how to realistically construct fracture network geometries, and (ii) how to realistically mimic fracture mechanical responses. The first question has been explored in (Lei et al., 2017), while the second question will be discussed in the current paper. The motivation of writing this dedicated Short Communication arises from both authors' observation of the field, where many numerical modellers attempt to use unrealistic joint constitutive models in their "fashionable" computer simulations, resulting in a vague connection to real-world rock mechanics and rock engineering problems.

We write this Communication aiming to guide modelling researchers to strengthen the realism of their simulation tools, so that they can properly consider the important fundamental characteristics of rock fractures in nature, as has been well documented in the literature based on extensive experimental evidence; see e.g. (Bandis et al., 1981, 1983; Barton et al., 1985; Barton and Choubey, 1977; Goodman, 1976) among many others. The rest of the paper is organised as follows. In section 2, we present an overview of the key mechanical characteristics of rock fractures as observed in the laboratory. In section 3, a review of some commonly used joint constitutive models is given together with some remarks on the model suitability and limitations as well as possible corrections. Finally, the paper ends with a short discussion.

2022

TUNNELS AND STATIONS THAT SHOULD BE DEEPER

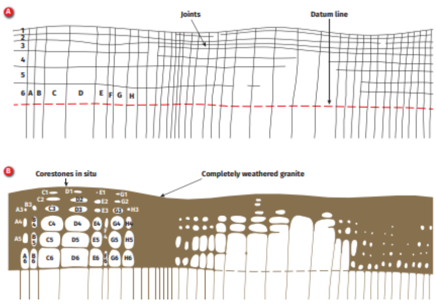
Dr N Barton (Nick Barton & Associates, Oslo) and **M Abrieu** (CVA Consortium, São Paulo) expose the false economies and dangers of shallow tunnelling for metros in urban areas, arguing that deeper tunnels and longer escalators are well worth the extra cost

Recently completed metro Line 4 (Yellow Line) of São Paulo metro was the first major underground construction project in the southeast corner of the 17 million-population Brazilian city. Following the usual, but unfortunate, wish of many owner-operators for shallow stations and short escalators, the contractor struggled to build 4.5km of shallow tunnels and five shallow stations which would be a much-needed addition to the city metro.

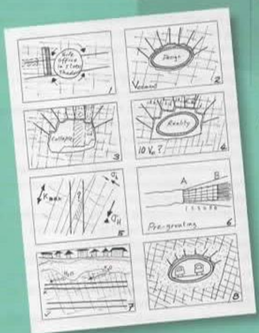
Problems encountered inevitably included mixed-face rock-saprolite conditions, deep differential weathering when in biotite gneiss, deeply weathered core-stone conditions when in granite, and generally more difficult saprolite and soil conditions than anticipated by the experienced contractor, who supplemented the owner's extensive vertical site exploration with around 30 deviated boreholes.

A single breakthrough to street level was also experienced, on this occasion caused by a very long, several hundred-ton slab of gneiss which penetrated through the bolt and shotcrete reinforcement. The failure was caused by the smooth-planar and deeply-weathered vertical boundary jointing which was aided by a saprolite cover of some 20m thickness that had been fully-saturated by 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.

Right, figure 1: A price has to be paid for tunnelling too close to the 'dotted line', illustrated from the particular case of weathered granite from Dartmoor, England, after Linton (1955) and Fookes et al. (1971). In general, Q-parameters are not easily determined in the Grade V (brown) saprolite



2022



ADDENDUM

Digging deeper tunnels may not necessarily be more costly

A reminder from Dr Nick Barton

In the article *'Tunnels and stations that should be deeper' (T&T April 2022)* the standfirst stated that 'deeper tunnels and longer escalators are well worth the extra cost'. The author, Dr Nick Barton, reminds T&T that deeper tunnelling is not necessarily more costly: "Tunnelling (deeper) in rock – of improved quality (drill-and-blast or hard-rock TBM) – or tunnelling deeper in London clay, or deeper in the SP equivalent 'tagua' (over-consolidated clay with sporadic fractures and slickensides like London clay) are all cheaper, respectively, than being shallow in weathered rock/saprolite (mixed face – as in Line 4 case record); or in shallow Reading Beds of gravel in London; or in weaker soils in Sao Paulo."

Furthermore, Barton would like to make clear that his article published last month was not a response to the recent collapse in Sao Paulo of February 2022, where an expressway collapsed above an under-construction subway tunnel. In fact, the article was based on Barton's paper which anticipated such collapses and was written in December 2006 – just a few weeks before the Pinheiros Line 4 collapse of January 2007. For various reasons, permission to publish the paper during the construction of Line 4 was not granted back in 2006. ■

Have your say...

If you would like to add to this discussion or comment on any other aspect of this publication please email your letters to: editor@tunnelsandtunnelling.com

2022

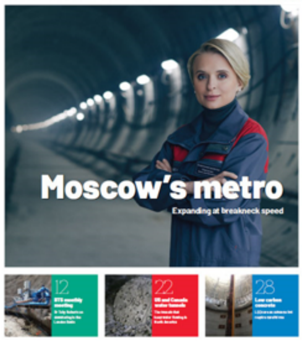
Barton responds to T&T Moscow Metro article

27 January 2022

Print Email

Sir,

I read with interest about the impressive Moscow metro expansion (from the interview with Anna Merkulova, T&T, November 2021, p18). One sentence in particular caught my attention: "We increased the pace of the programme by placing most of the new sections close to the surface rather than deep down." Some 15 years ago, I was hired to give advice to the consortium which was struggling with the time-increasing and risk-increasing consequences of the shallow Line 4 in Sao Paulo. The consequences of mixed face, such as soil to rock in the same station (Butanta) and huge overbreak events due to the closeness to the surface in this station – and in the nearby running tunnel – were clear for all to see. The difficulty of near-to-surface pre-injection also resulted in abandoned houses (to this day), and big settlements needing infill to prevent flooded sections of two roads. The consequences of a ridge-of-rock crushing lattice girder support and the resulting loss of life at the next station cavern (Pinheiros) is regrettably well-known.



These experiences caused me to title a subsequent lecture in Hong Kong: 'The shallow escalator syndrome'. Later, there followed five years as a rock mechanics reviewer of MTR metro expansion – mostly with in-rock station caverns and running tunnels, and I would say wiser and cheaper designs by the various firms responsible.

So, to the obvious question. Is it really 'increasing the pace' when struggling with unstable sands and soils, and the occasional need for freezing? Why did London choose to go deeper and mostly have the benefit of the London Clay? Or the benefit of granite or tuff, rather than saprolite in Hong Kong?

Is the frequent 'choice' of soil-related problems in many other metro-expansions around the world saving time and schedule, or is it because of the dominance of soil mechanics over rock mechanics in the rosters of our geotechnical umbrella organisations?

Nick Barton
Norway

2022

IX Latin American Rock Mechanics Symposium
IX Simposio Latinoamericano de Mecánica de Rocas
Asunción-Paraguay
October 16-19, 2022 *KEYNOTE PAPERS*

Keynote Lecture: Continuum or Discontinuum – That is the Question

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

ABSTRACT:

Several decades ago there was a strong focus on the need for discontinuum modelling to improve upon the empirically based analysis of excavations in jointed rock. The remarkable codes developed by Peter Cundall: UDEC and 3DEC were put to full use in the nineteen eighties and nineties. For example, Q-system based cavern support could be verified or improved with such analyses. Of course, these codes preferably require knowledge of rock mechanics and rock joint behaviour, and perhaps familiarity with non-linear constitutive models as in UDEC-BB. Regrettably the classic textbooks of Hoek and Bray and Hoek and Brown in this period were subsequently followed by the suggestions for continuum modelling using a still not finalized GSI – there are many attempts at improved quantification. JRC now reaching 50 years is also the subject of improved quantification, but it is not followed by the extraordinary page-wide equations for 'c' and 'φ' so no software is needed. The incorrect addition of these components of shear strength (as indeed in Mohr-Coulomb) in commercial continuum codes is the final source of error of so many analyses. So-called plastic zones are exaggerated around tunnels, and rock slopes are given seldom observed deep spoon-shaped failure predictions, ignoring the frequent influence of major discontinuities, and the usual failures within the slope faces. Of course, lake-bed open-pit slope deposits or extremely weathered rock will give spoon-shaped failures as for rockfill and soil, but competent jointed rock will not fail like this: major discontinuities will usually be involved, and wedge or planar failures will be the usual reality.

1 INTRODUCTION

We were advised more than 50 years ago by Brace and Müller that cohesion is broken before friction is fully mobilized. Gross errors are caused by adding these components of shear strength when estimating the maximum height of cliffs and mountain walls. Since 'c' is not the lowest component of strength, artificially lowered estimates are needed, or tensile strength and Poisson's ratio are used (Barton and Shen, 2017). There is precious little empirical basis for the Hoek-Brown equations for rock mass strength, but an excellent experimental basis of course for the earlier intact rock H-B criterion. We may ask if is it logical to downgrade the strength of intact rock to model rock masses (using opaque equations with joint roughness and number of joint sets ignored) or better to apply the equations for the shear strength of joints and fractures and estimate the initial cohesive contribution of intact bridges between the kinematically capable joint sets? In this lecture the author will be showing studies with UDEC, 3DEC, FLAC and FLAC3D and FRACOD, and will be illustrating both discontinuum and continuum analyses for tunnels, caverns and open-pit slopes. An earlier than UDEC phase, with fractured (2D) models of underground excavations, will also be shown as an introduction.

2022

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Current Trends in Civil & Structural Engineering
Iris Publishers
Mini Review Article

Numerical Modelling Trends in Tunnelling and Rock Slope Stability: Current Concerns of Some of us

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Mini Review

In recent keynote lectures the author of this brief 'opinion piece' has utilized variants of the title: 'Continuum or Discontinuum' – adding 'that is the question' and also 'GSI or JRC'? The reason for such titles is the last 20 years or so of numerical modelling practice for tunnel design and rock slope stability checks. These have seemingly been dominated by the marketing success of the rock mass classification method GSI – the so-called 'geological strength index', and the complex set of equations also proposed by Hoek and Brown and co-authors which we can abbreviate to 'H-B'. Both are utilized in Rocscience finite element models such as Phase 2 or RS 2. The simple-to-use software this company has developed which is needed to utilize the page-wide H-B equations has caught the attention of the younger generation, who can quickly obtain colourful plots of stress distributions and so-called 'plastic zones' surrounding supposedly over-stressed tunnel excavations. In the area of rock slope and open-pit stability the same methods (GSI, H-B, RS 2 or other FEM methods) can rapidly provide 'spoon-shaped' and colourful failure predictions, as if a slope in jointed rock is suddenly bereft of its geologic structure and 'fails' as if it was a slope in a 'continuous' isotropic medium like soil or rockfill. Both the above: 'plastic zones' and 'spoon-shaped' failures can be questioned for their link to real behaviour Figure 1 shows one realistic and one unrealistic simulation.

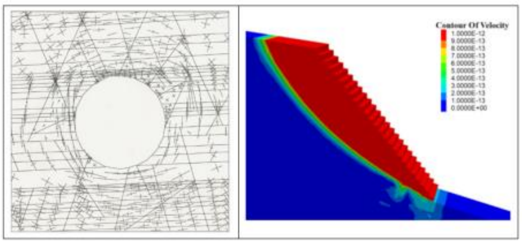

Figure 1: On the left an extract of a UDEC-BB model to represent the possible influence of geologic bedding planes and conjugate jointing on the stress distribution around a planned TBM access tunnel. Clearly this is a discontinuum model with discrete description of the joint properties using, among other parameters, the 50 years old JRC-joint roughness coefficient. On the right an actually sophisticated model of a rock slope with the shear strength of the assumed continuum modelled with the help of M-C (Mohr-Coulomb) and H-B (Hoek-Brown) non-linear behaviour with input data estimates based on GSI.

2023

GSI OR JRC – CONTINUUM OR DISCONTINUUM MODELLING – SOME SUGGESTIONS AND SOME CRITIQUE

2023 15TH ISRM CONGRESS

Nick Barton

NB&A, Oslo, Norway

Abstract

GSI has been applied for about 30 years and JRC for about 50 years. They are associated with either the Hoek-Brown based shear strength criterion for rock masses and *continuum modelling*, or with the Barton-Bandis based shear strength criterion for rock joints for use in *discontinuum modelling*. The latter, using input parameters JRC, JCS and φ_r provides for non-linear and block-size dependent shear-strength-displacement and dilation-displacement behaviour, and non-linear closure-aperture behaviour, including the potential for coupled hydraulic flow modelling, despite 2D limitations.

Introduction

During past decades there have been periods with FEM dominated continuum modelling, followed by decades of DEM dominated discontinuum modelling when for instance UDEC, 3DEC and FRACMAN became available thanks to early developments by Cundall and Dershowitz.

In more recent decades it seems that a return to continuum modelling of rock masses has been dominant. This has undoubtedly been in response to the commercial promotion of GSI and the Hoek-Brown equations for representing rock masses, and related commercial software.

In the author's paper some critical observations were made to emphasize what is lost when attempting to select a representation of 'geology' in the GSI diagram. *The subsequent 'loss of all geology'* is because there is a 'homogenization' of properties using Rocscience software to evaluate the 'page-wide' Hoek Brown equations for 'c' and 'φ' followed by FEM modelling..

Figure 1. Contrasting continuum and discontinuum models. This modelling was performed by Backer, 1993 (priv. comm.) just prior to the development of GSI, H-B, and FEM Phase 2 or RS2.

It is doubted that continuum methods can help the profession understand fundamental rock mass behaviour, as so-called 'plastic zones' around underground excavations have been proved to be exaggerated, and spoon-shaped 'failures' of slopes in jointed rock are not observed unless the rock material is very weak. Jointing and faulting influence is usually present in rock slopes, and likely to be present and effect the performance of tunnels and caverns in rock as we can see post-failure in slopes.

Rock joint behavior modelling

Input data for rock joint modelling requires testing or empirical estimation to provide some resemblance of reality, such as seen in Figure 2.

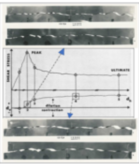

Figure 2. Fundamentals of behaviour for rough tension fractures. Photogrammetric roughness profiles were sheared and dilated as measured in the relevant direct shear tests. (Barton, 1973).

For modelling joints there are several options as shown in Figure 3. The non-linear and block scale-dependent Barton-Bandis model is now widely used for distinct element modelling and appears as a subroutine in UDEC-BB. Non-linear closure and flow modelling is a part of BB.

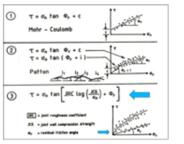

Figure 3. Three joint modelling options. #3 is naturally preferred

Figure 4 shows example JRC-based input data for UDEC-BB modelling of the Gjøvik cavern. Note the depth-dependent deformation moduli.

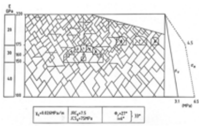

Figure 4. Input data examples for the 62m span Gjøvik cavern. The UDEC-BB predicted deformations of 7-8mm matched reality.

Examples of UDEC-BB models of a TBM access tunnel in interbedded sandstones (showing principal stress rotations in the EDZ), and joint shearing EDZ modelled in jointed tuff are shown in Figure 5. Many such details are inevitably lost in GSI-H-B based continuum models.

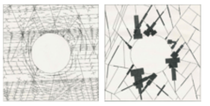

Figure 5. Examples of UDEC-BB models of a TBM access tunnel through sandstones and siltstones then into jointed welded tuff.

In contrast to discontinuum behaviour, some unexpected theories for rock slope modelling involving linear M-C or non-linear H-B are shown in continuum modelling literature. Strangely, circular or spoon-shaped failure is expected to apply, despite what is usually jointed and locally faulted rock. In reality, only rockfill and soil may comply with such assumed continuum 'circular' failure predictions. Faults may dominate.

Figure 7. Non-circular fault driven open pit failure. Bingham Canyon mine, Utah. Top-to-toe dimension is 3km. 70 million m3.

Critique of opaque GSI H-B equations

The page-wide opaque algebra of the GSI based H-B equations, conveniently programmed in commercial software, allows even novices to apply it without sufficient questioning. Note the strange appearance of GSI *16-times* in 'c' and *12-times* in 'φ'. Super sensitivity to a poorly quantified parameter (GSI) easily leads analyses astray. The disturbance factor D has to be guessed.

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}}$$

$$\varphi' = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right]$$

$$m_b = m_i\exp\left(\frac{GSI-100}{28-14D}\right) \quad s=\exp\left(\frac{GSI-100}{9-3D}\right) \quad a=\frac{1}{2}+\frac{1}{6}\left(e^{-GSI/15}-e^{-20/3}\right)$$

Figure 7. Where do we see a clay-filled rough set of joints? Where do we see two, or four sets of joints? In GSI?

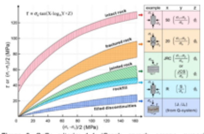

Figure 8. CcSs criteria: 1-4: 'Crack, crunch, scrape, swoosh' represent four possible progressive shear strength components.

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2023

GSI or JRC – continuum or discontinuum modelling – some suggestions and some critique

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ABSTRACT: GSI has been applied for about 30 years and JRC for about 50 years. They are associated with either the Hoek-Brown based shear strength criterion for rock masses and *continuum modelling*, or with the Barton-Bandis based shear strength criterion for rock joints for use in *discontinuum modelling*. The latter, using input parameters JRC, JCS and φ_r provides for non-linear and block-size dependent shear-displacement and dilation-displacement behaviour, and non-linear closure-aperture behaviour, including the potential for coupled hydraulic flow modelling. The mismatch of hydraulic and physical apertures, requiring lab-scale JRC_0 for the conversion. The paper provides some examples of joint-related behaviour in the case of tunnels, caverns and slopes. It also includes serious critique of GSI and the H-B based continuum modelling, due to the complex equations and the lack of representation of joint properties. So-called plastic zones are exaggerated around tunnels, and spoon-shaped slope failures belong in soil mechanics.

Keywords: modelling, rock masses, rock joints, JRC, GSI, shear strength.

1 INTRODUCTION

During past decades there have been periods with FEM dominated continuum modelling, followed by decades of DEM dominated discontinuum modelling when for instance UDEC, 3DEC and FRACMAN became available thanks to early developments by Cundall and Dershowitz. In more recent decades it seems that a return to continuum modelling of rock masses has been dominant and this has undoubtedly been in response to the commercial promotion of GSI and the Hoek-Brown equations for representing rock masses, and commercial software. In this paper some critical observations will be made to emphasize what is lost when attempting to select a representation of 'geology' in the GSI diagram. The actual 'loss of geology' is because there is a 'homogenization' of properties using Rocscience software to evaluate the 'page-wide' Hoek-Brown equations for 'c' and 'φ' and thence to FEM. It is doubted that such methods can help us understand fundamental rock behaviour, as so-called plastic zones around underground excavations are exaggerated, and spoon-shaped 'failures' of slopes in jointed rock are not observed unless the rock material is very weak.

Figure 1 illustrates the strong contrast between a continuum model and a discontinuum model when both are focused on the same problem: motorway and ramp tunnels in close proximity. The same rockmass deformation modulus are applied in each case, but the specified joints have normal and shear stiffnesses and can react to excavation under the applied stress.

Figure 1. The contrasting information from a Cundall continuum model and from a Cundall discontinuum model, respectively FLAC-MC (2D) and UDEC-BB (2D). Anisotropy and local instability and eventual overbreak are clearly not a part of the continuum model. (Pers. comm. L. Backer, more than three decades ago). (MC refers to Mohr-Coulomb, and BB to Barton-Bandis representation of shear strength). $\delta_1/\delta_2 \approx 1/3$.

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Review

Advances in joint roughness coefficient (*JRC*) and its engineering applications

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ABSTRACT

The joint roughness coefficient (*JRC*), introduced in Barton (1973) represented a new method in rock mechanics and rock engineering to deal with problems related to joint roughness and shear strength estimation. It has the advantages of its simple form, easy estimation, and explicit consideration of scale effects, which make it the most widely accepted parameter for roughness quantification since it was proposed. As a result, *JRC* has attracted the attention of many scholars who have developed *JRC*-related methods in many areas, such as geological engineering, multidisciplinary geosciences, mining mineral processing, civil engineering, environmental engineering, and water resources. Because of such a developing trend, an overview of *JRC* is presented here to provide a clear perspective on the concepts, methods, applications, and trends related to its extensions. This review mainly introduces the origin and connotation of *JRC*, *JRC*-related roughness measurement, *JRC* estimation methods, *JRC*-based roughness characteristics investigation, *JRC*-based rock joint property description, *JRC*'s influence on rock mass properties, and *JRC*-based rock engineering applications. Moreover, the representativeness of the joint samples and the determination of the sampling interval for rock joint roughness measurements are discussed. In the future, the existing *JRC*-related methods will likely be further improved and extended in rock engineering.

1. Introduction

Rock joints, mechanical discontinuities of geological origin, intersect almost all near-surface rock masses and significantly influence their engineering properties. Roughness is an essential component of the shear strength of rock joints, particularly in the case of undisplaced and interlocked features such as unfilled joints. This is because lack of planarity means dilation, higher local stresses, and increased permeability. Over the past five decades, researchers have proposed different methods to quantify the joint roughness (e.g. Barton and Choubey, 1977; Yu and Vayssade, 1991; Kulatilake et al., 2006; Tatone and Grasselli, 2010; Yong et al., 2017). Among all the joint roughness parameters in the literature, the joint roughness coefficient (*JRC*) is the one most widely used in practice.

It was first proposed by Barton (1973) some 50 years ago for evaluating rock joint shear strength and quantifying roughness. *JRC* was later adopted by the International Society for Rock Mechanics (ISRM) (Barton, 1978) for the description of discontinuities. It has the advantages of simple form, easy estimation, and explicit consideration of scale effects. Since the quantification of *JRC* is crucial to the shear strength of jointed rock masses, it has attracted a large number of attentions.

JRC-related methods have gradually developed as general methods for solving rock engineering stability problems. The concept of *JRC* has been applied in various fields such as geological engineering, multidisciplinary geosciences, mining mineral processing, civil engineering, environmental engineering, and water resources. In these *JRC*-related methods, the nonlinear rock joint shear strength criterion, called Barton-Bandis criterion (also called BB criterion), is the most used. This criterion, with the shear strength portion refined by Barton and Choubey (1977), received important additions from the scale-effects and normal-closure work of Bandis (1980). Since the 1980s, this criterion has been used in numerous computer codes for mechanical-hydraulic

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PRE-GROUTING OF TRANSPORT TUNNELS IN JOINTED ROCK FOR SUCCESSFUL CONTROL OF WATER

Forinjisering av transport tunneler i oppsprukket berg for vellykket kontrol av vann

Dr Nick Barton (Nick Barton &Associates)
Prof Steinar Roald (Dr S Roald A/S, Norway)

SUMMARY

Pre-grouting is an effective way of displacing water and severely limiting inflow to tunnels, if practiced correctly. Joint sets are successively sealed, and permeability tensors are known to rotate and reduce in magnitude for each set. This has been measured during 3D permeability tests. In fact, the needs for tunnel support and reinforcement are actually reduced by successful pre-grouting, but not when wet shotcrete or leaking bolt holes are seen following unsuccessful pre-injection. The possibility of dry tunnels depends on the use of stable non-shrinking grouts with microsilica additives. Due to extensional viscosity the latter are de-selected if using the inadvisable filter-pump which is favoured in some countries. Particle sizes should be appropriate to the estimates of mean physical joint apertures (E). Hydraulic apertures (e) estimated from permeability testing are idealized smooth parallel plates. They are smaller, mathematically derived apertures so are physically non-existing objectives for determining the cement particle fineness, using either ultrafine, or micro-cement, or industrial Portland cement. The rule-of-thumb of E needing to be greater than $4.d_{95}$ has been proved experimentally in rock joint samples. The aperture difference E ≥ e is due to hydraulic losses due to roughness. These apertures are approximately equal when greater than 1.0 mm. A poor pre-injection result like wet shotcrete and leaking bolt holes may also result from too low injection pressures. Local joint jacking is needed, with limited risk when flow of grout is occurring. There is an inevitable logarithmic to linear pressure decay from the injection borehole out into the intersected joint planes, with at least 50% loss of pressure within 1m for Newtonian-fluids, and obviously more for rough joints using cementitious grouts with their Bingham-fluid cohesion and friction. However, pressure must not be held when flow has stopped. Injection pressure must obviously be lowered when not needed, if there are large flows near the surface or in permeable crushed zones at depth. If for some reason one is not using stable cements with the necessary micro-silica additive, it will be necessary to use lower pressure anyway, but one must then expect poorer penetration and volume reduction when hardened, meaning the likelihood of wet shotcrete. The authors will draw on their experiences from confidential expert witness and court experiences of several pre-and-post injection projects in Norway and abroad.

Keywords: Pre-grouting; settlement-damage; high-pressure; micro-silica; joint-apertures

2023

POSSIBLE CONSEQUENCES OF CLASSIFICATION METHODS APPLICATION, COUNTRY BY COUNTRY, IN THE CASE OF ROCK SLOPE FAILURE MODELS

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Rock masses are by definition assemblies of rock blocks separated by joint sets and less frequent faults. Of course they can be very massive too. Over the years quite accurate methods have been developed for numerical modelling of what are often 'block assemblies', both in 2D (UDEC-MC, UDEC-BB) and in 3D (3DEC-MC). Many have used them for studying how tunnels, caverns and slopes might perform when excavated in these challenging media. Empirical characterization methods have also been developed which can assist in such activities as tunnel and cavern support, choosing stable slope angles, and mining stope dimensioning. These can complement the numerical modelling.

The apparently most frequent geographic application of various rock mass classification methods in numerous countries, thanks to the questionaires circulated and synthesised by Erharter et al. 2023 are summarized in Figures 1 and 2. Two potential problems caused by the geographic 'spread' of slope-related methods seem to be the over-simplified and difficult to quantify GSI, and the black-box complexity of Hoek-Brown et al. equations if these are applied following GSI estimation. In the opinion of the author of this 'possible consequences' discussion, a return to joint and rock mass characterization for application in discontinuum models is needed if we are to return closer to reality. Do colourful continuum models have a place in actual engineering in rock?

We made good progress in rock engineering many decades ago, until too many chose GSI and H-B, the easy way to lose sight of real behaviour since actually there is hardly any application of geology involved despite the 'G' in GSI. Any subsequent continuum modelling will mean a regrettable loss at least of structural geology.

On the subject of rock slope stability (Figure 2 empirical methods), the behaviour and occasional failures in open pit slopes in jointed rock are actually not very well related to modelled slopes in unjointed model simulations. We are readily able to observe the differences between real failures, often involving capable joint sets and faults, and the idealized modelled failures, typically 'spoon-shaped', if using currently popular (GSI, c, ϕ, H-B) methods.

In the writer's humble opinion it is remarkable that so many, perhaps mostly young people, are trusting the use of GSI, H-B, and continuum models – both for tunnels and caverns and slopes. The critique of the H-B equations presented in Barton, 2023 and partly reproduced in Figure 3 and its caption, should be noted.

nick barton

Nick Barton & Associates
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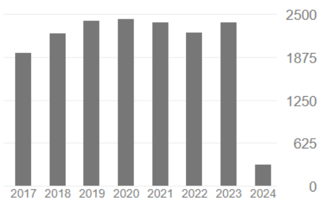

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