# 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 $\varphi_r$.

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 $\varphi_r$ 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**

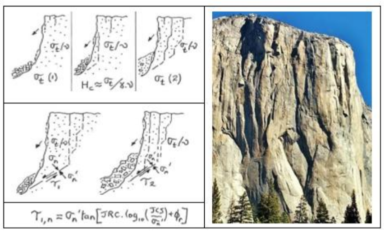
**Nye ideer om bruddmekkanismer i begmasser – fra tunneler til Prekestolen til El Capitan til Everest**

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.

**SAMMENDRAG**



2021

**TECHNICAL DETAILS OF SINGLE-SHELL NMT TUNNELS**

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*NB&A, Oslo, Norway*
E. Quadros
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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

9[TH] 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

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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).

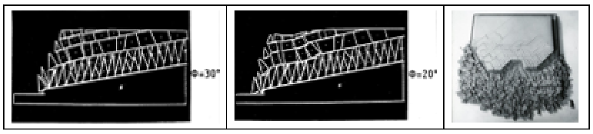



*Figure 1 The contrasting flexibility of the intelligent computer code μDEC: two of four results of varying angle Φ from Cundall, Voegele and Fairhurst, 1975, and the 'fixed-fracture-sets' fractured 2D models developed some years earlier by the author in 1968. Coming just before UDEC such 2D 'slab models'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

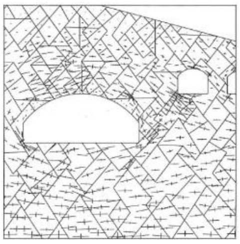
2022

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.



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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

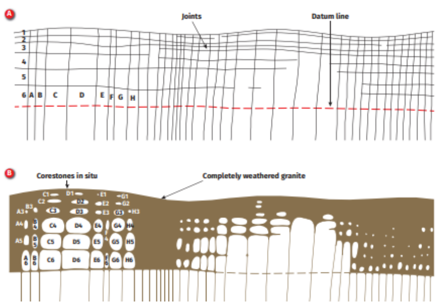
# TUNNELS AND STATIONS THAT SHOULD BE DEEPER

**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.

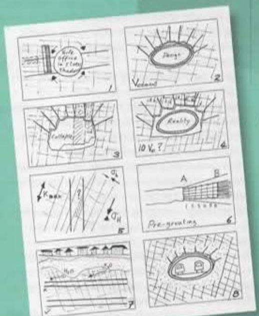
Problems encountered inevitably included mixed-face rock-saprolite conditions, deep differential weathering when in biotite gneiss, deeply weathered core-stone conditions when in granite, and generally more difficult saprolite and soil conditions than anticipated by the experienced contractor, who supplemented 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

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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.

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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.

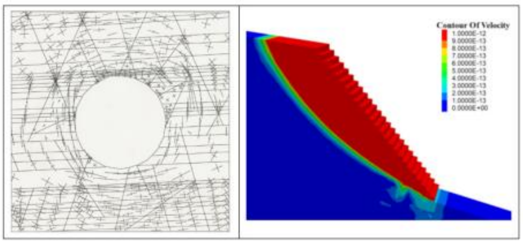


**Figure 1:** On the left an extract of a UDEC-BB model to represent the possible influence of geologic bedding planes and conjugate jointing on the stress distribution around a planned TBM access tunnel. Clearly this is a discontinuum model with discrete description of the joint properties using, among 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

**Nick Barton**

NB&A, Oslo, Norway

2023
15TH ISRM CONGRESS

## 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 $\varphi_r$ 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.

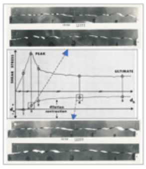


Figure 2. Fundamentals of behaviour for rough tension fractures. Photogrammetric roughness profiles were sheared and dilated as measured in the relevant direct shear tests. (Barton, 1973).

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.

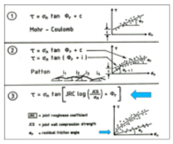


Figure 3. Three joint modelling options. #3 is naturally preferred

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

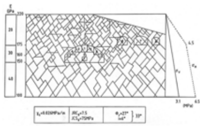

Figure 4. Input data examples for the 62m span Gjøvik cavern. The UDEC-BB predicted deformations of 7-8mm matched reality.

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.

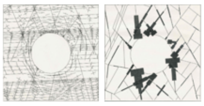

Figure 5. Examples of UDEC-BB models of a TBM access tunnel through sandstones and siltstones then into jointed welded tuff.

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.

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

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

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

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?

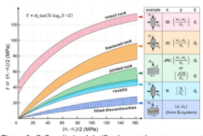


Figure 8. CcSs criteria: 1-4: 'Crack, crunch, scrape, swoosh' represent four possible progressive shear strength components.

## References

Barton, N. 1973. *Review of a new shear strength criterion for rock joints*, Engineering Geology, Vol. 7, pp. 287-332.

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ISRM
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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 $\varphi_r$ 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). $\delta_1/\delta_2 = 1/3$.

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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.

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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

2023



# 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 $E \geq e$ 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

By Nick Barton, NB&A Oslo. www.nickbarton.com

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.

2023

# Personal and ISRM memories of 30 years with Eda

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**Abstract.** Eda Quadros was president of the ISRM from 2015 to 2019. She was also vice-president from 2003 to 2007. She was the first woman to have had either of these ISRM roles. So as her husband from 2000 this was also something new, and a very rewarding conference-focussed addition to our professional and private life together. By very good fortune her strong hydraulic-testing back-ground was also an essential addition to the author's improved understanding of this important field, and resulted in some joint publications, including *how to reduce the influence of hydraulic problems*, namely pre-injection. Insight from 3D permeability testing performed by her at IPT during her PhD, including permeability tensor rotation and diminution before and after grouting was a unique contribution. The present paper starts at our beginning in 1995, is partly personal, partly ISRM and partly technical, with emphasis on her 3D work towards the end.

**Keywords:** Eda Quadros, ISRM president, Hydrotomography

**Fig. 1.** Left: Ove Stephansson signing Eda's sake box in Tokyo 1995. Right: Prof Sakurai, ISRM president from 1995 to 1999 and Eda in ISRM Wroclaw, Poland in 2013, where Eda was chosen as ISRM's first female president for the period 2015 (Montreal) to 2019 (Foz de Iguaçu).

2024

# Celebrating 50 years of Q system development for infrastructure design and follow-up

Nick Barton[1] Eystein Grimstad[2] Ricardo Abrahão[3] Neil Bar[4]

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**Abstract.** This keynote paper reviews the many developments related to the 1974 Q-system, which was developed by the first author when at NGI 50 years ago. Senior co-authors Lien and Lunde were essential sources of tunnelling and engineering geological experience at that time. The early case records were dominated by hydropower, and 50 rock types were represented. As early as 1981 mining engineers suggested using the first four parameters: Q' or RQD, Jn, Jr and Ja unchanged for contributing to stope design in mining. This is now widely used. These four Q-parameters closely resemble relative block size and inter-block shear strength. Thanks to major increases in tunnelling case records by co-author Grimstad in the late eighties and early nineties, mostly Norwegian road tunnels, the extension of Q-based single-shell NMT support to incorporate S(fr) was published in 1993, together with rock-burst based improvements in the SRF stress-strength parameter. In 1995 empirical links to seismic P-wave velocity and rock mass deformation modulus were included, with depth-dependent correction for each. In 2000 $Q_{TBM}$ was developed, involving the inclusion of several machine-rock interation parameters, and co-author Abrahão soon developed a user-friendly model for $Q_{TBM}$ prognoses of PR, AR and Time (or Utilization). In 2006 the first author published a book linking rockmass and seismic quality and at that time the empirical method $Q_{H2O}$ was added, allowing for depth dependent permeability estimation using a slightly modified Q equation. In 2007 the ratio Jn/Jr was suggested for predicting inevitable overbreak in tunnels and caverns, when this ratio is 6 or higher. Finally in 2015 Q-slope was published and thanks to nearly 600 case records collected by co-author Bar we now have a widely used method for selecting slope angles for rock cuttings, even open pit benches.

**Keywords:** Rockmass, Characterization, Tunnels, TBM , Slopes

2024

# Viewpoint:

***Dr Nick Barton, Nick Barton & Associates, Norway***
***Eystein Grimstad, Fomerly NGI, Norway (Retired)***
***Prof Krishna Panthi, Professor of Geological Engineering, Norway***

This Comment paper is in response to the article: 'Key principles for hydropower tunnel design, construction and operation' by D. Brox, published in *H&D* Issue 4, 2024. The letter responds to the author's opinon on the foreign application of the Norwegian Tunnelling Method and addresses the concerns raised.

The authors of this Comment, and other tunnellers in Norway, were surprised at the strength of Dean Brox's critique of the foreign application of 'the Norwegian Tunnelling Method', or NTM, as Brox had understood from reading possible student lectures in Chile given by Eivind Grov of SINTEF. We have registered some 45 publications with NMT (not NTM) in the title, or with NMT prominently discussed, also by some of the originators of the Q-system and its further development, but rather surprisingly none of these has been referred to by Brox, who based some views on his own opinions, referencing just one research project manager from Trondheim who did not develop Q or NMT or NTM.

Before addressing some of Brox's concerns and actual errors, let us consider how he would react if it was suggested (and prominently published) that for him to practice his opinions outside Canada would represent 'a fatal flaw'. That he should keep to 'best quality Canadian granite', that his 'false and dangerous' application of such and such opinions outside Canada would represent high risk. Brox would not appreciate such comments, and nor does the Norwegian tunnelling community appreciate such comments. His paper contains plenty of valid comments on the need to take great care with water sensitive rock and swelling clays. That has been part of the advice within the Q-system for 50 years and in such case records well before this.

Brox makes an incorrect assumption, as others have done who do not appreciatethe details of the Q-system, by lumping it together with RMR and GSI, and criticizing them collectively for "not providing an appropriate means of characterizing the long-term durability of water sensitive bedrock with adverse mineral constituents". The Q-system, because of this 'lumped critique' has also been criticized on previous occasions, even in a European keynote lecture, for not taking into account water or stress or tunnel depth. This is not correct.

The 'fatal flaw' opinion of Brox regarding the application of Q-system based shotcrete permanent support recommendations, except in the case of what would actually be self-supporting high quality Norwegian granites, is a risky viewpoint, as Brox seems to have misunderstood several key aspects of drill-and-blast tunnels. No, we do not need to fill overbreak, and no, we do not fill so-called 'rock traps' with rock, because at least those who know better do not design with the high flow velocities that would be 'needed'. Another presumed expert from Canada also questioned whether rocks could reach the turbines when nominally 'unlined' tunnels for hydropower (NMT: unsupported or single-shell B + Sfr) were being discussed in a court case in Australia some years ago.

It is sure that those pioneers behind the earlier 99 per cent hydropower-based electric grid in Norway discovered how to avoid 'rocks in the turbines' at least 100 years ago. In this connection, the Brox reference to the problems at Hidroituango in Colombia in his Table 1 of 'updated collapses' is somewhat misplaced as it was specifically the result of much too high flow velocities (10 m/s and even greater) without invert concrete or increased erosion protection. A velocity even in excess of 36 km/hr and flow around a quite sharp bend has no place in Q-system tunnel support strategies and is far different from the 60 per cent of hydropower case records used in Barton *et al* [1974] to develop the Q-system.

It is not known whether we can exactly equate the Brox reference of 'NTM' to NMT as published for many years by the undersigned and colleagues, probably starting in World Tunnelling in 1992 with multiple authors from several Norwegian design, owner and contractor companies. It has become a very big NMT reference list (> 45 publications). When the undersigned have been authors or co-authors the publications never refer to 'NTM' only NMT (Norwegian Method of Tunnelling). None of these NMT publications have been referenced by Brox, who has only referenced what may be a lecture on 'NTM' to students in Chile, by Eivind Grov of SINTEF. This is what is suggested in Google. Apologies if incorrect, but the Brox reference to Grov was not sufficient.

The Q-system and the implicit NMT that followed, that is, 'single-shell' tunnelling (shotcrete and systematic bolting), with Q-based characterization for selecting support and support class, was strongly based on hydropower case records from Norway and Sweden from pre-1974 cases. (60 per cent of the case records for developing Q in 1973 were from hydropower projects). There were 50 different rock types in the first 212 case records, and granite, though very common was not the only common rock type. Brox, even with a tunnelling book to his name, seems to have a very biased (and incorrect) picture of Norwegian and Swedish tunnels. To have the opinion that the Q-system is only reliable in high quality Norwegian granites, where actually no B+S(fr) would even be needed, is clearly absurd, and is an example of his blind bias

There are now > 3500 km of hydropower tunnels in Norway. This is much, much more than in Canada. There are numerous rock types involved, numerous depths including > 1000 m, and extensive water- and unloading-sensitive swelling minerals like montmorillonite in countless hydrothermally altered weakness and fault zones. Thick multi-layered shotcrete and appropriate bolting, maybe in the form of RRS is the recommended tunnel support result. Brox has misunderstood RRS (rib reinforced shotcrete arches), as he refers to a 12 to 15 cm thickness. This is in error by a factor of between 2 to 6, depending on how low the Q-value is, and what the tunnel span is.



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ORIGINAL PAPER

# Q-Slope: Rock Slope Engineering 10 Years on

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**Abstract**
The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of rock slopes in the field. It enables geotechnical engineers and engineering geologists to make adjustments to slope angles as ground conditions become apparent during the excavation of reinforcement-free slopes in civil engineering and mining projects. Q-slope was developed by supplementing the Q-system which has been extensively used for 50 years for characterising rock exposures, drill core and tunnels under construction. 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 (O-factor) 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 (strength reduction factor) categories have been developed for slope surface conditions, stress–strength ratios and major discontinuities such as faults, weakness zones or joint swarms. Through over 600 case studies in 36 rock types across 5 continents, a simple relationship between Q-slope and long-term stable slope angles has been established. It includes several failure modes and applies to slopes ranging from less than 5 m to more than 250 m in height. This paper discusses Q-slope application and use for the last 10 years. It presents updated Q-slope stability charts and discusses the time-dependent behaviour of rock slopes.

**Highlights**

- Q-slope method now supported by a database of over 600 case studies, which is provided to the reader for their use and future research.
- Updated slope stability charts for empirically assessing rock slope stability and selecting appropriate slope angles.
- The Q-slope method for rock slope engineering has been adopted in over 45 countries around the world over the last 10 years.

**Keywords** Q-slope · Slope stability · Civil engineering · Mining engineering · Rock mass classification · Empirical method

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**List of symbol**

| | |
|---|---|
| $Q_{slope}$ | Q-slope value for assessing rock slope stability |
| $Q_{slope}'$ | Modified Q-slope value that does not consider orientation factors, external factors, and stress |
| $\beta$ | Long-term stable slope angle in degrees |
| RQD | Rock quality designation |
| $J_n$ | Joint sets number |
| $J_r$ | Joint roughness number |
| $J_a$ | Joint alteration number |
| O-factor | Orientation factor for the ratio $J_r/J_a$ |

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# Q-BASED DEVELOPMENTS DURING THE LAST 50 YEARS

# Q-BASERT UTVIKLING I LØPET AV DE SISTE 50 ÅRENE

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**SUMMARY**
The Q system was published in 1974 with senior NGI colleagues Lien and Lunde. Some details of rock mass classification will be summarized first. Co-author Grimstad was responsible for a very extensive increase in tunnel 'case records' and details regarding fibre-reinforced shotcrete S(fr) and the stress parameter SRF based on experiences from road tunnels under high rock pressure. Design of reinforced ribs of shotcrete (RRS) and energy absorption was published in 2002 by Grimstad. RRS will be illustrated and compared with steel arches and lattice girders. The Q system was originally based on many hydropower projects with moderate water transport rates, e.g. 1.5 to 2.5 m/s and no block transport along the invert. The risk of erosion damage increases when too high water flow velocities are allowed. In 1995, Qc was used to correlate with P-wave velocity and rock mass E modulus, both corrected for depth. In 2000, Qtbm was developed in book form, and a user-friendly prediction model was soon developed by co-author Abrahão. TBM tunnels need prediction of both PR, which is more common, and also time-dependent AR and utilization U, both of which are tunnel length-dependent. Q-H2O with correlation to depth-dependent permeability is from 2006 and a textbook on rock mass and seismic quality. A later development based on Q was Q-slope from 2015, where angles for rock cuttings and benches in open pits can be related to Q-slope. For example, unreinforced 85°, 65°, or 45° slope angles are stable when Q-slope is 10, 1, or 0.1. Co-author Bar is responsible for the very extensive increase in 'Q-slope case records'.

**SAMMENDRAG**
Q-systemet ble utgitt i 1974 med senior NGI-kolleger Lien og Lunde. Noen detaljer om bergmasseklassifisering vil oppsummeres første. Medforfatter Grimstad var ansvarlig for en meget omfattende økning i tunnel 'case records' og detaljer vedrørende fiberarmert sprøytebetong S(fr) og spenningsparameteren SRF basert på erfaringer fra veitunneler under høye bergtrykk. Design av armerte sprøytebetongbuer (RRS) og energiabsorpsjon ble publisert i 2002 av Grimstad. RRS vil bli illustrert og sammenlignet med stålbuer og gitterdragere. Q-systemet var opprinnelig basert på mange vannkraftprosjekter med moderat vanntransporthastighet, f.eks. 1,5 til 2,5 m/s og ingen blokktransport langs sålen. Risikoen for erosjonsskader øker når det tillates for høye vann hastigheter. I 1995 ble Qc brukt til å korrelere med P-bølgehastighet og bergmasse E-modul, begge med korreksjon for dybde. I 2000 ble Qtbm utviklet i bokform, og en brukervennlig prediksjonsmodell ble snart utviklet av medforfatter Abrahão. TBM-tunneler trenger prediksjon av både PR som er mer vanlig, og også tidsavhengig AR og utnyttelse U som begge er tunnellengdeavhengige. Q-H2O med korrelasjon til dybdeavhengig permeabilitet er fra 2006 og en lærebok om bergmasse- og seismiskkvalitet. En senere utvikling basert på Q var Q-slope fra 2015, hvor vinkler for fjellskjæringer og benker i dagbrudd kan relateres til Q-slope. For eksempel er uforsterkede 85°, 65° eller 45° helningsvinkler stabile når Q-slope er 10, 1 eller 0,1. Medforfatter Bar er ansvarlig for den svært omfattende økningen i 'Q-slope case records'.

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Conference for the 75th anniversary of the Faculty of Civil Engineering Skopje
"FROM SLIDE RULE TO ARTIFICIAL INTELLIGENCE"
October 4th, 2024, Skopje, N. Macedonia

# JRC, JCS and $\phi_r$ (a constitutive model for rock joints) – applications in the last 50 years

Nick Barton

**ABSTRACT:** This highly illustrated article, with quite limited text, is basically an abstract followed by many figures and figure texts, with short introductions to new topics. It ends with a reference list that goes beyond Barton and Choubey [1] – which is where many published articles 'stop' in relation to referencing the author's work with JRC. The BB criterion which has been part of UDEC-BB since 1985 is seldom addressed. There are by now more than 60 roughness profile-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' in rock mechanics. 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 of course) 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, 12 to 14 were just to illustrate the range of surfaces that were shear tested. During this research [1], 130 natural rock joints were characterized and tested, from seven different rock types. There were 380 other roughness profiles, since three per sample. The main focus was the accuracy of the peak shear strength prediction. We used (non-damaging) gravity tilt or (horizontal) pull tests at mostly approx. 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 recording 'real' 3D behaviour, 2D profile predictions obviously are not. Those not reading past our 1977 article [1] miss scale effects and coupled behaviour, which of course depends on normal stiffness and apertures, both physical and hydraulic. Numerical modelling with the BB criterion are also missed in the widely practiced '1977 reference truncation'. The following figures give some examples of where JRC, JCS and $\phi_r$ have been used in the years following 1973/1977.

*Keywords: rock joints, roughness, profiles, shear strength, numerical modelling*

## 1. INTRODUCTION

### 1.1 *Why a roughness-and-strength criterion*

From earlier work with the shear strength of tension fractures [2] in carefully designed brittle model materials it was clear that the compressive strength of the sample was an important component of shear strength. A basic 'flat surface' ϕ-basic or ϕ-residual was also clearly relevant for planar joints with no significant roughness. Finally, there was the very important question of how to describe non-planarity and roughness. The challenge of stability when no significant roughness is present is illustrated in Fig. 1.

Figure 1. Lack of roughness and sufficient joint sets.

lapped) areas in Figure 3. At very low normal stress this damage will not occur. As a general case the components shown in Figure 4 will apply and have been confirmed even in scale-effect studies. The $S_A$ component is often of similar magnitude to $d_n$ [4].

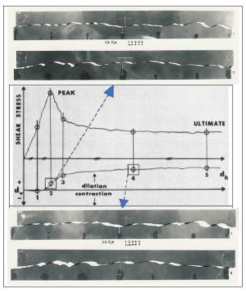


Figure 3. Model tension fractures (represented by their 2D

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ORIGINAL PAPER

# Q-Slope: Rock Slope Engineering 10 Years on

Neil Bar[1,2] · Nick Barton[3]

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**Abstract**
The Q-slope method for rock slope engineering provides an empirical means of assessing the stability of rock slopes in the field. It enables geotechnical engineers and engineering geologists to make adjustments to slope angles as ground conditions become apparent during the excavation of reinforcement-free slopes in civil engineering and mining projects. Q-slope was developed by supplementing the Q-system which has been extensively used for 50 years for characterising rock exposures, drill core and tunnels under construction. 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 (O-factor) 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 (strength reduction factor) categories have been developed for slope surface conditions, stress–strength ratios and major discontinuities such as faults, weakness zones or joint swarms. Through over 600 case studies in 36 rock types across 5 continents, a simple relationship between Q-slope and long-term stable slope angles has been established. It includes several failure modes and applies to slopes ranging from less than 5 m to more than 250 m in height. This paper discusses Q-slope application and use for the last 10 years. It presents updated Q-slope stability charts and discusses the time-dependent behaviour of rock slopes.

- Q-slope method now supported by a database of over 600 case studies, which is provided to the reader for their use and future research.
- Updated slope stability charts for empirically assessing rock slope stability and selecting appropriate slope angles.
- The Q-slope method for rock slope engineering has been adopted in over 45 countries around the world over the last 10 years.

**Keywords** Q-slope · Slope stability · Civil engineering · Mining engineering · Rock mass classification · Empirical method

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**List of symbol**

| | |
|---|---|
| $Q_{slope}$ | Q-slope value for assessing rock slope stability |
| $Q_{slope}'$ | Modified Q-slope value that does not consider orientation factors, external factors, and stress |
| $\beta$ | Long-term stable slope angle in degrees |
| RQD | Rock quality designation |
| $J_n$ | Joint sets number |
| $J_r$ | Joint roughness number |
| $J_a$ | Joint alteration number |
| O-factor | Orientation factor for the ratio $J_r/J_a$ |


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Nick Barton & Associates Rock Engineering

**REFLECTIONS ON AN UNREALISTIC CONTINUUM BRANCH OF ROCK MECHANICS – DISCONTINUOUS BEHAVIOUR COMPARISONS.** ***NB&A, Rock Eng. Jan. 2025, 29p.***

**Abstract**

There are hundreds, if not thousands of published articles in rock mechanics journals involving Evert Hoek's GSI and the Hoek-Brown equations developed for assumed rock mass 'parameterization' for 'c' and 'ϕ' and '$\sigma_{cm}$' and 'E modulus'. Yet the shear strength equations are based only on a modified intact rock strength criterion. The latter is more likely to be a valid empirical method, based as it is on thousands of triaxial tests, but it does not adequately address rock joint behaviour as affecting perhaps 90% of rock masses. Numerous authors confidently refer to 'Hoek-Brown rock masses' but in reality such may very seldom exist. After all rock masses are jointed and may be locally faulted. A major problem is that GSI, as a result of substitution of the three supporting equations for $m_b$, s and a, is actually 'utilized' respectively 16, 12 and 10 times in the three principal H-B strength equations for 'c', 'ϕ' and '$\sigma_{cm}$.' Disturbance factor D also appears multiple times, adding to the uncertainty and undermining credibility in relation to the more usual 'single-time' reference of a variable in any given equation. For instance if JRC had appeared 16 times in the Barton, 1973 peak shear strength equation for rock joints, due to 'supporting equations', or JCS 12 times, the Barton-Bandis model would have never been adopted, nor indeed published. Strangely, reference to these unacceptable and error-magnifying short-comings has not been seen in GSI-H-B literature, though may have been over-looked. (Barton, 2023). A further limitation of the Hoek-Brown (and Mohr-Coulomb) shear strength equations is that rock masses do not reach failure by overcoming cohesive and frictional strength simultaneously. CSFH is needed, preferably with joints and faults involved as in reality. Several examples of alternative UDEC-BB modelling are shown, with graphic explanations of the more physically real input data. A progressive failure CcSs alternative is proposed, which contrasts greatly with the unrepresentative methods so widely in use.

**General concerns about GSI and H-B Equations**

GSI itself is clearly a gross simplification of the complexities of rock masses, since an arithmetic scale of 5 to 100 (as with RMR) is inadequate when one considers the orders of magnitude ranges of shear strength, deformation modulus and permeability. Concerning both intact and real rock masses it is incorrect (if using Mohr-Coulomb or Hoek-Brown criteria) to add the cohesion and friction components. One must degrade cohesion (at small strain) and mobilise friction (at slightly larger strain) as in reality. (See latest review by Alidaryan et al. 2023). In the case of cliffs and mountain walls in largely intact rock the classic M-C solutions give gross errors in terms of maximum cliff and mountain wall heights, because the cohesion of intact rock is too high and is not the weakest link. The 'classic' but incorrectly interpreted crack initiation principal stress level that approximates 0.4 (+/-0.1) x UCS should have nothing to do with UCS: crack initiation and AI acceleration is tensile strength and Poisson ratio related. The standard '0.4 x UCS' (also used in the Q-system) just happens to be arithmetically close to $\sigma_t$ /v (Barton and Shen 2018). It is time to think afresh about the failure of rock masses, as existing jointing is also involved in the process. Black-box intact rock based algebra as in Hoek-and-Brown equations does not describe the process, nor do continuum analyses in general. Do the nice-looking families of curves as the complex algebra is mobilized with different GSI input actually have real value when the basic structure of Hoek-Brown equations is in question?

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Review Article



## Twenty Strange Years in the World of Rock Mechanics and Engineering Geology

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**Abstract**

Those working in the hard, to moderately hard, rocky-end of our 'geo' subjects (as opposed to our nearer-surface soil specialists) inevitably have rather more challenging site investigation tasks prior to designing and optimizing tunnelling and mining projects. The geologic complexities of jointed and sometimes faulted rock masses cannot be readily sampled and tested in the way that soil, or concrete or steel can be brought into the test laboratories. The scope and manpower needs for the myriads of site investigations, designs and constructions are huge. This is clear if we include transport tunnels for road and rail, city metro when tunnels and stations are mostly in rock, hydropower and pumped storage projects, underground and open-pit mining projects, and major dam projects in steep-walled river valleys. All of them are expensive constructions and mostly in rock. There are therefore tens of thousands of geologists, engineering geologists and rock engineers involved, and hundreds of thousands perhaps millions developing and working in the related civil engineering and mining projects. Clearly this is a trillion-dollar industry as emphasized by the reality of a recent 2.5 billion dollar claim when things went badly wrong at a major hydropower project. This article focusses on the strengths and shortcomings of methods of rock mass description and modelling of tunnels, caverns and rock slopes. It addresses in particular the problems with the Hoek-Brown GSI (geological strength index) and its widespread application in unrealistic but simply performed continuum models. These are colouful productions but are misleading our students and likely misleading the owners who pay for cut-price analyses. A radical rethink is needed if rock engineering design is to be of actual use to our numerous clients.

### Rock Mass Characterization Methods RMR, Q and GSI

Because of the difficulty of testing rock masses at sufficient scale it has been common practice for the last 50 years to use quantitative rock mass characterization methods. Following an earlier Terzaghi

in 1974, and Hoek's GSI (geological strength index) originating in 1994. RMR and GSI share a more or less equal numeric range of about 5 to 95 and have a 'joint condition' description in common, adapted from Bieniawski,1989.

2025

## 4 AR, Time, Tunnel Length and Geology Using $Q_{TBM}$

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### 4.1 Introduction

In this chapter we will focus more on advance rate (AR) than penetration rate (PR), and focus on the effect of tunnel length and geological variations, and therefore the fundamental question of time (T). Case records from 1,000km of mostly open-gripper TBM will be supplemented with a synthesis of world records for different sized TBM and their variation with time. Two fundamental questions will finally be addressed: is it correct to use TBM 'because the tunnel is so long'? And is it correct to use TBM: 'because conditions will be bad'? Logic suggests that in general the rock mass quality statistic, for instance described by the Q-value, will contain more outliers such as more serious fault zones, or more cases of exceptionally hard massive rock, the longer the planned tunnel. It is the outliers that will have the strongest influence on AR and therefore total project time. The $Q_{TBM}$ prognosis method will be described and illustrated with examples, including Follobanen where four Herrenknecht double-shield TBM drove approximately 9km each. The use of seismic refraction results and correlation of P wave velocity ($V_P$) with Q and therefore $Q_{TBM}$ prognosis will be demonstrated.

The following symbols will be used: PR (m/hr), means the penetration rate with continuous boring, while AR is the actual advance rate e.g. in m/week, m/month, m/year. U is utilization which depends on machine-type, diameter, geological conditions and on the time period considered, while time T is total time: such as 24hrs, 168hrs, 730hrs, 8730 hrs, for a day, week, month and year. If each AR time period is given in m/hr then the influence of time and tunnel length can be better understood. U is very much a time-dependent variable, and 24 hours tells little.

The two blue cubes in Figure 4.1 correspond almost exactly to the overall mean values of PR, AR, and UCS (200MPa) for the four double-shield TBM driving approximately 9km each at the Follobanen rail tunnels south of Oslo. They also apply very closely to four TBM at the Guadarrama rail tunnels which each tunneled 14km, though here the UCS was more variable and the overall AR was 0.55m/hr. ~~The~~

**2026**

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### *Strengths and Weakness of NMT and NATM and due Care with Numerical Modelling*

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**ABSTRACT**

The Q-system was developed in 1973 from case records in which just bolts and shotcrete B+S or B+S(mr) were the principal reinforcement and support, with concrete-lined sections reserved for seriously faulted rock. A majority of cases were hydropower tunnels and caverns. In 1993, Grimstad and Barton published an overdue update incorporating B+S(fr) based on Grimstad's extensive collection of mostly road tunnel case records in which the revolutionary steel fiber reinforced shotcrete was used. This represented a paradigm shift. It was already seen in 1979 in a western Norway hydropower cavern, and in 1980 it was used for a central Norway road tunnel. The term NMT was coined in a multi-company World Tunnelling article by Barton et al. (1992) and also by Grimstad and Barton (1993). NMT emphasized single-shell tunnel and cavern support as compared to double-shell NATM with its final concrete lining. In these Norwegian updates, RRS–rib reinforced shotcrete arches were already described, and their design and selection were subsequently improved by Grimstad and his former NGI colleagues. In this *keynote paper, the differences between NMT and NATM will be emphasized, including the filling of over-break in the case of NATM, but not with NMT, and the use of 'soft' unbolted lattice girders in NATM compared to the stiffer bolted RRS arches. For water control, high-pressure pre-injection with stable grouts is common in NMT, while drainage fleeces and membranes and final concrete are standard elements of NATM road and rail tunnels. The paper contains some critical comments on numerical modelling, focusing on the illogical GSI and Hoek-Brown approximations, and the assumption that 'plastic zone' modelling might justify adjustments to empirical design routines. Using UDEC-BB or 3DEC more realistic behaviour is seen.

***Keywords:*** NMT; NATM; Q; Shotcrete; RRS; GSI; UDEC-BB

**- presented at the 10th Indian Rock Conference ( INDOROCK) 5-7, November, 2025, New Delhi*

**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

2026

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**Tunneling, Mining and Rock Slope Analyses using Q-parameters and JRC-JCS parameters, in place of H-B and the Repeating GSI**


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**Abstract.** This invited opening lecture will address various aspects of the ISAMET-2026 conference topics. From Theme 1 *Mining*, these sub-themes will be partially illustrated: Rock Mechanics and Ground Control, Design of Surface and Underground Mines, Advances in Drilling and Blasting. From Theme 3 *Tunneling and Excavation*, the following will be partially illustrated: Tunneling in Different (and Difficult) Ground Conditions, Advances in Urban Tunneling, Tunnel Design and Planning and Underground Spaces and Caverns. As promised by the title of the paper, use will be made of methods developed by the author such as the Q-system parameters. These will sometimes be utilized in pairs for special purposes: Jn/Jr for predicting overbreak, Jr/Ja for predicting frictional strength. The whole six-parameter Q-formula is used when predicting tunnel and cavern single shell (NMT) reinforcement and support, both for mining and civil constructions. This will be contrasted to double-shell NATM. A truncated version of Q termed Q', just RQD/Jn x Jr/Ja is utilized for mine stope dimensioning using hydraulic radius rather than span or height. When relating Q to the deformation modulus and P-wave velocity, *both* of which are depth- or stress-dependent, the normalized form Qc = Q x UCS/100 is used. This gives an almost eight-orders of magnitude scale of rock mass quality, far closer to the geological reality than the two '5 to 95' (approx.) methods of RMR and GSI. Although the latter two are very


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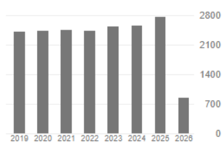


2026 (April)