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

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

1 INTRODUCTION

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

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

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

2 THE EXCAVATION DISTURBED ZONE

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

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

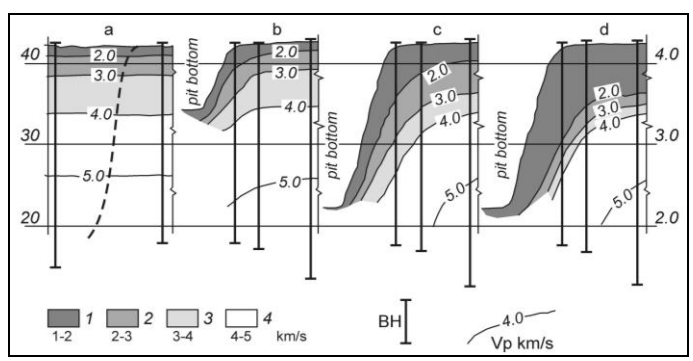

Figure 1. Cross-hole site characterization (left), and monitoring of ship-lock excavation stages. There is a one-year delay between diagrams c) and d). Savich et al. 1983.

Nevertheless, *in principal* we should be able to characterize this site, and classify support needs using the above parameters. An iterative element arises if, through our choice of a conservative rock class, we support in two stages: at half-depth and at full-depth. The former would influence the latter, and both would perhaps prevent the degradation seen with 1 year of delay in the right-hand diagram.

An EDZ of partially different character is illustrated in Figure 2, again involving cross-hole seismic, but with both improved and reduced velocity as a result of excavation. The tangential stress concentration raises the maximum velocity by almost 1 km/s about 1 m into the wall of this 5 m diameter tunnel, and the 'negative' aspect of the EDZ reduces the velocity by a similar amount. Mean velocities were 3.5, 5.5 and 4.5 km/s, the latter in the undisturbed rock mass.

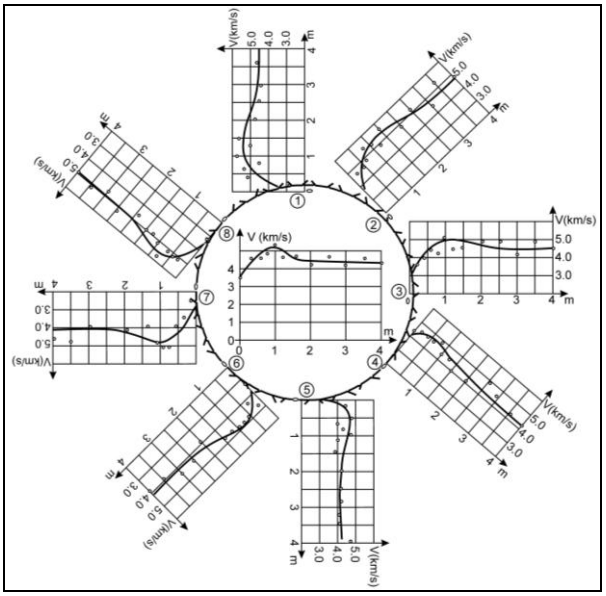

Figure 2. Cross-hole seismic performed at a pressure tunnel by Kujundzíc et al., 1970. Mean velocities of 3.5, 5.5 and 4.5 km/s are seen in the visible EDZ, in the invisible EDZ, and in the undisturbed zone.

In the present context we should refer to 4.5 km/s as the simple seismic characterization of the site, perhaps corresponding to a Q-value of 10, or less than 10 if the depth is greater than 25 m or so (see later). The 3.5 km/s 'EDZ-result' *should* be predictable by a correct application of the SRF operator in the Q-system, and perhaps by D in GSI. One would perhaps need to be conditioned or trained to assume a reduced RQD for the EDZ, as an additional measure of conservative classification, for subsequent selection of support needs.

We see immediately that there is a potential hazard in using the seismic velocity as a measure of rock quality, *if we do not take the stress level into account*. Joints that are more tightly closed by the tangential stress are likely to result in higher deformation modulus due to higher normal stiffness, each of these contributing to the 1 km/s increase in seismic (P-wave) velocity. Yet we must acknowledge that rock quality per se has not improved, unless we link this with stress level.

There might perhaps be reduced permeability at 1 m depth in the four sectors where shear stresses do not arise, and potentially increased permeability in the sectors that may suffer some shearing, mostly close to the excavation. The regular existence of shear stress in four sectors is illustrated in Figure 3, in the classic Mohr-Coulomb solutions given by Shen and Barton, 1997, and also by simple UDEC-BB models of circular tunnels, from Christianson (pers. comm.. 1985). The case shown is with equal horizontal and vertical stress, nevertheless generating shear stress.

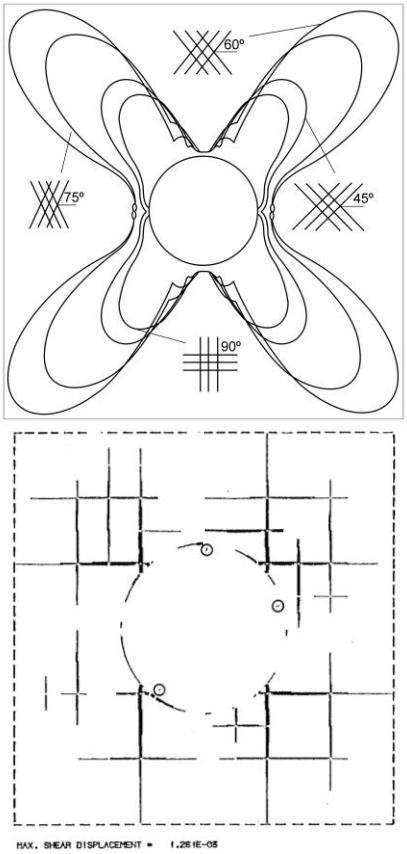

Figure 3. Sectors where shearing may affect the details of EDZ classification. Mohr-Coulomb solutions from Shen and Barton, 1997, and an early UDEC-BB model from Christianson, 1985.

3 EXTRAPOLATING QUALITY USING SEISMIC VELOCITIES

The richness of seismic and index test data for understanding the characteristics of a site are very well illustrated in the Japanese studies for the Honshu-Shikoku Bridges, that are reproduced in part, in Figure 4. The Japanese Highways rock mass classification at the lower end of the quality scale, mostly in Tertiary granites, is particularly well demonstrated, with a selection of index data including water content (S_W), porosity (n), and density.

Foundation sizes as large as 50 x 50 and even 100 x 100 m in plan, yet requiring resistance to contact pressures as high as 1 and 2 MPa, were needed beneath the sea. Seismic-velocity based methods were

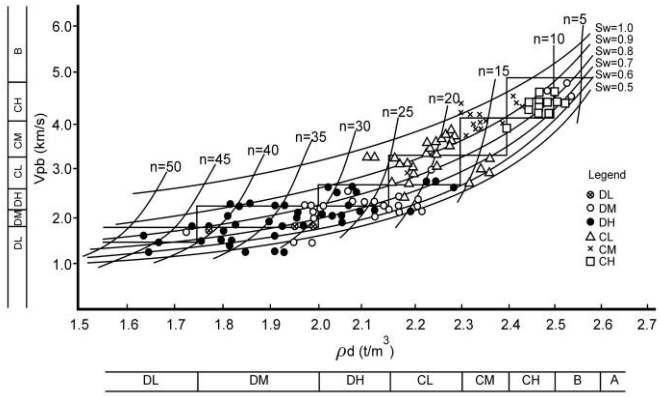

Figure 4. Rock mass velocities linked to rock class in the Japanese Highways scheme, with extension into the low class (D_L, D_M, and D_H). Honshu-Shikoku Bridges investigations, mostly in weathered Tertiary granites. Ishikawa et al. 1995.

used for extrapolating in situ deformation tests performed more readily on land, to offshore sites using down-hole pressure-meter and velocity logging. These would be examples of characterization. With subsequent exposure of such foundations in drained caisson constructions, classification would be needed to 'distribute' the previously determined velocity-deformation data per rock class.

Seismic characterization of an interbedded marl-sandstone sequence at a dam site in Italy was described by Oberti et al., 1979, using both cross-hole and sonic logging in three vertical boreholes. A rather good comparison was noted, with velocities oscillating, according to rock layer, between 4.0 and 5.5 km/s within the 30 m measured depth. However, a need to relate velocity to deformability was also required as part of the characterization, and in Figure 5 we may note an interesting problem of EDZ *classification*, with modulus depending on direction of loading and on depth of measurement, and all the measurements apparently affected by an EDZ.

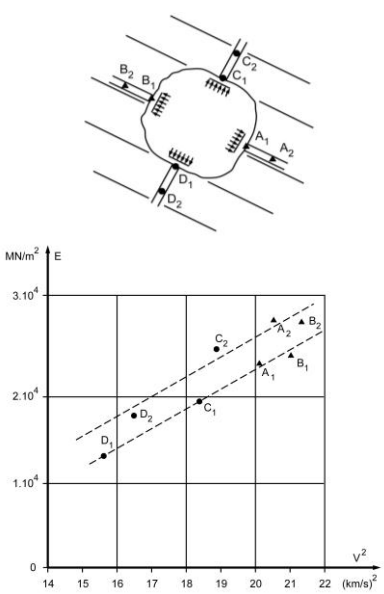

Figure 5. Classification of the EDZ of an exploratory gallery using plate-loading, sonic logging, and MPBX recordings to relate results to specific depths. Oberti et al., 1979.

A supplementary question we may ask is whether our frequently used RQD, RMR and Q parameters would be able to capture the differences seen in Figure 5.

The anisotropic (orthotropic) marl-sandstone sequence exposed in the gallery provided deformation moduli varying from about 13 to 27 GPa, and sonic velocities varying from about 4 km/s to 4.6 km/s, *apparently* at the lower end of the pre-gallery characterization results, which ranged from 4.0 to 5.5 km/s. However, here we need to be careful, as there is a velocity-stress correction that has not yet been addressed.

In the remainder of this paper we will concentrate attention on the possibilities of using one of the frequently used rock mass quality descriptors (the Q-system) to relate to seismic velocity, starting with shallow refraction seismic and also utilizing RQD. An excellent starting point is the extensive investigations presented by Sjøgren et al. (1979), who used shallow refraction seismic profiles (totalling 113 km) and local core logging results (totalling 2.9 km of core) to derive the following *mean trends for hard rocks* shown in Figure 6.

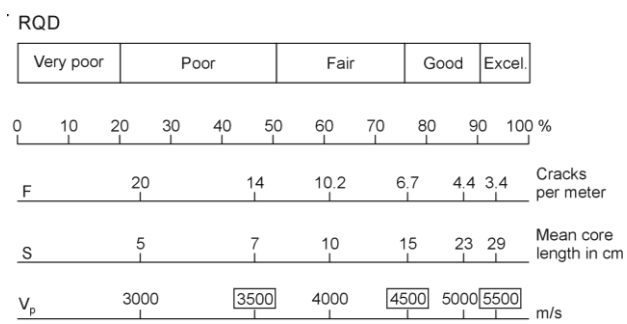

Figure 6. Shallow refraction seismic at predominantly hard rock sites in Scandinavia. Sjøgren et al. (1979) combined 113 km of seismic profiles and 2.9 km of core logging to derive these mean trends.

Using a slightly liberal smooth extrapolation (dotted lines in Figure 7), we can present this data in graphical form, and also add the Barton, 1995 addition of a Q-value scale. The latter was derived by trial and error, following several years of collecting refraction seismic velocity data at sites where core (and rock exposures) had been Q-logged, often by the author, or by colleagues.

For the velocity-Q links that follow, a useful first step is to 'linearize' the velocity-quality relation, with appended RQD and joint frequency (λ m^{-1} or F m^{-1}) data. The simple initial relation $V_P \approx 3.5 + \log Q$ (km/s) may be noted. Note that the data and linkages shown in Figures 6, 7 and 8 all apply to *shallow* refraction seismic in essentially *hard rock sites with low matrix porosities*.

A more generalized relation between Q and velocity is shown in Figure 9. This development was described in Barton, 2002, and it has been demonstrated in numerous contexts in Barton, 2006. It will be noted that Q has been normalized in order to better correlate with a wider variety of weaker (and

stronger) rocks. In essence Q is adjusted to smaller or larger values, termed Q_c, depending on whether the rock matrix has lower or higher uniaxial compression strengths than 100 MPa.

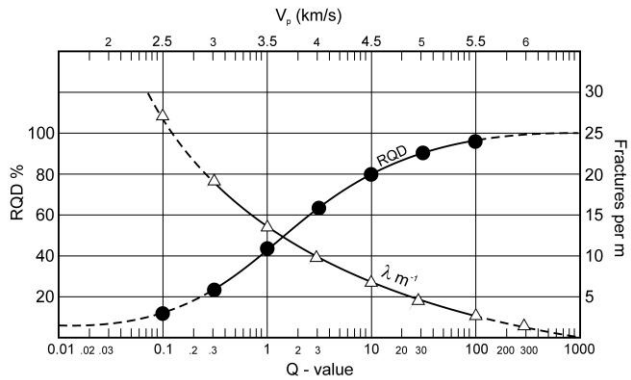

Figure 7. Mean RQD and joint ('crack') frequency trends as a function of shallow seismic refraction measurements for predominantly hard, low porosity rocks. Data derived from Sjøgren et al., 1979, with the addition of a Q-scale from Barton, 1995.

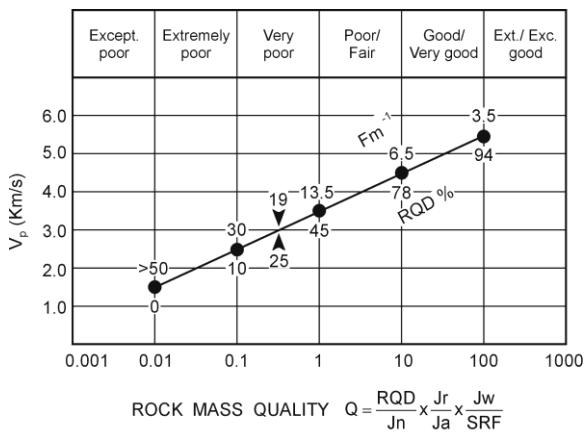

Figure 8. Hard rock, shallow seismic refraction, mean trends from Sjøgren et al. (1979). The Q-scale was added by Barton (1995), using the *shallow hard rock* correlation $V_p \approx 3.5 + \log Q$, where V_P is in km/s.

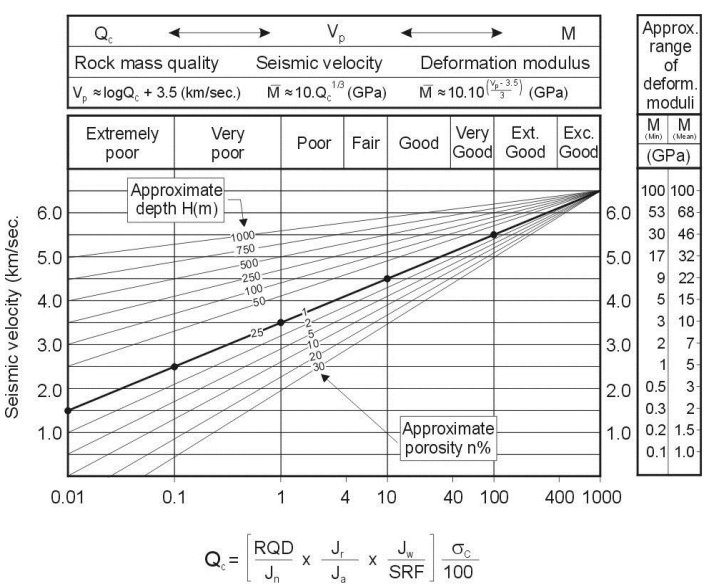

Figure 9. An integrated empirical model for linking Q-value (via Q_c) to P-wave velocity, depth, matrix porosity, and deformation modulus. Barton 2002, 2006.

It is important in rock mechanics modelling to allow for the increased moduli of deformation with depth or stress level, that will have been determined by site characterization, using seimic velocity, Q-logging, and these empirical correlations. There will be other changes with depth, especially in the context of high-level nuclear waste isolation, due to thermal over-closure of joints. The thermal expansion coefficient of the rock mass is never the same as for the intact rock material, because of this effect, contrary to recent assertion. Barton, 2007a.

Figure 9 shows that velocity-depth adjustment (+ve) and velocity-porosity adjustment (-ve) are provided, based on a wider collection of seismic cross-hole tomography data from several countries, also from weaker porous rocks such as chalks and chalk marls. Deformation modulus estimates based on plate loading and back-calculation of shaft and tunnel deformation monitoring are also appended on the right hand side, with an adjusted scale (E_{min}), for poorly explained lower values of modulus not sufficiently captured in site description, but presumably connected with the EDZ problem, as demonstrated in Figure 5.

A comparison of tunnel support pressure needs from the independently-derived Barton et al. 1974 support pressure formulation, with the deformation modulus scale of Figure 9, suggests inverse proportionality between support pressure and deformation modulus (Barton, 2002). This is presumably logical, but the simplicity is nevertheless surprising.

Here we see the ability to cross-check rock mass characterization predictions of increased or decreased moduli with depth or rock quality, with the subsequent classification needs for selecting rock support class.

4 VELOCITY-DEPTH GRADIENTS

In Barton, 2006, a large number of velocity depth profiles are given, based on a wide review of geophysics literature. It is common to read that the upper 50 m of the weathered crust are considered the most difficult to characterize in seismic investigations, and in fact the strong velocity gradients considerably affect the models of velocity needed at greater depths. By chance this complex near-surface 50 m commonly impacts dam foundations, shallow tunnels and portals, and deep foundations for bridges and large buildings.

In Figure 10 a velocity-depth diagram has been derived from the empirical model data of Figure 9, that provides individual velocity-depth gradients for specific Q_c rock classes, where $Q = Q_c$ in the case of UCS ≈ 100 MPa. This diagram helps to explain why faulted rock ahead of a deep tunnel may sometimes be 'invisible' or of such 'high' velocity, like 4 km/s that it is misinterpreted (Barton, 2006) during characterization performed ahead of the tunnel.

Such 'high' velocities may subsequently cause tunnel collapse, or trap TBM. In fact such rock is still probably displaying an important contrast to the

surrounding rock mass. In the case of soft rock, acoustic closure probably will prevent such differentiation.

In general, 'Q-jumping' will be experienced when progressing downwards to greater depth, i.e. rock qualities tend to improve, giving steeper V_P – depth (s^{-1}) gradients. This will partly be a function of reduced weathering and clay content in discontinuities, and partly due to changed rock types. In all probability, this will also apply to another 'Q' factor, the seismic Q or inverse of attenuation that geophysicists have used for at least 40 years to

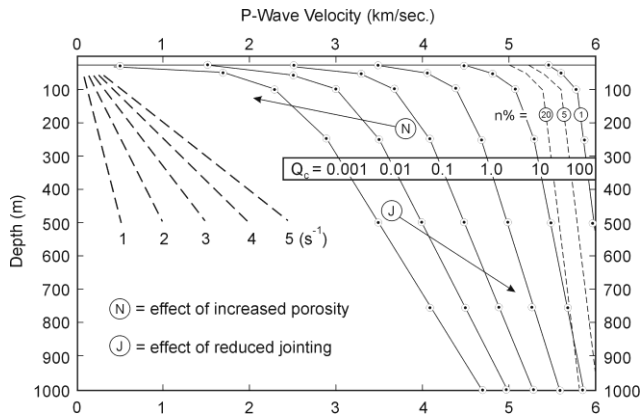

Figure 10. The depth-velocity trends for different Q_c values. Barton (2006). V_P – depth gradients (s^{-1}) tend to be in double-figures in the top 25 m due to 'Q-jumping'. This should be borne in mind when interpreting the results of V_P, so as not to over-estimate the rock quality or Q-value.

describe their concept of 'rock quality', specifically seismic quality. So seismic Q, or Q_{seis} (to distinguish it from Q_{rock}), will also be found to 'Q-jump' in the upper tens of meters. Numerous examples of Q_{seis} from different depths, and from different disciplines and effective stress levels, are given by Barton, 2006.

An example of Q-jumping in basically one (basaltic) rock type to a depth of 1000 m is given in Figure 11. This is from a mid-ocean East Pacific Rise 'wide aperture profile' from Kappus et al. 1995. This exotic location is only chosen because of the rarity of measurements to such depth in one rock type. However there is a structural complication of pillow lavas, dykes and sills within these young basalts.

The 'Q-jumping' was interpreted by Barton, 2006, by plotting the rapidly increasing velocities with depth, specifically in the first 500 m depth. Reducing porosity, increased age, and increasing hydrothermal sealing of joints and fractures seems to be the cause of the interpreted rock quality improvements.

5 PERMEABILITY FROM Q AND Q_{H20}

Pre-measurement estimation of permeability from rock mass characterisation at any given depth in a rock mass, based on core-logging is never easy, and may indeed be inadvisable, since there are potential problems such as flow-channels within the joint

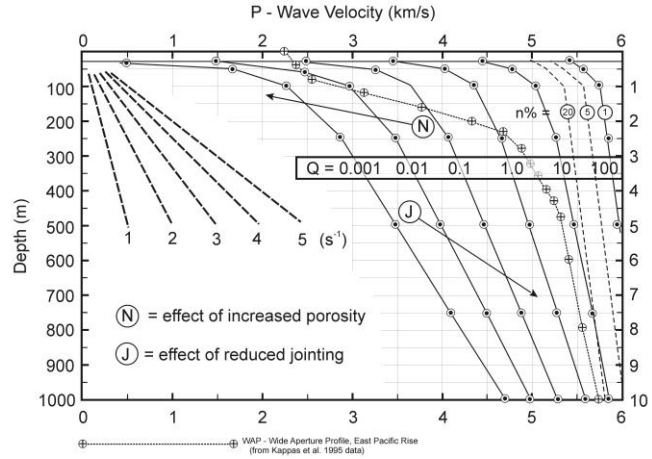

Figure 11. Rapidly increasing P-wave velocities in sub-ocean basalts to at least 500 m depth, due to reducing matrix and joint-related porosity, increasing age, and increased hydrothermal sealing of tectonic joint structures. Wide aperture velocity profiling (WAP) data from Kappus et al., 1995.

planes that have suffered erosion or solution-effects. There are also joints that may be clay-sealed, therefore having both low permeability *and* low Q-value.

For hard, low porosity, jointed rock masses *without clay*, the approximate Lugeon scales shown in Figure 12 may have some practical merit, when 'out in the field' in a tunnelling situation, and needing, for example, to assess pre-grouting needs from classification. Table 1 shows a collection of potential inter-relationships derived from this figure, where 'proving them wrong' is also useful, as anomalies may thereby be uncovered and test-needs identified. Where clay is present, a greater level of sophistication is needed than that shown in Table 1, or in this figure.

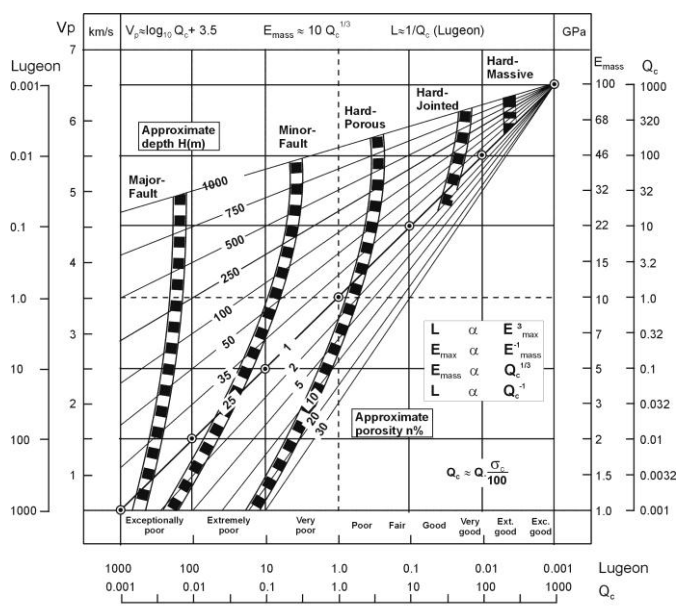

Figure 12. An extension of Figure 9 to include *very approximate estimates* of Lugeon value, strictly for the case of rock masses without clay-filling of the joints (and therefore suffering flow-blockage). For a more general case, the modified term Q_{H2O} is used. This is shown in Figure 13. Note the 'type-curves' in the above figure for e.g. 'massive rock' and 'jointed rock'. There are likely to be type curves for seismic Q as well.

Table 1 A set of inter-related geotechnical approximations that are useful when assessing results in the field. Note: $Q_c = Q \times \sigma_c/100$. Barton, 2002, 2006.

Q_c	0.1	1	10	100
Lugeon	10	1	0.1	0.01
K(m/s) ≈	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_p (km/s)	2.5	3.5	4.5	5.5

It is noted in recent work from Sweden (Svensk Kärnbränslehantering AB, 2006) that the permeabilities at the Laxemar site in the range 10^{-4} to 10^{-9} m/s, show the mean, depth-variation given in Table 2. The results are based on 100 m test scales (in other words smoothed data, in relation to that obtained with close packer-spacing).

Table 2 Smoothed permeability data from the Laxemar site in Sweden. (Svensk Kärnbränslehantering AB, 2006). Extreme values, even at 600-800 m depth sometimes range from at least 10^{-6} to 10^{-11} m/s.

K m/s	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Depth	≈ 20m	≈ 30m	≈ 50m	100m	300m	900m

Strictly considering the smoothed data, there is a certain implicit indication of improved rock quality with depth (not just stress-closure effects), which of course is supported by detailed Q-logging of cores from similar deep boreholes. Since stress levels are also increasing with rock quality, it is likely that other rock mechanics characterization data will also vary with depth, and indeed with direction.

Table 3 shows the previously described $L \approx 1/Q_c$ approximation for clay-free rock masses, and a more logical relation with permeability-in-general, that is obtained by a modified Q-calculation. The modification involves the inversion of Jr/Ja to the form Ja/Jr, whereby clay filling will result in an increase of Q_{H2O} (and a reduction of the permeability estimate), with increase in roughness or discontinuous jointing giving a similar effect (Figure 13).

Table 3 The two versions of 'Q-permeability' estimation. It should not need to be emphasised that both are approximate. Both are presently based on limited test data.

1) $L \approx 1/Q_c$ (1 Lugeon ≈ 10^{-7} m/s ≈ 10^{-14} m^2 for water at 20ºC) (*hard, jointed, clay-free*, rock masses)
Q_c = RQD/Jn x Jr/Ja x Jw/SRF x $\sigma_c/100$ (standard equation, normalized by $\sigma_c/100$)
2) *General case, with depth/stress allowance, and consideration of joint wall strength JCS*
Q_{H2O} = RQD/Jn x <u>*Ja/Jr*</u> x Jw/SRF x 100/JCS $K \approx 0.002 /(Q_{H20}\, D^{5/3})$ m/s

Figure 13 shows the depth-dependence that is presently built into the permeability estimate. As in the case of the Q_c-depth curves in Figure 10, one will usually experience curve-jumping as quality improves at depth. Q_{H2O} will tend to increase with depth, like Q_c.

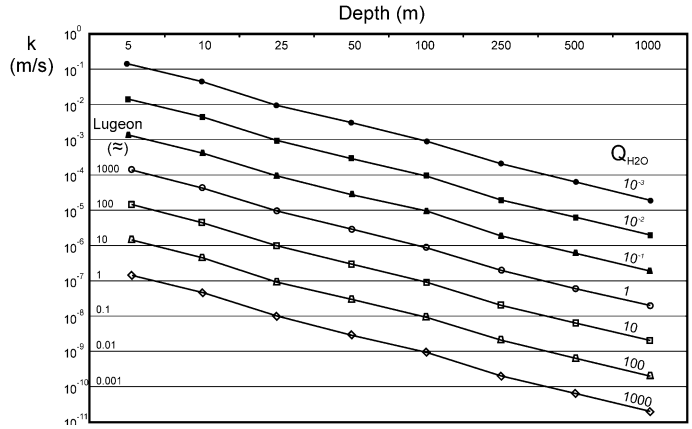

Figure 13. Depth-dependent permeability estimation using Q_{H2O} concept. (Table 3, equation 2). As in the case of the Q_c-depth curves in Figure 11, one will usually experience curve-jumping as quality improves at depth. Q_{H2O} will tend to increase with depth, like Q_c. So actual gradients may be steeper than these.

Example of Q_{H2O} estimation: Weak, well-jointed rock at 100 m depth with a low assumed joint-wall-compression-strength JCS of 10 MPa:

Regular Q-value =

$$\frac{50}{9} \times \frac{1.5}{4} \times \frac{0.66}{1}$$

= 1.4, i.e. 'poor quality'

$$Q_{H_2o} = \frac{50}{9} \times \frac{4}{1.5} \times \frac{0.66}{1} \times \frac{100}{10} = 98$$

$$K \approx \left(\frac{2}{1000 \times 98 \times 100^{5/3}} \right) = 9 \times 10^{-9} \text{ m/s}$$

(Quite low permeability despite the extensively jointed nature of this rock mass, due to nearly closed, compressible, clay-coated joint walls).

This model can be extrapolated to many kilometers depth by those familiar with the operations of SRF in the Q-system of *characterization*. Permeabilities as low as 10^{-12} m/s can be predicted where continuous jointing is essentially absent (for Q > 1000). Naturally, where SRF attains high *classification* ratings in stress-fractured EDZ, where there was previously highly stressed massive rock, a low value of Q_{H2O} would be predicted, giving an assumed much increased estimate of permeability in such an EDZ. Whether water is present in such massive rock masses at depth is another question (discussed in Barton, 2007c).

6 ROCK MASS VARIABILITY AND ITS EXPRESSION

Widely contrasting rock mass qualities that may challenge both the civil and mining professions are shown in Figure 14. The highly jointed, clay-bearing and weathered core is from a project that has not

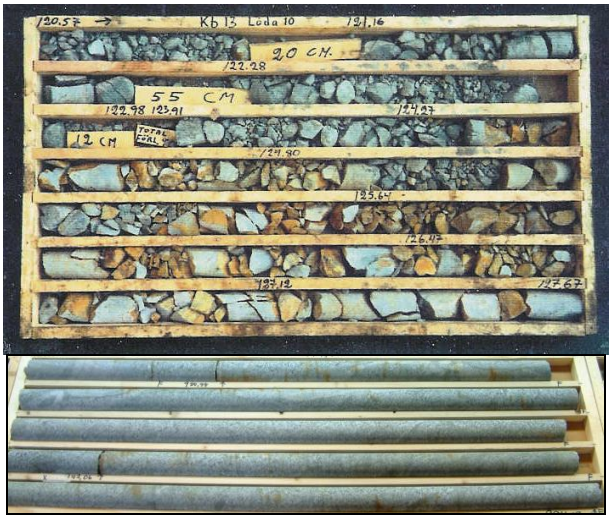

Figure 14. The contrast shown by these two core boxes suggests orders of magnitude differences in quality. Quantitative descriptions of shear strength and deformation modulus would vary by orders of magnitude as well.

been *completed* during ten years of trying. The second project may not be *started* for at least ten years. The first should already have been passing high-speed trains, the other may have high-level nuclear waste some time in the future. They are both from the same country, but may have six orders of magnitude contrast in Q-value.

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

The six-orders-of-magnitude range of Q is a *partial reflection* of the potentially enormous variability of geology and structural geology. It is probably because of the relative sensitivity of a classification that can show wide numerical variation (i.e. 10^{-3} to 10^{3}, or an even wider range using $Q_c = Q \times \sigma_c/100$), that correlation with a very varied geologic and hydro-geologic reality is achieved, using rather simple correlations. Some of course would claim that the correlations are too simple. Without this range of Q (approx. 10^{6}) or Q_c (approx. 10^{9}), correlation would anyway be more complex.

Consider for example a case from Hong Kong, where borehole investigations at a sewage tunnel project showed a 'record' intact core length of 57 m. Yet the same tunnel had an undiscovered regional fault width of similar dimensions, and delayed tunnelling by months. Clearly it is inadequate to quote respective RQD values of 100% and 0%, just as RMR values of 95 and 5 also seem inadequate, though of course have a more specific meaning. Respective seismic velocities of 6.5 km/s and 1.5 km/s, or permeabilities of 10^{-12} m/s and 10^{-4} m/s are much more descriptive of the differences. In the opinion of the writer, respective Q-values of 2000 and 0.002 also come closer to the real differences of conditions.

A particularly good illustration of the simplicity of some Q-system operations is the accuracy with which the simple ratio of Jn/Jr (the number of joint sets divided by the joint roughness) can predict the onset of over-break, or indeed the possibility of block-caving in mining. This is discussed in some detail in Barton, 2007b, and is illustrated in simple terms in Figures 15 and 16. Of course, the ratio Jr/Ja representing the friction coefficient may modify this simplicity, as clay-fillings may 'convert' a higher Jr value to something essentially unstable – if there are sufficient joint sets present.

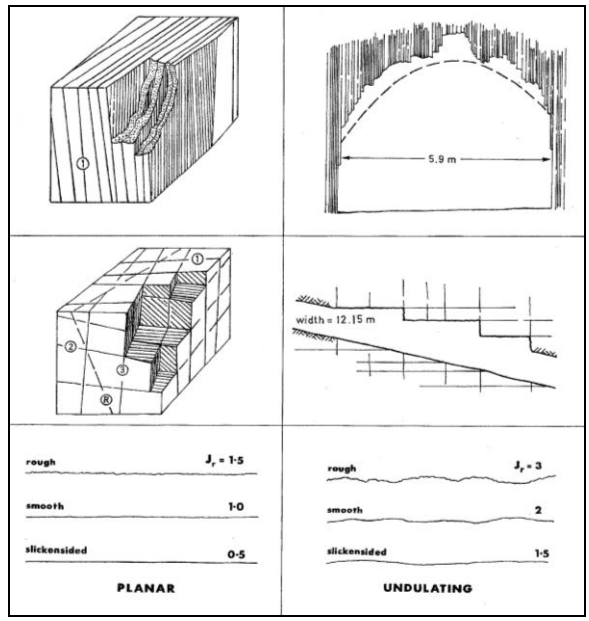

Figure 15. Two of the most important components of Q in rock mass classification are the number of joint sets, and the joint roughness. The general level of over-break and ease of carrying out classification in tunnels are also fundamentally affected by these two parameters. In block caving in mining, the ratio Jn/Jr may be fundamental for initiation of such mechanisms.

Figure 16. A station cavern showing the need of pre-injection for both water control, and for reduction of over-break. The simple ratio of Jn/Jr (about 9/1.5) is sufficient to explain over-break. Values more or less than 6 are often discriminatory.

7 INPUT DATA FROM CLASSIFICATION

A long time ago, in the late 1960's, there was a move in some rock mechanics circles to try to move beyond the confines of continuum modeling, and focus on the possible effects of jointing on the performance and reinforcement needs of rock excavations, be they tunnels, slopes or dam abutments.

Thanks to the late 1960's modeling developments of Goodman and his colleagues with joint elements in FEM codes, followed by Cundall in the early 1970's, first with μDEC, then UDEC and later with 3DEC, this focus could be fulfilled by an increasing number of rock mechanics practitioners around the world. However, utilizing these codes correctly, with realistic input data, needs experience, time and therefore budgets to match. Ironically, input data for some continuum codes seems now to be considerably more complex than for discontinuum codes, as suggested in Figure 17.

GSI-based Hoek-Brown formulations for *'simple'* geotechnical input data for the rock mass, shown in Figure 17, such as deformation modulus, cohesion and friction angle, appear to have reached 'black-box' levels of complexity, which seems to be detrimental to the idea of rock engineering, if engineering judgement is still to be exercised in this rewarding field of engineering.

Presumably as a result of time and budgetary pressures, and also the developing need to model large-scale mining problems, there has been a marked trend for using *'convenient'* continuum codes, which also have particularly good graphics representation of results. Simple software packages for handling the complex input data calculations (e.g. Figure 17) are also provided, so that a smart user might theoretically need only limited understanding of rock mechanics principles to use the codes 'successfully'.

The writer has often used the 'Chinese method' of rapidly left-thumbing from the back of a consultant's report to the front, whereby the coloured appendices of endless stress distributions and deformation patterns, can be read almost as in a film. Does all this 'colour' represent anything real? Would the numerical modelers know how to input a neglected clay seam – without 'smoothing-it-out' in a continuum approximation? Would the complex estimates of c' and φ' in Figure 17 change very much?

8 CC AND FC: THE COHESIVE AND FRICTIONAL COMPONENTS OF Q_C

In Figure 17, simple Q-based equations for 'c' and 'φ' are shown, that are actually found to be composed of each 'half' of the Q_c-formulation. They have the advantage of not requiring software for their calculation – they already exist in the calculation of the Q_c value. By chance, or because case records were defining lack of cohesion and/or friction, they appear to give realistic estimates.

$E_m\ (\mathrm{GPa}) = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{(GSI-10)/40}$	$E_m \approx 10 \times Q_c^{1/3}$
$\sigma'_{cm} = \sigma_{ci} \times \frac{(m_b + 4s - a(m_b - 8s))(m_b/4 + s)^{a-1}}{2(1+a)(2+a)}$	$\sigma_{cm} \approx 5\gamma Q_c^{1/3}$
$\phi' = a\sin\left[\frac{6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a) + 6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}}\right]$	$"\phi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$
$' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right]\left(s + m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1 + \left(6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}\right)/((1+a)(2+a))}}$	$"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right.$

Figure 17. The extraordinarily complex formulæ (left), for developing input data for some recent continuum models, and comparison to some of the less developed, and equivalent Q-based formulæ. There is no possibility to have any feel for the influence of local rock quality on the rock mass compression strength, friction angle or cohesion, when formulations require software, rather than estimation for their evaluation. The formulæ on the left can no longer be considered 'empirical', with the exception of the first equation for estimating modulus.

cohesive component:

CC = RQD/Jn x 1/SRF x σ_c/100

frictional component

FC = $\tan^{-1}$[Jr/Ja x Jw]

Examples of these rock mass component strengths are given in Table 4, for a range of possible Q-values for increasingly jointed rock masses. The

P-wave velocity and (pseudo-static) deformation modulus estimates are from the central diagonal, near-surface (25 m depth) inter-relationships given in Figure 9. They could equally well be quoted for greater depths, if more relevant to the case.

Table 4. Five hypothetical rock masses with reducing quality from top to bottom of the tabulation. Note the difference between Q and Q_c due to normalization by $\sigma_c/100$. The sensitive, logical values of FC and CC already exist in the Q_c calculation, requiring no further empiricism. Note that 'MPa' has been added to the CC term. This is necessarily preliminary, but it seems to be realistic.

RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5
10	20	1	6	0.5	5	0.008	10	0.0008	5°	0.01	0.4	0.9

Plate loading tests taken to such high stress levels that *rock mass failure* occurs are rare. However, measurement of P-wave velocity at such sites may allow tentative extrapolation to other sites through a common rock mass quality estimate. Such data can then be a source of tentative rock mass strength ($\sigma_{c\ mass}$) estimation.

Table 5 suggests compressive (and cohesive) strengths in rock masses somewhat higher than those usually assumed. They also show some implicit variation from the values set up in Table 4 (from specific Q-parameter combinations), but reinforce the idea of potentially very high cohesive strengths (e.g.10's of MPa) in competent rock masses. This table of values seems to imply very different values of cohesion to some of the earlier RMR-based estimates of cohesion for rock masses, where 'c' was generally given as < 1 MPa for a wide range of RMR. (Bieniawski, 1989).

Table 5 Plate load tests driven to failure, with corresponding velocity and modulus data for the different rock masses. Savich et al. (1974).

Velocity V_p (km/s)	2.3	3.7	4.0
Modulus E_{mass} (GPa)	1	3	15
Rock mass σ_{cm} (MPa)	4	20	50

Examples of rock masses with particularly low CC and particularly low FC values are shown in Figures 18 and 19. These require relatively more shotcrete and relatively more bolting, respectively. The original Q-system case records have apparently reflected these different needs, and the Q-parameter ratings developed by trial-and-error over a period of several months have apparently given the possibility of realistic CC and FC values. In many ways this is remarkable good fortune.

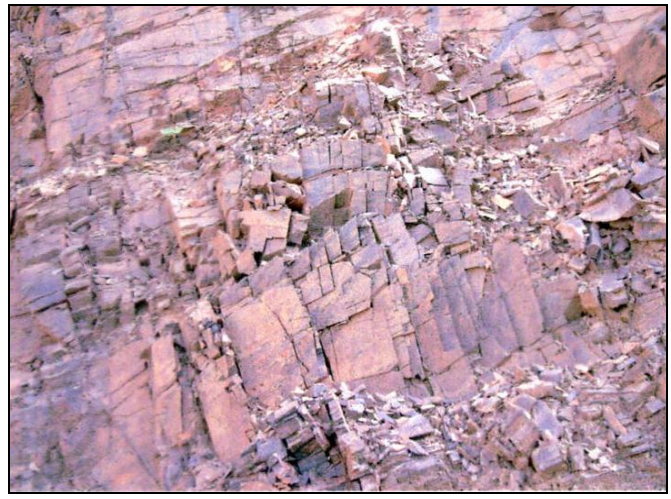

Figure 18. Example of a rock mass with particularly low CC, that would result in a rock mass classification that required selection of shotcrete rather than bolts. (e.g. Barton et al., 1974). Block-sizes are down to a few centimeters.

Figure 19. Example of a rock mass with low FC, that would result in a rock mass classification that required selection of rock bolts rather than shotcrete. Block sizes are up to several meters.

9 CONCLUSIONS

1. Rock mass classification and rock mass characterization have been distinguished in this paper by considering the rating of rock conditions *in* an excavation, and *before* excavation, respectively. Links to seismic velocity have been used to illustrate this distinction.

2. The excavation disturbed zone that should be classified by standard measures of rock quality, such as RQD, RMR or Q presents a challenge, as velocity and deformability may be different in different sectors, due to anisotropy of stress and structure.

3. The wide range of Q-values (0.001 to 1000) reflects *to some degree* the very wide range of geological conditions, and is probably responsible for the fact that empirical equations based on the Q-value or on Q_c are particularly simple.

4. The Q-parameters Jn and Jr are very useful for evaluating over-break potential and also cavability in mining. When Jn/Jr ≥ 6, significant overbreak will tend to occur, unless limited by timely support close to the excavation face. A modifying factor is of course the ratio Jr/Ja representing frictional strength.

5. An integration of the Q-value with seismic and permeability data has been developed because there is a limit to how many boreholes can be drilled, how many cores can be logged, and how many permeability tests can be performed. The ability to extrapolate these 'point sources' of information helps to project rock quality classes along a tunnel, to different parts of a large cavern or mine, and to different localities of a future project.

6. Due to the effect of increased stress at greater depth, it must be expected that deformation modulus and seismic velocity will both increase. Eventual sonic logging or cross-hole tomography ahead of a tunnel face may therefore give a higher velocity than the rock quality may suggest. Failure is possible.

7. Steep velocity-depth gradients, and a trend for steep permeability-depth gradients in most near-surface rock masses can be related to specific estimates of rock quality. It is generally found that 'Q-jumping' or improved quality with depth is needed to match the increasing velocities and the reducing permeabilities.

8. The most simple approximation for permeability is that the number of Lugeon: $L \approx 1/Q_c$. This is strictly for the case of clay-free, jointed, low porosity rock masses. A more generally applicable approximation using the term Q_{H2O}, uses an inverted Ja/Jr term and 100/JCS to give a better link to permeability.

9. Input data for continuum codes, with links to GSI, have recently acquired remarkable complexity and require software for evaluation of the strength components c and φ. Mohr-Coulomb and Hoek-Brown strength criteria may need to be modified to the form 'c *then* tan φ' for more correct determination of so-called 'plastic zones', since cohesion will tend to be degraded before friction is fully mobilized, due to different magnitudes of strain, especially when massive rock fails.

10. The terms CC and FC from the Q_c calculation show promise in giving a direct preliminary estimate of the magnitudes of rock mass cohesive and frictional strength. Logic would suggest that these components should also not be *added* in an eventual failure criterion.

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INVITED PAPER

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

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

1 INTRODUCTION

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

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

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

2 THE EXCAVATION DISTURBED ZONE

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

Subsequent cross-hole seismic between supplementary holes shows the increasing development of an EDZ actually much better than our rock mass

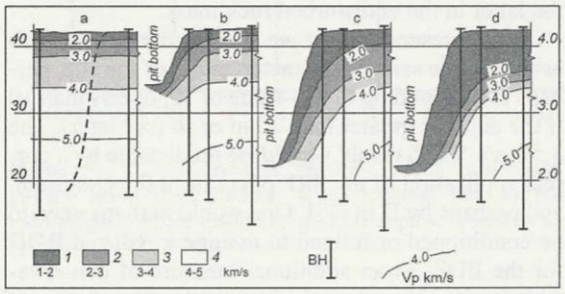

Figure 1. Cross-hole site characterization (left), and monitoring of ship-lock excavation stages. There is a one-year delay between diagrams c) and d). Savich et al. (1983).