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Understanding the need for pre-injection from permeability measurements: What is the connection?

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ABSTRACT

Pre-grouting ahead of tunnels has three main functions: to control water inflow into the tunnel, to limit groundwater drawdown above the tunnel, and to make tunnelling progress more predictable since rock mass quality is effectively improved. It helps to avoid settlement damage caused by consolidation of clay deposits beneath built-up areas, since towns tend to be built where terrain is more flat, due to the clay deposits. There are so many instances of settlement damage that the profession needs to take note of the need for high-pressure pre-grouting, to use micro-cements and micro-silica additives. The use of high-pressure injection may cause joint jacking, but this is local in extent when the rapid pressure decay away from an injection hole is understood. This effect is variable and depends on the geometrical parameters of the joints. This pressure-decay advantage must not be violated by maintaining high pressure when grout flow from the injection hole has ceased. The latter can cause damage to the grouting already achieved. Simplified methods of estimating mean hydraulic apertures (e) from Lugeon testing are described, and from more sophisticated three-dimensional (3D) permeability measurement. The estimation of the larger mean physical joint apertures (E) is based on the joint roughness coefficient (JRC). Comparison is then made with the empirical aperture-particle size criterion $E > 4d_{95}$, where d_{95} represents almost the largest cement particle size. Depending on joint set orientations and on the available micro-cements, the decision must be made of which range of pre-injection pressure should be aimed for, using successive reductions of the water-cement ratio w/c. More simple estimation of permeability, also with depth dependence, can be made with the empirical link between a modified rock mass quality Q and permeability, which is termed Q_{H2O}. The value of this parameter can be based on core-logging or in-tunnel face logging. The 3D before-and-after-grouting permeability measurements have been used to justify the quantification of rock mass quality Q-parameter improvement, and the consequent increases in expected P-wave velocity and deformation modulus, for application in dam foundation treatment and its monitoring.

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1. Introduction

As is well known to all working in rock mechanics, the behaviour of rock masses, whether shear strength or deformability or the shear strength and permeability of the component rock joints, has been a life-long interest and pre-occupation of Professor Ted Brown, in between his remarkably active academic and professional career, and his strong involvement in mining and dam engineering in particular.

In this paper, the writers have technically related interests. By focussing on pre-grouting, we simultaneously address practical ways of *improving the properties* of rock masses where strength, permeability and deformability are considered to be inadequate for problem-free drill-and-blast and tunnel boring machine (TBM) tunnelling, and presently inadequate for the desired range of properties in dam foundations where relative impermeabilization (somewhat lowered Lugeon values), increased modulus, reduced uplift pressures, and stable abutments are the principal goals.

With a long-standing rule for injection pressure gradients of approximately 0.23 bar/m depth (1 bar = 0.1 MPa) for dam foundation grouting in the USA, but usually higher elsewhere (Quadros and Abrahão, 2008), it is clear that there will be reactions when

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50–100 bar pre-injection pressure is recommended by an experienced contractor for a stretch of tunnel whose 20 m depth suggests only 5 bar. The reasons for performing high-pressure (50–100 bar) injection when pre-grouting ahead of tunnels is that water inflows have to be controlled, perhaps down to 1–2 L/min/100 m. Permeabilities lower than 10^{-8} m/s, or lower than 0.1 Lugeon are implied – and these are also achieved when owners and consultants become aware of the abilities of sealing with stable ultrafine and micro-cements using necessary additives like micro-silica and plasticizers. Many experiences (and court-case preparations) 'abroad' suggest a rather slow-moving acceptance, perhaps fed by fundamental but quite simple misunderstandings of pre-injection, to be discussed later. We will not address *post-injection*, except to point out the '3 × 10' rule: ten times more expensive, ten times longer, and ten times less effective. Experience supports this.

It has been found from recent Norwegian tunnelling projects, often but not always in good quality rock, that high-pressure pre-injection may be fundamental to a good result, i.e. much reduced inflow and improved stability. The pressures used are significantly higher than those that have traditionally been used at dam sites, where in Europe, Brazil and the USA, the maximum grouting pressures (for deep dam foundations) have been limited to about 0.1, 0.05 and 0.023 MPa/m depth (Quadros and Abrahão, 2002).

An important detail, still ignored by some, is the fundamental mismatch of the mean physical and theoretical hydraulic joint apertures, which was graphed in Barton (1972) and emphasised again following coupled in situ tests in subsequent work with colleagues (Barton et al., 1985). The larger-aperture data were updated by Quadros in Barton and Quadros (1997). The concept is by now widely accepted. The above included experiences with a unique 8 m^3 volume in situ block test in 1980–1981, where the mismatch of joint closure and change of hydraulic aperture were confirmed, also as a function of temperature. As will be discussed later, the mismatch of the physical groutable aperture (E) and the smaller flow-interpreted hydraulic aperture (e) has an important impact on the optimal size of the chosen micro-cements in the case of high-pressure pre-grouting of tunnels.

During the late-nineties, the second author was conducting three-dimensional (3D) in situ permeability tests in distinct types of rock masses linked to dam foundations and abutments and mining works, usually confirming the strong effect of scale and anisotropic characteristics in the analysis of the permeability tensors (e.g. Quadros, 1992; Quadros and Correa Filho, 1993; Quadros et al., 2000). A significant rotation and reduction in magnitude of permeability tensors was experienced, in unique before-and-after grouting 3D tests in a dam abutment in Brazil by Quadros and Abrahão (2002). This field-work had followed extensive laboratory permeability testing in individual tension fractures in granites and basalts (Quadros, 1982).

The Q-system parameters, which will figure later, fortunately include both the number of joint sets, and specific joint properties (joint roughness number J_r and joint alteration number J_a) and of course an estimate of tunnel inflow. It was therefore possible to subsequently interpret these permeability tensor rotations and reductions in magnitude as representing successively sealed joint sets (effectively reduced joint set number J_n and effectively increased rock quality designation (RQD)), and improved ratios of J_r/J_a for the remaining (perhaps) unsealed joint sets, and of course an improved joint water parameter J_w rating. The combination of permeability reduction and Q-parameter improvement (perhaps by a factor of minimum 10 to maximum 100) can be carried directly into the world of TBM tunnelling, where severe delays in faulted rock are known to be Q-value related (Barton, 2000, 2013) and where needed pre-injection not only displaces water, if successful, but improves the strength and reduces the deformability of approaching weakness zones or faults. TBM progress is sometimes made possible by pre-grouting, where it otherwise is stopped.

2. Settlement damage

A life-time associated with tunnelling projects has demonstrated, at irregular intervals, a range of damage associated with uncontrolled tunnel inflow. We need look no further than the authors' home cities Oslo and São Paulo. As one might expect, damage occurred in the face of ignored advice about the need for pre-injection, which was given by several consultants. Too close to home, one case involved our slightly damaged row-house in Brazil, together with less fortunate neighbours living about 50–150 m from the new and much too shallow (Yellow Line) metro tunnel. The settlement developed into hundreds of metres radius-of-influence and maxima of 1 m of settlement along two nearby roads, which have since been filled-in. There remain some abandoned homes close to the line of the tunnel due to extensive damage.

In Norway, a drained lake and rotated trees were experienced by tourgoers above the Romeriksporten Tunnel north of Oslo. There were also numerous damaged houses, and the need of recharge wells was demonstrated in the case of this long rail tunnel. In Hong Kong, China, a distant sub-sea water-table drawdown with office and container-port damage occurred, a remarkable 1 km ahead of a sub-sea, sub-pollutant, sub-clay-layer TBM due to sometimes insufficiently controlled inflow of water associated with long delays in fault-zones. Systematic pre-injection in this case was the only way to progress through a wide regional fault zone whose rock mass Q-quality was much worse sub-sea than that under an adjacent heavily populated valley in down-town Kowloon. The wide regional fault zone defied several attempts of core-sampling in its sub-sea low rock mass quality $Q = 0.01$ un-grouted condition, yet the 3 m diameter TBM could later penetrate this same zone because of the continuous pre-injection ahead of the TBM – proving the benefits of pre-injection.

Unfortunately, double-shield TBM, even with (or because of) reliance on bolted and gasketed PC-elements, have demonstrated many instances of draw-down of ground-water levels up to 1 km distant, due to under-estimation of permeability, sometimes existing in the form of sub-horizontal and unexpected connectivity in otherwise good quality 'vertically-fractured' crystalline bedrock. The problem is the 'delay' of PC-element installation in the tail-shield, 15–20 m distant from the tunnel face. A normal one to two days delay can be extended in time if a troublesome weakness zone, or fault, or hard mixed-face is involved, the latter maybe causing cutter damage.

The most remarkable ground-water draw-down due to a TBM project known to the authors reached to 2.9 km. It was apparently transmitted along the fractured zones beneath two intersecting valleys in Sri Lanka. Thousands of houses were damaged, and hundreds of wells dried up. A small river relied upon by farmers also disappeared. There are of course thousands (maybe tens of thousands) of examples of draw-down damage caused by tunnelling worldwide.

Why are so many tunnel designers and contractors so optimistic about the 'zero' risk of settlement damage? There is after all, an expected and logical high frequency connection between the location of towns and cities in the flatter, sediment-and-clay-filled areas, under which tunnelling is needed. This is exactly where great care is essential (i.e. pre-injection), in preference to the optimistic: 'We do not expect settlement damage'. The need for good pre-injection routines should be clear to all civil engineers who have seen the suffering of city commuters, house-owners, building occupants, and sometimes farmers, forest tourgoers, and even boat-park recreationists, caused by tunnelling that has triggered ground-water draw-down. Furthermore, and usually distant from

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cities, TBM need not get stuck in fault zones beneath mountains, if good probe drilling and timely impermeabilization (pre-injection) routines have been employed.

New dams of course must guarantee their intended function with the expected foundation treatment using structural-geological focussed grout curtains, following equally focussed permeability measurements, preferably using variously-angled boreholes from an early stage in the site investigation. River valleys where dams are almost always located are there for a reason: more pervasive fracturing and faulting were and/or are inevitably exploited by rivers.

3. High-pressure pre-injection

Water under pressure is one of the most difficult rock mass conditions, which urgently needs control when driving tunnels. If significant water inflows are suddenly experienced at the new tunnel face, the needed control is already compromised, as post-injection has to be at lower and therefore less effective pressures. Even sealing of leaking bolt holes is time-consuming and frustrating work as water 'always finds a way in'. The water surrounding the tunnel is drawn down to atmospheric pressure in an irresistible manner, so overlying clayey materials may then consolidate due to lowering of the pore pressure. Near-tunnel clay-filled discontinuities can even be eroded, possibly allowing rock-blocks to loosen. In faulted rock, this can cause sudden large-scale in-rushes, which have claimed lives on occasion and delayed TBM projects for months, or even forever, needing drill-and-blast tunnel completion.

When high-pressure, high-volume water inflows are already experienced at the face of a tunnel, it is 'too late' to do the pre-injection that should have been done, if timely information had been obtained from probe drilling ahead of the face. The highest pre-grouting pressures, now reaching 5–10 MPa, are not *only* needed in the mostly hard jointed rocks in Norway, where there are more than 5500 km of rock tunnels. One of the problems in Scandinavia, following retreat of the last 2–3 km thick ice-sheet, is the elevation of marine clays and other sediments far up many valleys, due to prior sea-levels 200–300 m above today's coasts. Sweden and Finland, with fewer tunnels and fewer mountains, have similar challenges from the consequences of isostatic rebound, with equal needs for good pre-injection practice.

Injection pressures as high as 5–10 MPa are nevertheless still rare, partly due to conservatism, and worry about *hydraulic jacking*. With incorrect technique such as holding pressure constant when there is no longer any flow of the grout, this concern is warranted; even *hydraulic fracturing* can occur. In the case of weaker rock types, it is unlikely to be necessary to use such high pressures, as deformability and permeability are likely to be much higher before performing the pre-grouting.

Recently, three rail tunnels on the west side of Oslo, with a total length of 12 km, were all systematically pre-injected, following thorough pre-investigations and pre-grouting analysis based on specialized core logging. The highest pressures used (5–10 MPa) will have significantly and deliberately exceeded assumed local minimum rock stress. In other words, the high pressures will have hydraulically jacked the joints, but only in the immediate neighbourhood of the injection holes. The radial reduction in pressure, linear or even logarithmic, is important. It is indicated in Fig. 1.

In the case of pre-injection in hard competent rocks, the high pressures have proved to be *necessary* for achieving dry tunnels, where 1–2 L/min/100 m inflow is the goal. Less stringent 'environmental limits' like 8 or 12 L/min/100 m have proved insufficient for the *in-tunnel* environment, and allow continued dripping, which is damaging to rails and may allow icicles to form in cold climates. In the case of unlined pressure tunnels, the highest pressures (almost 1000 m in Norway) are *possible* because of a *surrounding minimum rock stress* that in this case needs to be *greater* than the maximum water pressure of 10 MPa. This obviously requires deep tunnels, even as much as 1000 m, since the minimum rock stress is sometimes measured as low as $0.4\sigma_{vertical}$, where $\sigma_{vertical}$ is the estimate of vertical stress based on rock density (Barton et al., 1987).

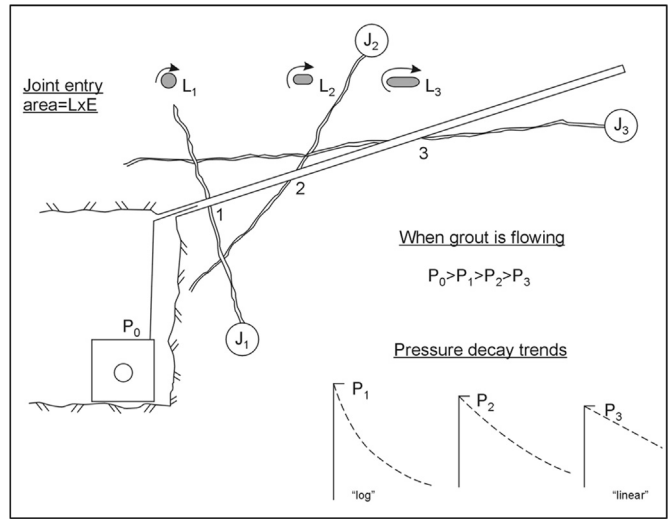

Fig. 1. Conceptual pre-grouting hole (one of 30–60 holes) showing different joint intersection angles. Experiments with water flowing into a joint have shown that more than half the joint-entry pressure is dissipated approximately 1 m distance from the hole (Cruz et al., 1982). Greater head losses can be expected from the much more viscous (frictional and cohesive) Bingham fluid when the grout penetrates in a rough joint. Head losses are high when the cement grout is travelling into each rock joint, due to extra entry losses, contributing also to reducing the extension of the grouting, which is one of the reasons for using high pressures. Joint direction J_1 will limit the size of the chosen grout and will be more difficult to grout than J_2 or J_3. Joint entry areas are $L_{1\ \mathrm{to}\ n}$ multiplied by e for water, but a larger $L_{1\ \mathrm{to}\ n}$ multiplied by E for grouting. $E > e$ due to joint roughness coefficient (JRC) (Barton et al., 1985).

There are practical reasons why such high-pressure pre-injection is both necessary and possible. Firstly, the joints may be too tight (e.g. 50–100 μm) for most grout particles to enter and flow the desired few metres. Secondly, while grout flow is occurring, there is a strong *linear to logarithmic pressure decay* away from each injection borehole (Fig. 1). It allows good deformation control despite the high pressures, when flow is still occurring.

According to Bliss and Rushton (1984), the effect of permeability tests performed at intervals of 3 m length along the borehole is restricted to an approximate radius of 10 m around the borehole. This suggests that due to the head losses occurring when the fluid penetrates the intersecting rock joints, the permeability estimated from water pressure tests is valid only for a limited length into the rock mass.

Analysis of the effective hydraulic head versus the distance from the borehole walls based on laboratory experiments was reported by Quadros and Cruz (1995). The experiment showed that the pressure is quickly dissipated at small distances from the borehole wall and that the distance depends on the amount of flow, the flow regime and the Reynolds number (Rissler, 1977; Quadros and Cruz, 1995). Based on head-loss measurements, only about 50% of the injection pressure may remain at 1 m radius. This agrees with theory.

In fact, the effect of pressure dissipation is much higher for the penetration of cement grout into the joints. There are a series of head losses acting when a viscous fluid penetrates into a rock joint: the head losses in the entrance, head losses due to the effect of roughness in the joint walls, head losses due to the variation of the temperature gradient with time, head losses due to the influence of rock-to-rock contacts in the joint and so on. Collectively these have

a strong influence on the pressure decay when a water pressure test is performed in a borehole and especially, when a cement grout is injected in a specific length of the borehole.

Prior to deciding on, or avoiding, the use of high pressure or lower pressure when planning the use of cement grouting to strengthen the rock mass in the surroundings of a tunnel or in the foundation of a dam, it is advisable to consider many factors like the quality of the rock (high pressures normally are used for high-quality rocks), the depth of the tunnel or the extent of the grout curtain in the case of a dam, and of course the number of fractures appearing in the volume of the rock mass of interest.

It is surprising to see from the case records and interviews with Finnish, Swedish and Norwegian pre-grouting specialists, as reported in the study of Tolppanen and Syrjänen (2003), how often pressure is held constant at the end of grouting, when the highest pressure is usually reached, due to lower water/cement (w/c) ratios such as 0.5. Pressure is held for 5 min in many cases (potentially damaging at the beginning) and for just 1 min in few cases (maybe already too long). Of course, grout flow-back should be prevented, but following lowered pressure in the now closed high-pressure grouting hose(s).

In the case of pre-injection ahead of tunnel-faces, local and desirable *hydraulic jacking* will tend to occur in a limited radius that also depends on the depth of the tunnel. Undesirable and damaging *hydraulic fracturing* will not occur, unless pressure is held constant when no flow of grout is occurring, due to a setting grout front. Fig. 2 illustrates this danger conceptually. Pressure must be reduced when flow stops. Rather surprisingly, this important detail does not seem to be appreciated, so damage may occur. Of course, grout flow-back needs preventing, but no more than this.

In dam engineering, depending on the depth of the grout curtain, the pressure is normally lower than that used in tunnelling, and is guided by the type of dam, with high arch dams obviously requiring quite high excess pressure due to valley/gorge bottom rock stress concentrations affecting the normal stresses acting on the different joint sets. Unlike the single 'umbrella' (actually a diverging cone) used in pre-grouting ahead of tunnels, the deep grout curtain for a high dam may need two to three offset rows of holes, drilled from galleries in the abutments and within the base of the concrete dam. Outside rows are grouted before the deeper central curtain. An excellent recent review of (mostly) Chinese practice is given by Chen (2015).

To emphasise the role of pressure-drop-while-flowing, the following tunnelling example is illuminating – also illustrating the advantage of a 'blocker-screen' in some cases. A shallow urban tunnel in phyllite, with 5 m of rock cover, was injected at invert level to a final pressure of 6.5 MPa, and to 5 MPa even in the shallow depth of the arch Barton (2004), with personal communication from pre-grouting practitioner B. Klüver (see also Klüver and Kveen, 2004). However, the establishment of an outer 'blocker' screen was necessary for this shallow location. The reality is that while grout is still flowing, deformation in the rock mass is limited to local, near-borehole joint aperture increases, due to the steep pressure decays just mentioned (See Fig. 2). Without this pressure dissipation mechanism, cases such as the above would obviously be damaging to overlying rock and soil. Application of such high pressures is discounted in most countries, maybe because of incorrect practice and failure to appreciate the above flow-dependent pressure decay.

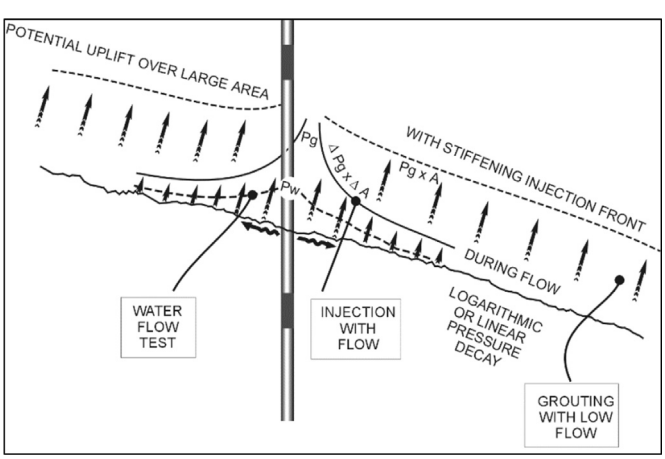

Fig. 2. Approximately logarithmic decay of pressure, which is accentuated when joints are rough (high JRC), means that hydraulic jacking can be controlled and hydraulic uplift over larger areas can be avoided, by preventing the maintenance of high pressure when flow has ceased. This is contrary to most 'standards' which should be changed.

The presumed effects of local high injection pressures are to cause local joint aperture increase, probably in the region of fractions of a millimetre in competent rock. Local grout-take is usually in the range of about 1–6 L/m^3 of rock mass, in the assumed 5–6 m thick grouted zone 'cylinder'. As shown later, this implies that there is some joint opening occurring, in response to the near-borehole high pressures. However, in weathered terrain, and in weak rock, grouting pressures need to be limited, as grout-takes will be significantly higher. Careful observation is essential in such cases, to check that the tunnel face stability is not compromised, nor outflow occurring from the face due to too high pressure. Besides use of lower pressure, packers can be located one or two metres deeper behind the tunnel face if necessary. Pre-grouting needs good confinement, so that applied pressures can be high.

Pre-injection of the rock mass some tens of metres ahead of the face, as illustrated in Fig. 3 (top) and Fig. 4, using high pressure, has been shown to 'normalise' tunnelling progress, largely removing surprises, and making tunnelling possible, even when tunnelling through serious fault zones. Experienced drill-and-blast contractors, using three-boom jumbos, can drill and inject some 1–1.5 km of pre-injection holes (a typical 'umbrella') in 24–30 h.

4. Lugeon tests for grout estimation

Fig. 5 shows how one can make a preliminary estimate of the mean spacing of *water-conducting* joints, using Lugeon tests. It is assumed that the water-conductors follow a Poisson distribution down the borehole, as indicated by Snow (1968). Experience shows that the water conducting joints may number far less (such as 1/10th) of the breaks/joints logged in the core. The key input is the percentage of zero (or measurement-limit) water inflow stages. Continuous test stages are needed, not 'focussed' testing of only jointed zones, as sometimes seen. Packer spacings of 3 m, 5 m or 7 m are typical civil engineering examples. A key simplifying assumption is that the water conducting joints *can be approximated* as a cubic network of parallel plates, as shown in Fig. 6. There are many more joints found in cores recovered from most rock types, due to limited connectivity. These are shown in a symbolic manner, as 'rough' joints occurring with greater frequency, between the more continuous hypothetical conducting planes.

The reality of water-flow and therefore grout-flow may obviously be anisotropic and will be *very much less* homogeneous than that shown in Fig. 6. Because of stress transfer across joints and therefore points of rock-to-rock contact, there will also tend to be tortuous flow between the joint walls, especially when JRC is higher and Reynolds number is lower. Flow may be concentrated in channels. However, channels alone do not explain the significant permeability of many, even most rock masses in the upper few hundreds of metres where typical civil engineering tunnels are found.

The *average* physical aperture (E) of individual joints which are potentially groutable follows the rule $E \geq e$, where e is the *mean*

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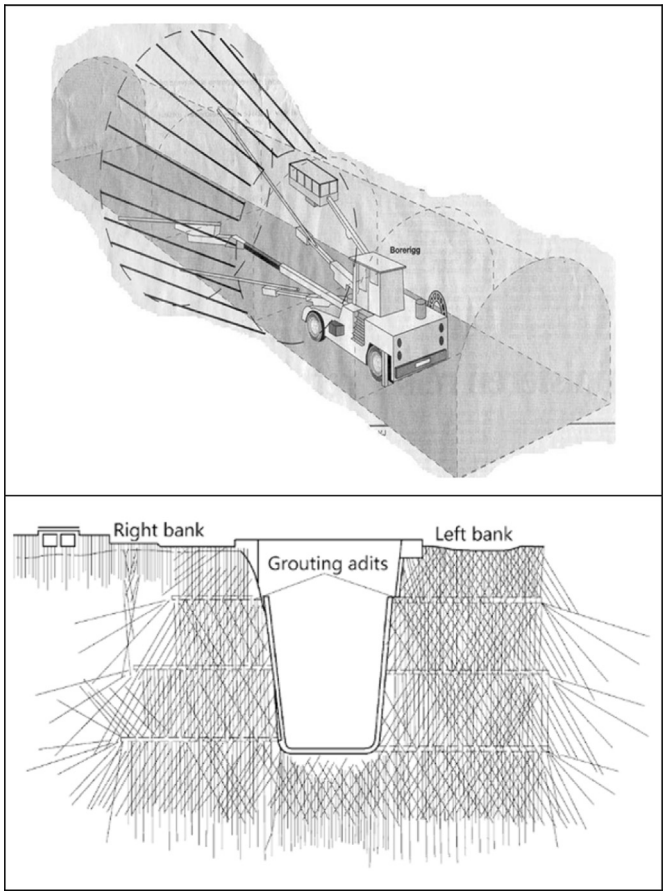

Fig. 3. Contrasting levels of effort to impermeabilize the rock mass around a tunnel, and that surrounding a 150 m high arch dam. Some 30–60 holes may be used for tunnel pre-grouting, as shown in the photo from Bærum twin-track rail tunnel, Norway (Fig. 4). From 20 h to 30 h for the drilling and injection remains the goal for an experienced contractor. It is essential to have truck-mounted and computer-steered equipment, with bulk storage and duplicate mixing facilities on board (Hognestad and Frogner, 2005; Chen, 2015).

Fig. 4. Pre-injection in 110 m^2 tunnel using >70 injection holes in shaley limestone in Bærum Tunnel, Norway.

hydraulic aperture. The ratio of apertures depends on joint roughness JRC (shown quantified in the Appendix). In practice, E is (of course) a distribution of apertures, even in one joint plane, and due to both normal stress and roughness differences, one set will usually be easier to grout than others, causing q-p 'flow-pressure plateaux' when raising pressure (p) and therefore flow (q) during pre-injection.

Assuming for the present that the cubic law is sufficiently valid for engineering purposes and that we can ignore nonlinear or turbulent flow, we can write permeability $K = e^2/12$ for one parallel plate, and therefore, for one set: $K_1 = (e^2/12)\,(e/S)$, where S is the mean spacing. Snow (1968), and also Louis (1969), further assumed that the 'rock mass permeability' would consist on average, of flow along *two of the three sets* of parallel plates. At this point in the 'chain of assumptions', one may insert an adjustment to allow for anisotropy. Therefore, the 'two of three joint sets' recommendation (see 2 times in Eq. (1)) can be adjusted, thereby channelling more or less flow, into a particular set of conducting joints. Calculated hydraulic apertures will then change.

Based on the assumption of just two active sets (with three mostly less conducting sets when the gradient is rotated away from parallel flow in two of them), the rock mass permeability estimate k_{mass} can be written as

$$K_{\text{mass}} = \frac{2e^2}{12}\frac{e}{S} = \frac{e^3}{6S} \tag{1}$$

Making further 'engineering' simplifications that 1 Lugeon $\approx 10^{-7}$ m/s $\approx 10^{-14}$ m^2 (at 20 °C), we obtain 1 Lugeon $\approx 10^{-8}$ mm^2, and can therefore write the simplified relation:

$$e \approx (L \times 6 \times S \times 10^{-8})^{1/3} \tag{2}$$

where e and S are in mm, and L is the average number of Lugeon. (Each of the above applies to a given structural domain, to the whole borehole, or to a specific rock type). In Fig. 7, five type-curves of e versus S for given Lugeon values are shown. A typical range of conductor spacings of $S = 0.5$–3 m has been assumed, based on evaluation of Eq. (2).

5. Apertures *e* and *E* and particle sizes for grouting

The mean hydraulic apertures (Snow, 1968), as plotted with five illustrative Lugeon curves in Fig. 7 (based on Eq. (2)), will need conversion to mean physical apertures (E) using estimates of the small-scale roughness JRC_0. The relevant equation and conversion curves are reproduced in Fig. 8.

Because of the small conducting apertures seen in Fig. 7, it is easy to imagine the *likely* difficulties of grouting rock masses of less than 1 Lugeon, because of mostly $e < 50$ μm with the given conductor spacings ($S = 0.5$–3 m), unless we can argue for $E > e$, and/or can increase E by using much higher pre-grouting pressures than that in the 1 MPa excess pressure Lugeon test. In addition, it may be necessary to use the much more expensive finely-ground cements like UFC (ultra-fine) or MFC (micro-fine). Examples of three different cement particle grading curves are given in Fig. 9, and the figure also shows the consequences of applying the rule-of-thumb and empirically verified equation (see Bhasin et al., 2002):

$$E \geq 4d_{95} \tag{3}$$

where d_{95} is almost as large as the largest cement particle (d_{100}) as sketched in Fig. 9.

The important aperture inequality $E > e$, from Barton (1982), still not recognised by many, is illustrated in Fig. 8 and conceptually in Fig. 10, specifically in relation to the choice of suitable grouts.

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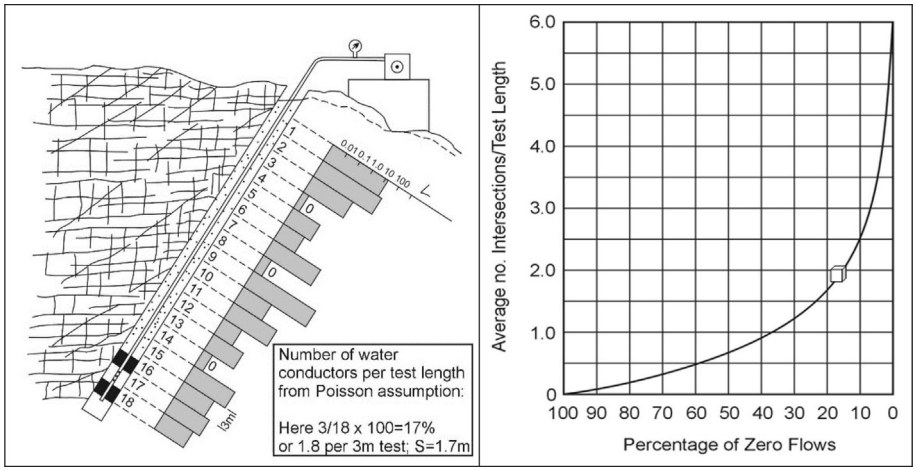

Fig. 5. Left: Lugeon testing with focus on zero flow stages as a percentage of the total. Right: Poisson distribution suggested by Snow (1968) for interpreting average number of water conductors. Here there are 17% zeros or 1.8 conductors/test length, if packer spacing = 3 m, and S = 1.7 m. With wider packer spacing, the percentage of zero flow stages reduces, and the average number of conductors ('intersections') per test length obviously increases.

The tabulated *E*-*e*-JRC conversion in Fig. 11 has coloured areas across the spectrum of the three *groutable aperture limits of* 40 μm, 90 μm and 350 μm from Fig. 9 (blue, green, lemon, orange, red, and purple). If we round upwards to 50 μm, 100 μm and 400 μm, these

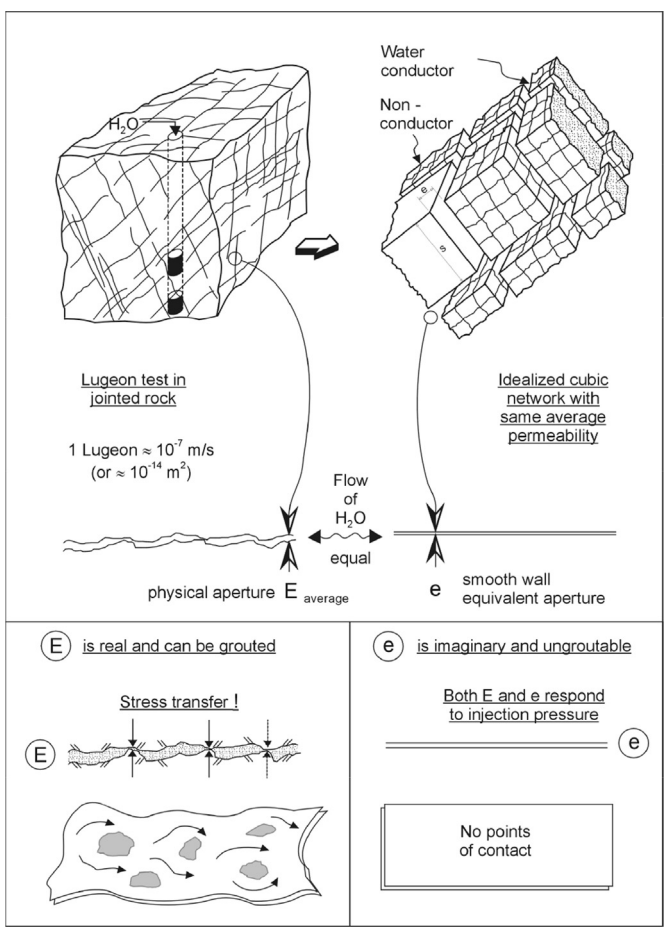

Fig. 6. Representing a regularly-jointed rock mass with a cubic network of hydraulic conductors of mean aperture (e) and mean spacing (S), based on Snow (1968). Note the distinction between apertures E (groutable mean physical aperture) and e (ungroutable theoretical hydraulic aperture) (Barton, 2002a).

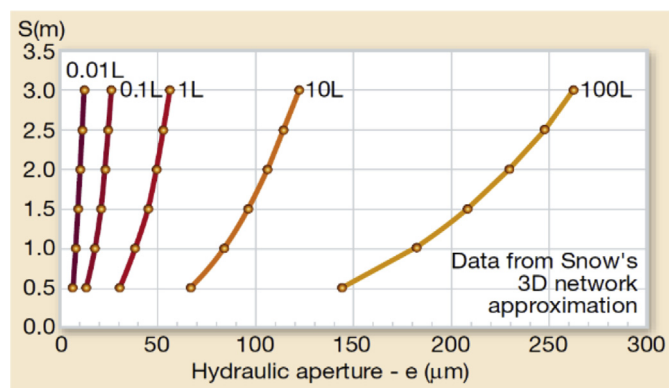

Fig. 7. Five curves showing Lugeon trends in relation to e verses S, from Eq. (2) (Barton and Quadros, 2003).

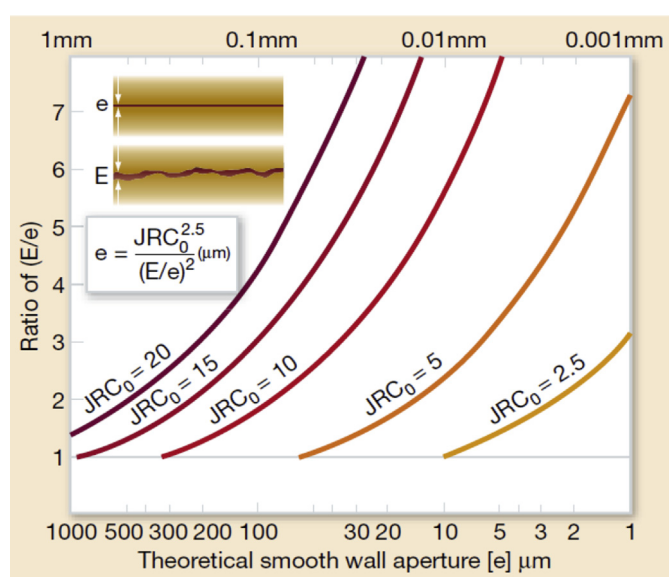

Fig. 8. The ratio of the average physical aperture (E) and the smooth-walled theoretical hydraulic aperture (e) can be approximated by the curves showing different JRC values (nominal JRC_0 representing 100 mm sample).

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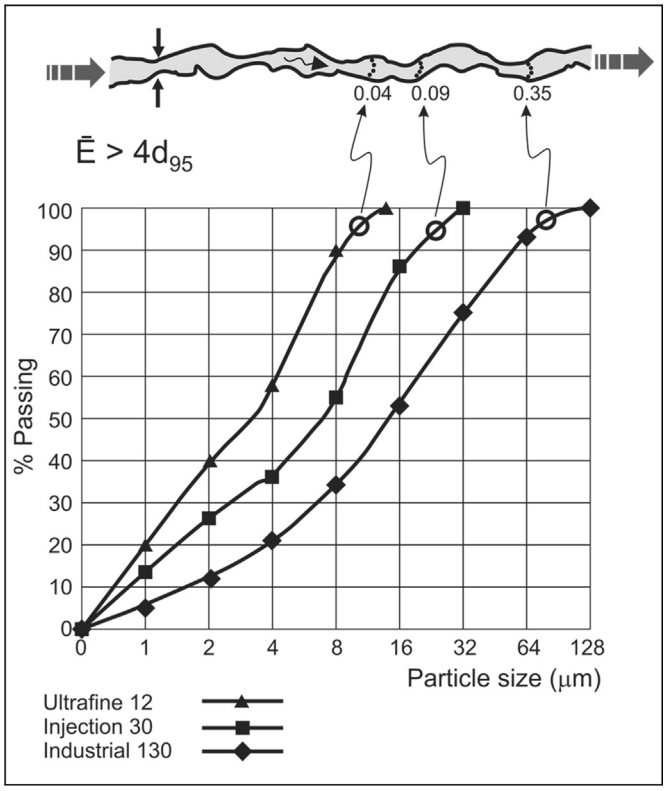

Fig. 9. Swedish Cement AB particle-grading curves for ultrafine, micro-cement (Injektering 30) and industrial cement. Note application of $4d_{95}$ (measured) rule-of-thumb for injecting the three grouts into joint apertures of approximately 40 μm, 90 μm and 350 μm, without recourse to high pressures.

more easily remembered apertures can be associated in general with ultra-fine, micro-, and industrial-cements (UFC, MFC, OPC) and have been adopted widely, at least in our local Scandinavian countries. The MFC is clearly most useful over a wide range of E (45–450 μm). However, the choice of high pre-injection pressures may allow the use of cheaper OPC, and this was indeed used with success at one of the rail tunnel projects where hydraulic and physical apertures were interpreted to be generally larger, based on the Lugeon-tests and JRC.

A remarkable published opinion is that the physical joint aperture needs to be *ten times* the largest grout particles, rather than the three

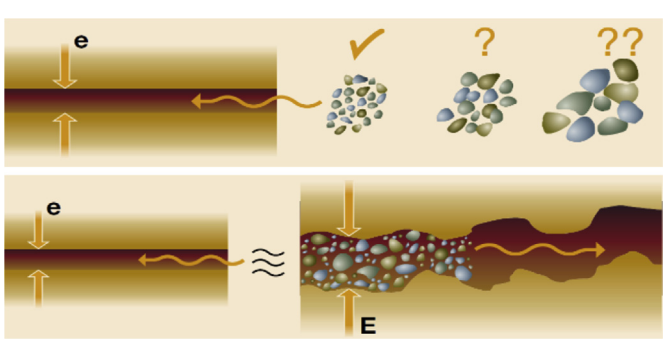

Fig. 10. The joint aperture inequality $E \geq e$ was graphed by Barton (1972) based on thesis work by Danielsen (1971), and Sharp (1970) and later related to JRC by Barton (1982), based on in situ testing and further collection of data. The aperture inequality has been confirmed by numerous authors' water-flow experiments since those times. It is important because it allows grout particles to penetrate real joints (mean apertures: E) even when the theoretical hydraulic apertures (e) are apparently too small, as illustrated in the top diagram. The grouts illustrated have imaginary particle sizes, but grading curves given in Fig. 9 suggest approximately 10 μm, 20 μm and 80 μm for the various d_{95} sizes.

to four times found from realistic experiments. The improbable physics needed to explain the stability of towers (actually high walls) *nine times d_{95} grout particles in height* (the tenth allowing flow?), should have alerted the Swedish author to the problems caused by the persistent use of the filter pump for qualifying (usually disqualifying) the desirable, stable, cohesive grouts. This important problem, caused by the high extension-viscosity of the desirable micro-cements, therefore resisting simultaneous separations of flow through an artificial (filter pump) screen, will be discussed later.

If each of the three grouts was available, and the contractor was allowed to make use of them, then in theory, if the characterisation was systematic, a solution could be found for each of the coloured apertures, with minimal injection pressures. Of course this is impractical, and for longer sections of tunnel geology, a single grout is used, with variable but preferably high pressure, which is achieved by successively reducing the $w/(c+f)$ ratio from e.g. 1 down to 0.7 and 0.5 at the end, where $(c+f)$ represents the cement and filler.

At the Norwegian Rail project's 2.7 km long Jong-Asker Tanum Tunnel, analysis of the systematic permeability measurements, using the methods described earlier, indicated quite tight hydraulic apertures (e) of 25–45 μm. Most (mean) physical apertures (E) were estimated to be 45–150 μm. The in situ (and theo-empirical) rock mass porosities ($n = 3E/S$: see Fig. 12) varied from 0.004 (shales) to 0.12 (nodular-limestones) (Barton, 2002a).

Note that the purely theoretical grout-take estimates for a permeable dam site, shown in Fig. 12, are 'moderate' because no injection-induced deformation has been assumed, just the idealised groutable porosity $= 3E/S$. As referred earlier, high-pressure pre-grouting usually supplies from 1 L to 6 L of grout to each 1 m^3 of rock mass, based on an assumed (mean) 6 m thick grouted tunnel annulus, since bolt holes of 4 m and 5 m length seldom leak when pre-grouting is correctly designed and performed (high pressure, micro-cements, micro-silica and dispersants).

6. Water-sick rock syndrome

Some years ago, a tunnelling consultant unintentionally caused a year to be added to the completion date of an important sub-city tunnelling project, with hundreds of millions in (local currency) cost over-run. He refused to allow a more experienced contractor to use micro-cement and micro-silica, and sufficiently high pre-injection pressures. Analyses of the Lugeon tests, as described in this paper, confirmed the need, both for stable micro-cement and for high pressures (5–10 MPa), to achieve the necessary tightness, in place of only 2.5 MPa as allowed by the consultant.

Inflow to the tunnel needed to be restricted permanently due to the numerous overlying buildings founded on clay deposits. The adverse result was 'water-sick' rock (Fig. 13) with more water in the rock mass than before pre-grouting, due to separation of the cement from the water in a 'coffee-filter' action, since the permitted grout was unstable. Furthermore, all bolt holes were leaking and had to be post-grouted. The appearance of the wet shotcrete, which should have been dry as in the next photograph (Fig. 14), was evidence of the consultant's failed pre-grouting 'design'.

Testing of a locally available micro-cement had indicated that it would have inferior penetration properties compared to the consultant's choice of cheaper industrial cement, which 'logically' (in the consultant's eyes) was therefore recommended to the unsuspecting and so far satisfied owner. To know what went wrong in this project gives a valuable insight into the necessary comparison of joint apertures and particle sizes, and appropriate test methods.

In earlier studies connected with initial site characterisation for a nuclear power plant to be sited underground (Barton, 1972), application of the Snow (1968) method prior to the above JRC correction had shown mean conducting apertures (e) of

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JRC_{mean}	3.5	7.5	12.5	17.5
	$J_r = 1$	$J_r = 1.5$	$J_r = 2$	$J_r = 3$
e (μm)	E (μm)	E (μm)	E (μm)	E (μm)
45	45	80	160	240
75	75	110	200	310
90	90	120	220	340
160	160	160	300	450

Fig. 11. During core logging, or when tunnel logging, it is convenient to have a profiling gauge and straight-edge for estimating both J_r and JRC_0. A table showing examples of the conversion from e to E, and consequences for choice of grouts (or perhaps the choice of elevated pressures), is shown on the right. The table shows approximate conversion of hydraulic apertures (e) to mean physical apertures (E) at a rail project. Note use of JRC and roughly equivalent J_r values (from rock mass quality Q-logging). The estimates of hydraulic apertures (e) will have been derived from Fig. 7 and Eq. (2).

approximately 90 μm, 70 μm and 60 μm in approximate depth zones of 1–20 m, 20–40 m and 40–120 m, respectively, based on flow tests down a 45° inclined borehole. The estimated mean spacing of water conducting joints (S) in the banded gneiss (Brenntangen, Oslofjord) were respectively 1.5 m, 2.6 m and 3.1 m. Significantly, from 5% to 20% of the 5 m long permeability test sections had shown zero flow, despite the fact that the core had 15-30 joint intersections in the same 5 m test length.

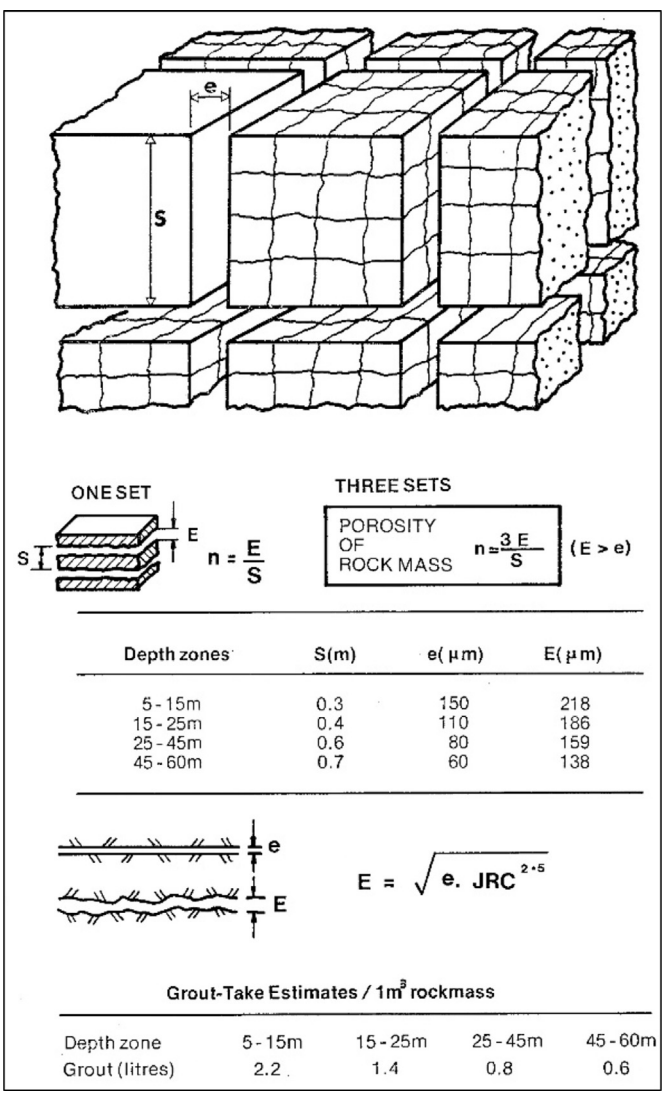

Fig. 12. A relatively early (1978) application of the Snow (1968) 'cubic network' method, with introduction of the JRC method of distinguishing between e and E for a permeable dam site in Surinam. Note the assumed physical joint porosity = $3E/S$. Note also the reducing permeability and reducing aperture estimates at depth. This logic extends to the estimates of the spacing of conducting joints (S), in other words more 'zero flow' stages at increasing depth (Barton et al., 1985).

Boundary layer theory (slow boundary flow, faster central flow), in fact a theoretical parabolic velocity distribution, probably applies for flow of grout in rough-walled rock joints. It also applies for flow of ore in ore-schutes in mines. The limiting (theoretical) result is three maximum-size d_{max} particles side-by-side, the two outer ones stationary, the central one just able to squeeze by, or a more realistic $4d_{95}$ as a practical rule-of-thumb, since d_{max} should be a rarity, by definition.

This relationship has been carefully tested in controlled laboratory experiments, using rock joints 'closed' by normal-stress. Measured hydraulic apertures (e) and JRC-converted mean physical apertures (E) were compared with $4d_{98}$ (see Bhasin et al., 2002). It was found possible, for instance, to inject a rock joint under a normal stress of 3.5 MPa, with a water-flow interpreted hydraulic aperture (e) of 25 μm, using a stable super-fine grout. This had a re-checked d_{98} particle size of 12 μm. With $JRC = 6$, the $e = 25$ μm aperture converts to an (E) aperture estimate of 47 μm. In this case, we have $E \approx 4d_{98}$.

Some Swedish opinion, on the other hand, due to an unfavourably chosen test method for grout characterisation (the unrealistic filter pump), suggests that from *8 to 10 times the d_{95} particle-size* is the necessary joint aperture for UFC and MFC grout penetration. These unexpected opinions from several PhD students, which were 'confirmed' by their professor in a subsequent text book and numerous lectures, need further analysis. The opinions (basically 'disqualifying' stable micro-cements due to filter pump experiences) have been used for many decades and have made pre-grouting opinion and practice in Norway and Sweden widely divergent when permeabilities are low. Nevertheless, settlement damage still needs to be minimised by appropriately understood pre-grouting.

The contrasting potential behaviours with varied $w/(c+f)$ (water/cement plus filler ratios), sketched in Fig. 15 (see #1 and #2) are not the main problem with the filter pump. Extensive North Sea well technology experience and tests by Statoil have shown that the ideal ultra-fine (UFC) and micro-cements with micro-silica and dispersing agents have significant *extensional viscosity*, which limits penetration through artificial screens (and sand). Roald and Saasen (2004) described extensive tests on cements and additives from more than twenty manufacturers. They identified the high *extensional viscosity* as the reason that the filter pump gave false information for choosing optimal grout mixes for injecting rock joints. Missing this adverse property of the filter pump has been expensive.

Since the Swedish and Norwegian opinions are so divided concerning joint aperture limits in relation to grout particle size, a

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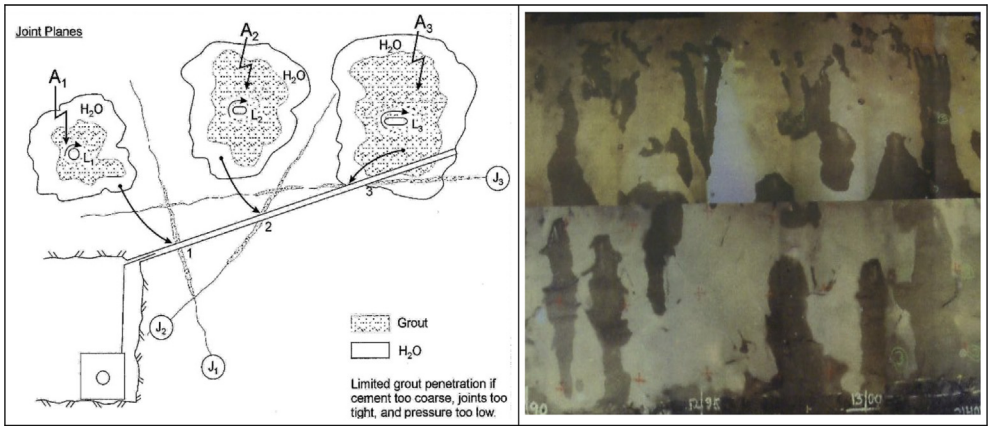

Fig. 13. If a grout is unstable and separates (bleeds) as shown schematically in the diagram on the left, and in addition if too low pressure is used, then 'water-sick' rock may result. When subsequently blasted, the rock mass is extra wet, and the shotcrete and bolt holes will leak, as shown here on the right from an actual project.

Fig. 14. About 80 m length of a pre-injected tunnel of 110 m^2 cross-sectional area, in interbedded shales and limestones, is shown. There is negligible leakage. Presumably a local post-injection is being performed, of a 'point' leakage, like the one seen on the right wall. From Bærum Tunnel, Norway.

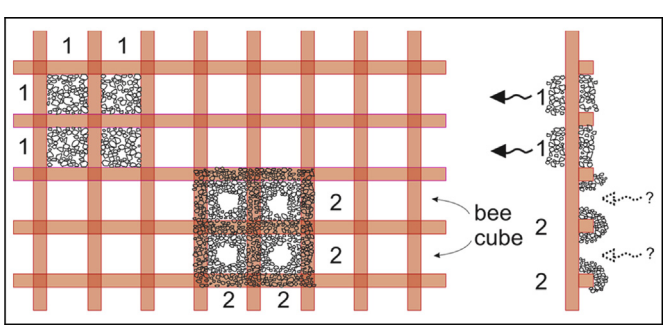

Fig. 15. The field test version of the filter pump has screens of different mesh/sieve sizes, which are fixed in turn across one end of what resembles a large cycle-pump. This rough sketch deliberately poses the problem of potential 'bee-cube' blockage by particles on the fine screens, when using the desirable lower $w/(c+f)$ ratios represented by #2 instead of a higher w/c ratio easy-flowing thin grout, as represented by #1. Characterisation with the filter pump has been one of the Swedish criteria used to classify grouts for many decades, and results in incorrect assumptions about the 'supposed difficulty' with the desirable stable micro-cements used with great success elsewhere.

simplified view is necessary. Firstly, Fig. 16 suggests the obvious: if the entry aperture is too small, use ultra-fine or micro-cements. But by no stretch of the imagination can the right-of-centre diagram, which shows a 'height' of 9 or 10 micro-particles, be considered the necessary degree of fineness to enter a pressure-widened joint. Seen at large-scale as in Fig. 16 (right), the idea of two nearly stationary ore-schute boulders on each wall, and a central boulder trying to pass between them, is easier to visualise and a more correct representation of 'blockage'. Single boulder-width (particle diameter) flow, one at a time, needs rejecting, because variations in aperture and 'block' size would soon cause blockage. However, a 'column' of more than 8 to 10 stones (d_{95} cement particles) in line, which would represent the surprising Swedish opinion, is obviously hard to justify.

A traffic flow analogy is also helpful here. In Fig. 17, the slowest movers in the traffic will be those driving close to eventual parked vehicles, equivalent to the rough joint walls with stationary grout particles. The dense traffic might also have a (hypothetical) parabolic velocity distribution due to the 'boundary layer' effect, just as expected in fluid/grout/ore-pass flow. The filter pump, as used in Sweden to characterise (and often disqualify) stable and desirable micro-cements, can be likened to an attempt to pass this extremely dense city traffic (or cohesive grout with its marked *extensional viscosity*) through a *toll station* (its equivalent: the filter pump screen).

This process would be made extra difficult if all larger vehicles had to change shape (elongate) to get through the toll area: a traffic-equivalent representation of the extensional viscosity resistance to 'squeezing through a rectangular hole'. Stable and desirable micro-cements-with-additives tend to get disqualified in the filter pump, a situation caused by 'bee-cubing' or flocculation as in the sketch of Fig. 15 (case #2).

The more relevant (traffic-analogy) test is not the unfortunately low (or zero) velocity through the toll station (the 'filter pump screen') but whether the traffic can flow in the next kilometres along equally crowded streets (with parked vehicles, i.e. roughness). In other words, whether the desirable cohesive stable grout can flow some few metres inside an equivalent, narrow rough-walled rock joint (and into neighbouring intersecting sets) before stopping where 'the roads' become too narrow (for the heavy traffic). A stiffening grout-front, maybe involving several injected joint sets, is the ultimate neighbourhood traffic-jam in our analogy.

7. Joint deformation due to injection pressure

Extensive normal closure testing of rock joints, reported in Bandis et al. (1983), demonstrated a strongly concave stress-versus-closure behaviour, with marked hysteresis on the first cycle of

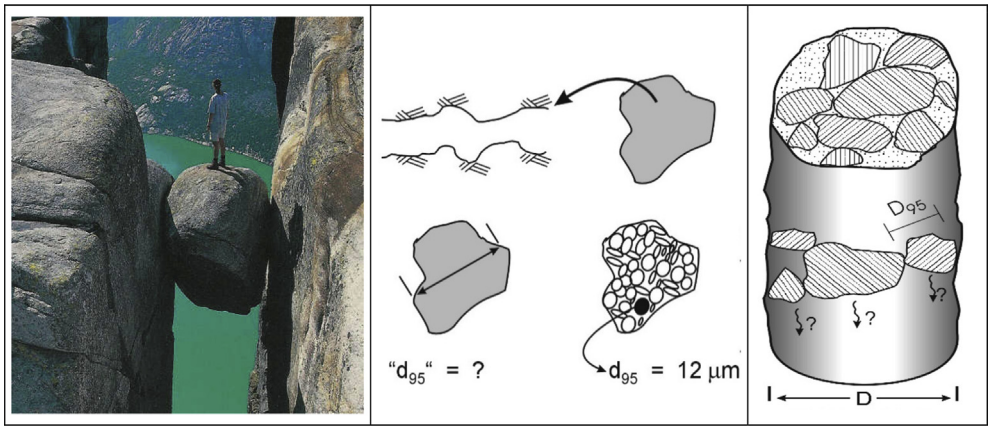

Fig. 16. One of the two options for grout entry is to reduce particle size, i.e. use of UFC. At much larger scale, even larger scale than ore-schutes in mining, the 2 m boulder on the left clearly needs to be smaller, or the crack wider (or something of each), for the man to regret his risky 600 m vertical-drop photo-opportunity. The ore-schute experience has shown that $D \geq 4d_{95}$ is needed to avoid 'hang-up' or blockage, due to the slow blocks next to the wall. This resembles the parabolic velocity distribution assumption, with maximum velocity in the central region of a joint (Personal communication with J. Hadjigeorgiou).

loading (following the inevitable sampling disturbance). For this reason, the samples are cycled (loaded up and down), in order to get beyond the disturbed (hysteretic) behaviour. For the same reason (sample disturbance caused by unloading), the fourth almost repeating load-unload cycle is used in specific distinct element modelling (UDEC-BB). We use this laboratory load-unload experience in Fig. 18.

Fig. 18 demonstrates, in principle, what happens to the joint aperture, when performing a Lugeon test, and when pre-injecting grout at much higher pressure, say 10 MPa. Due to the deliberate high-pressure injection, joint aperture changes, i.e. $\Delta E > \Delta e$. In fact, by injecting at a pressure of say 10 MPa, the effective normal stress could *locally* be reduced by 10 MPa, so that the joint opens by some small fraction of a millimetre. In practice, this may not be achieved, because of the rapid pressure loss away from the injection hole, and pressure losses in the tubing and packer, while grout is still flowing (see three curves in Fig. 1).

In Fig. 19, 'cloud' symbols are used to represent grout filling of the available joint aperture, following the fourth ('fully consolidated') load cycle. The initial unstressed aperture in this simulation is 200 μm. It may be noted from the simulation that a theoretical reduction of effective stress by a 'maximum' 10 MPa injection pressure would only be predicted to open the joint by about 30 μm (increasing from 30 μm to 60 μm). It is logical to conclude that a negative effective normal stress, meaning *(local) hydraulic jacking*, would probably be desirable for grout penetration, if for instance $E_4 + \Delta E$ (Fig. 18) still created insufficient mean physical aperture for the planned type of grout and its specific $4d_{95}$ characteristic.

Although at many hundreds of metres depth in the Åspö underground research laboratory, an inconclusive result of attempted pre-grouting could be seen when performing careful *Q*-logging in Sweden's SKB pillar-experiment (APSE) drift (Barton, 2003b). The left photo of the roof area (Fig. 20) gives the distinct impression that only one joint set was injected. There can be several reasons accounting for this, including less well-connected secondary joints, or insufficient injection pressure, or not sufficiently fine grout. The grey-coloured grout can be seen next to white areas of calcite. The result showed leakage continued with some pooling of water. Presumably pre-injection pressures even higher than used might have solved this minor inflow inconvenience.

8. The importance of grout additives

A national research project in Norway was conducted in 2001–2004, following a disastrous delay on a prestigious 15 km long

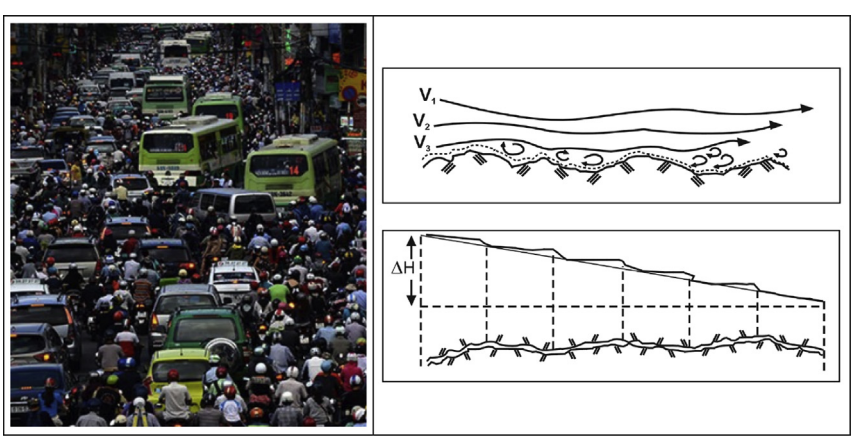

Fig. 17. Left: A (Vietnam) traffic analogy to grout flow. Eventual parked cars and parked buses would represent boundary layer 'roughness' along the route, and would hinder uniformly smooth (parallel) flow. Too many large units (buses, lorries or d_{95} grout particles), each trying to flow in parallel, would also hinder, or even prevent flow. Right: The flow-velocity and head-loss concepts illustrated by Quadros (1982) and by Louis (1969) are part of the same analogy for grout flow.

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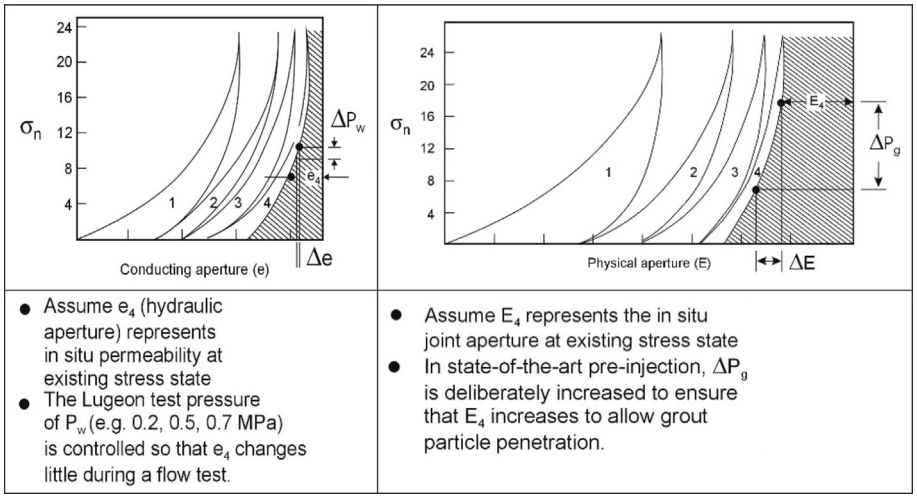

Fig. 18. Left: A hypothetical stress-aperture (e) fourth unload cycle performed on a rock joint, showing the small Δe caused by a Lugeon test (maximum $\Delta P = 1$ MPa). Right: A hypothetical stress-aperture (E) fourth unloading cycle showing the larger ΔE caused (deliberately) by a high-pressure pre-injection treatment.

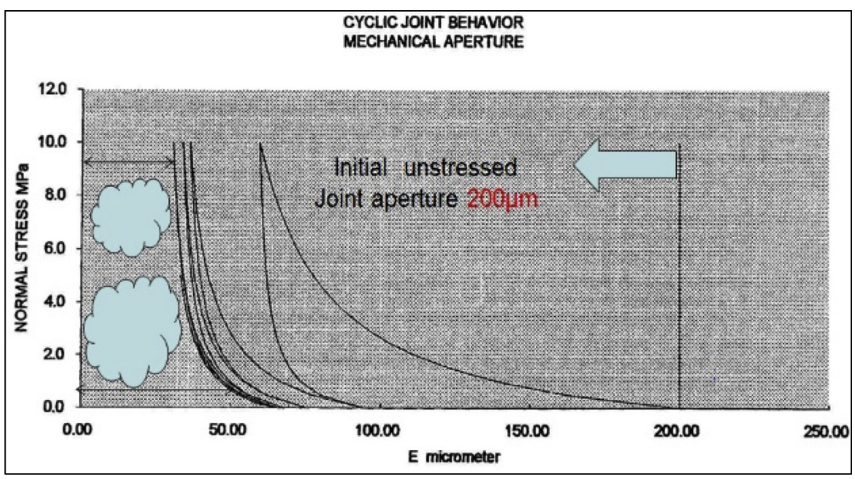

Fig. 19. Application of the Bandis et al. (1983) normal closure part of the Barton-Bandis (BB) model, with relevant input for small-scale joint roughness JRC_0 and wall strength JCS_0. Blue 'clouds' represent the aperture (E) available for grouting.

Fig. 20. Pre-grouting at depth with probable insufficient pressure resulting in grout in only one joint direction (left) with remaining wet conditions seen at tunnel-scale in the right-side photograph (Barton, 2003a).

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tunnel project to the new airport, prior to the Winter Olympics of 1994. The Romeriksporten Tunnel was due to be finished, and the rail line in operation, in time for the Olympics. The contractor was given insufficient time to carry out thorough pre-injection. A vital year was subsequently lost with costly and difficult post-injection. No investigation boreholes, a thin engineering geological report (inevitably, and obviously from the lowest bidder), and a tunnel driven under a rift-valley and under two lakes (all the above questioned by many to no avail), saw the smallest lake drained into the tunnel, all the trees round the lake collapsing inwards. Dozens of houses suffered settlement damage, as much as 500 m away from the line of the tunnel. It was time for a re-think, and stable, non-shrinking micro-cements, injected at high pressure, was one of the gradually developed responses.

The traditional use of cheap (OPC – ordinary Portland cement) cements for grouting, using limited additives (perhaps just less desirable bentonite) to try to minimise shrinkage, with the familiar need for second and third rounds of injection, is fortunately a thing of the past. Problems encountered in the past could include pressure-filtering in which water is squeezed out, leaving a filter-cake of larger particles preventing further inflow. Unstable cement suspensions, not sufficiently thixotropic, could even be thinned and washed out by the insufficiently controlled water. Bleeding and shrinkage during the hardening process, and gradual loss of chemical constituents and strength complete the list of inadequate performance, in relation to today's demands of dry tunnels and minimum surface damage or groundwater drawdown. In some locations in Norway, with overlying marine clays and housing, tunnel inflows as low as 1–2 L/min/100 m are hardly sufficient to prevent damage (Elkem/Roald, personal communication).

One of the major pre-injection breakthroughs came when the advantage of micro-silica was recognised, especially in the convenient form of slurry, and when combined with micro-cements. A significant advance has also come with the gradual recognition of the benefit of high pressure, which is still very slow to be accepted in many countries, judging by the frequent need for post-injection in tunnels and in some critical (cross-over rail) caverns known to the authors. This perhaps is because of fear of hydraulic fracturing, due in turn to the lack of understanding of the steep pressure gradient out from each pre-injection hole, especially for the case of low w/c grouts. These Bingham fluids with cohesion and friction cause even 'steeper' pressure loss than Newtonian low-friction, non-cohesive fluids like water, as discussed earlier.

A form of micro-silica called GroutAid, developed as a slurry by Elkem, consists of spherical micro-silica particles, more or less as fine as cigarette smoke. An image of the huge difference in size in relation to the cement particles is given in Fig. 21, and some experimental results concerning the control of bleeding and volume loss are illustrated in Fig. 22.

A range of available cement particle sizes was tabulated by Tolppanen and Syrjänen (2003) in their major review of Finnish, Swedish and Norwegian grouting practice (Table 1). The foregoing theoretical $E \geq 3d_{max}$ rule-of-thumb (and the more practical and experimentally confirmed $E \geq 4d_{95}$) indicates the suitability of these tabulated cements for physical apertures (E) over a theo-empirical range of approximately 24–384 μm.

As emphasised by Hansen et al. (2002), the rule-of-thumb concerning the maximum grain size and minimum fissure or joint size (which is more practical when expressed as $4d_{95}$), will only apply in practice if:

- The grout is stable (<2% bleed);
- Pressure filtration does not occur; and
- All fine particles are separated and fully dispersed (using superplasticiser and high-speed mixers).

As discussed earlier, extremely fine (UFC and MFC) stable grouts with state-of-the-art additives are 'rejected' when tested in the filter pump, due to the artificial (but temporary) separation of the grout into fine streams when/if it manages to be drawn (by semi-vacuum) through the screen. For instance, Dalmalm and Janson (2001) found that a '12 μm cement' (i.e. UFC) needed 8 times the particle size for penetration through a filter pump screen, while a '30 μm cement' needed only 3 times. This 'rejection' of the finest and most stable cement-and-admixtures may be relevant in the case of flow through sand, and these ideal rock-joint-injection-grouts may also be rejected in the sand-column test, whose name is self-explanatory. However, they are not rejected in the so-called NES test, which involves a more joint-like flow through a narrow slit of fixed aperture, such as 50 μm.

According to the experimental investigations performed by Khan et al. (2004), the extremely steep pressure decay experienced when trying to grout sand and porous media is due to the dominance of the *extension viscosity* in relation to the *shear viscosity*. According to Roald and Saasen (2004), the extension viscosity may be orders of magnitude higher than the shear viscosity, in the case of the stable grouts that are in practice, ideal for pre-grouting in rock masses with rock joints.

9. Theo-empirical estimation of grout-take

Earlier when referring to Fig. 18, we saw the theoretical effect of pre-grouting pressure on joint deformation, and the possible/probable need of some *hydraulic jacking* (close to the injection holes). The following ultra-simple theo-empirical example of rock mass deformation (as a result of joint openings) may be of interest here. It involves application of the simplest and most well-remembered equation in every civil engineer's university education: Hooke's law: $E = \sigma/\varepsilon$ *('modulus equals stress over strain')*. We will apply it to estimate the rock mass deformation that is likely between pressurised joints. Let us consider a relatively 'poor quality' rock mass with $Q = 1$, needing improvement by pre-injection. Fig. 23 indicates that the static *rock mass* deformation modulus (M) may approximate 10 GPa at nominal 25 m depth (see

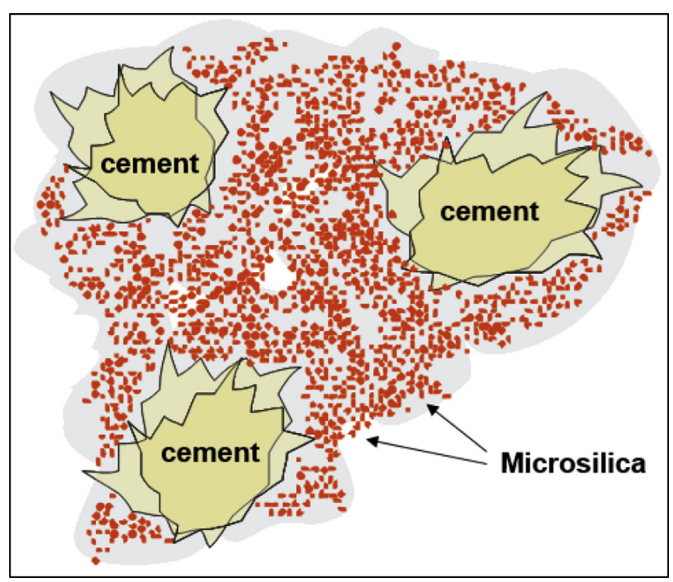

Fig. 21. The relative sizes of cement particles and micro-silica, which resembles cigarette smoke in its fineness. There may be 50,000 to 200,000 micro-silica particles per cement particle. A pozzolanic reaction results. Use of micro-silica (in slurry form) gives remarkable advantages in terms of reduced shrinkage. (Roald, personal communication).

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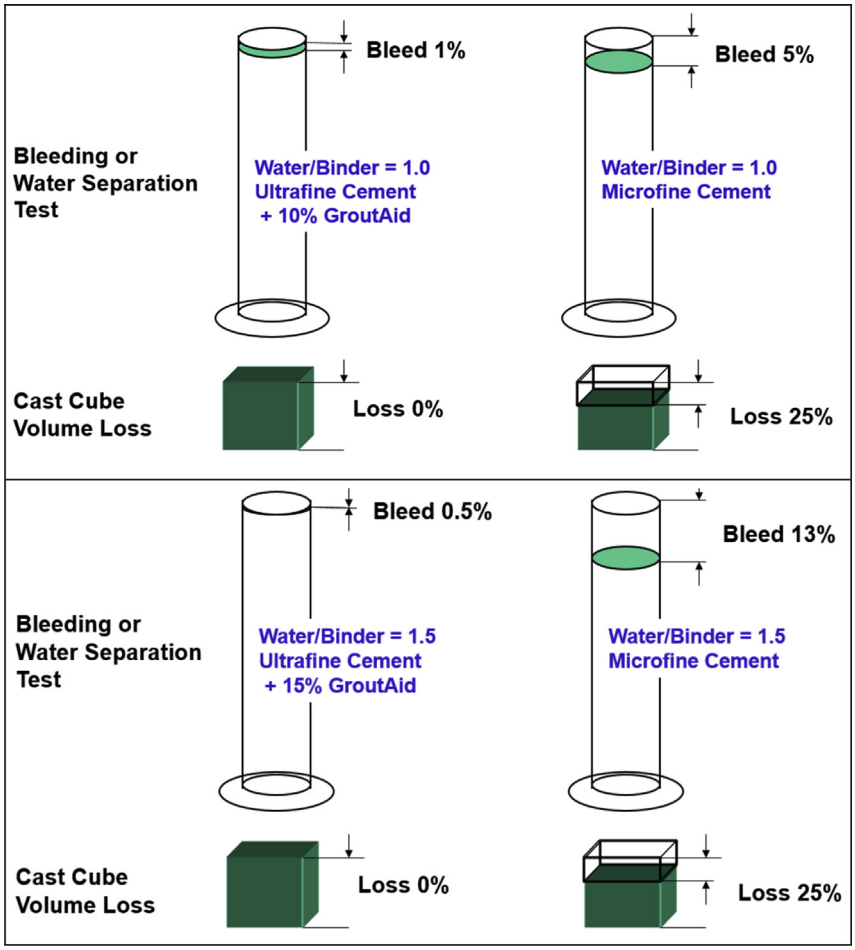

Fig. 22. The desirable and undesirable degrees of bleeding and volume loss, steered by UFC and MFC with or without micro-silica based GroutAid. The diagrams on the left side indicate the advantages of using GroutAid. Serious bleeding and volume-loss are seen (on the right) when not using micro-silica (S. Roald, personal communication).

central diagonal, and table on right side). What is needed next, is the estimate of the theoretical mean spacing of water-conducting joints (S) from Figs. 5–7. The imaginary joints are then 'injected' under pressure, and the deformation modulus (based on Q, and depth) is used to estimate what effect this is likely to have on the rock mass deformation between the joints, or more particularly, what the physical apertures of the imaginary joints might increase to, in each of three idealised perpendicular directions.

Due to the deliberate choice of simple numbers, the assumed very high 10 MPa local (close-to-borehole) fluid-pressure causes $\varepsilon = \sigma/E = 10/10{,}000$ (MPa/MPa) = 0.001, or 1 mm joint opening per 1000 mm of rock. A cubic block of 'jointed' rock of 1 m^3 has a (fluid-equivalent) volume of 1000 L. Three joints, opening 1 mm on each of three adjacent sides of this in situ block could absorb *an extra* 3 L of grout per m^3 of rock.

From compilations of practical tunnelling experiences, we can derive the following estimates of grout-take, as used in successful, high-pressure pre-injected tunnels (Åndal et al., 2001). Values in parentheses (Table 2) signify presumed 'escape' of grout in two cases, and local break-down of the '6 m grouted cylinder' assumption.

A low percentage of leaking bolt holes of 4–5 m in length is the logic behind an average choice of a 6 m cylinder of pre-grouted rock. We can see from Table 2 that 1–6 L of grout per cubic metre of rock mass is a typical range. The projects achieved post-grouting water leakages mostly in the desired (almost dry) range of 1–4 L/min/100 m of tunnel. Tunnel cross-sectional areas were mostly 65–95 m^2.

10. Relating permeability to Q and to Q_{H2O}

Estimation of permeability from rock mass *characterisation*, as opposed to its estimation from permeability testing in boreholes at specific depths, is never easy, and may indeed be inadvisable, since there are potential problems (actually for both methods), such as flow-channels within the joint planes that

Table 1
Cements available (list from 2002) give evidence of numerous efforts to produce, at considerable extra expense, ever finer cements for challenging pre-injection and grouting projects. Table assembled by Tolppanen and Syrjänen (2003).

Cement type	Grain sizes achieved by manufacturers (μm)
Cementa Anläggningscement	$120d_{95}$ and $128d_{100}$
Cementa Injekteringscement 64	$64d_{95}$ and $128d_{100}$
Cementa Injekteringscement 30	$30d_{95}$ and $32d_{100}$
MBT Rheocem 650	$16d_{95}$ and $20d_{98}$
Cementa Ultrafin cement 16	$16d_{95}$ and $32d_{100}$
Orginy Spinor A16	$16d_{98}$
Dykerhoff Mikrodur P–F	$16d_{95}$
MBT Rheocem 800	$13d_{95}$ and $20d_{100}$
Cementa Ultrafin cement 12	$12d_{95}$ and $16d_{100}$
MBT Rheocem 900	$8d_{95}$ and $10d_{98}$
Orginy Spinor A12	$12d_{98}$
Dykerhoff Mikrodur P–U	$9.5d_{95}$
Dykerhoff Mikrodur P-X	$6d_{95}$

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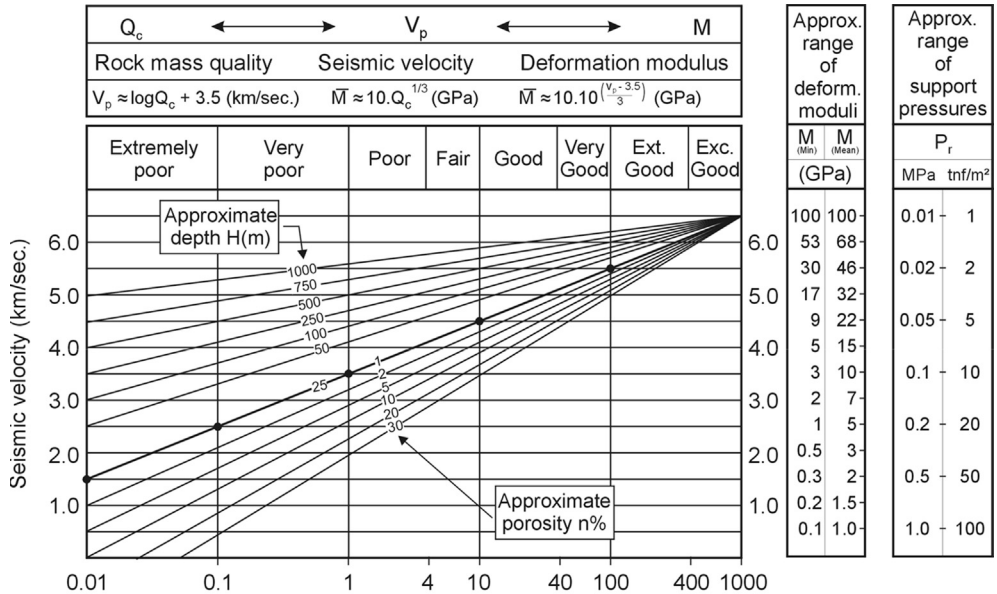

$$Q_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right]\frac{\sigma_c}{100}$$

Fig. 23. It is convenient to refer to this Q-V_P-$M_{modulus}$ diagram from Barton (1995), for making rough estimates of the potential effect of grouting pressure on rock mass deformation, and therefore potential joint opening (hydraulic jacking). This behaviour lies 'outside' (on the negative side) of the joint stress-closure behaviour of Figs. 18 and 19. (The equation below the figure is the Q-value normalised by values of uniaxial compression strength σ_c different from a nominal hard-rock 100 MPa. The symbol Q_c is used for this. According to convention, V_p is the symbol for the P-wave velocity).

have suffered erosion or solution-effects. There may also be joints that are 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 Fig. 24 may have some practical merit, when 'out in the field' in a tunnelling situation, and needing, for example, to assess possible pre-grouting needs. Table 3 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 can be identified. Where clay is present, and where there is strong anisotropy of structure and permeability, a greater level of sophistication is obviously needed than that shown in Table 3, or in this figure.

It is noted from extensive site characterisation work from Sweden (Hartley et al., 2006) that the permeabilities at the Laxemar site are generally in the range of 10^{-4}–10^{-9} m/s. However, this is based on 100 m test scales (in other words smoothed data, in relation to that obtained with close packer-spacing). This smoothed data show the mean depth-variation given in Table 4.

Table 2
Pre-grouting of tunnels (kg or litre) data derived from Åndal et al. (2001). 1–6 L/m^3 seems to be the most typical result, if no escapes of the grout.

Rock type	Pre-grouting quantities expressed in three different ways		
	Tunnel surface (kg/m^2)	Rock mass (kg/m^3)[a]	Rock mass (L/m^3)[a]
Gneiss	11–16.5	1.8–2.8	1.2–1.9
Granite	12–52	2–8.7	1.3–5.8
Phyllite	26	4.3	2.9
Rhomb porphyry	28 (99)	4.7 (16.5)	3.1 (11)
Syenite (dike)	30 (186)	5 (31)	3.3 (20.7)
Fracture zone	19–50	3–8.3	2–5.5

[a] An average cylindrical annulus thickness of 6 m of grouted rock mass has been assumed. A grout density of 1.5 g/cm^3 is also assumed. This of course varies with the w/c ratios used during the grouting, and is only approximate. Assumed escapes (–).

Strictly considering the smoothed data, there is a certain indication of improved rock quality with depth, which of course is supported in general by Q-logging of cores from similar deep boreholes in a nuclear waste site in Sweden (Barton, 2003b). Since stress levels are also increasing with rock quality, it is likely that other rock mechanics characterisation data will also vary with depth, and indeed with direction.

Table 5 shows the Lugeon-Q value approximation $L \approx 1/Q_c$ discussed in detail in Barton (2006) 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 J_r/J_a to the form J_a/J_r, whereby clay filling will result in an increase of Q_{H2O} (and a reduction of the permeability estimate), while an increase in roughness or discontinuous jointing will give an opposite effect. Fig. 25 shows the depth-dependence that is presently built into the permeability estimate. As in the case of the Q_c-depth curves in general, one will usually experience curve-jumping as quality improves at depth. Q_{H2O} will tend to increase with depth, like Q_c.

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'

$$Q_{H2O} = \frac{50}{9} \times \frac{4}{1.5} \times \frac{0.66}{1} \times \frac{100}{10} = 98$$

Substituting in the empirically derived permeability versus depth equation yields

$$k = \frac{2}{1000 \times 98 \times 100^{\frac{5}{3}}} = 9 \times 10^{-9}\ \mathrm{m/s}$$

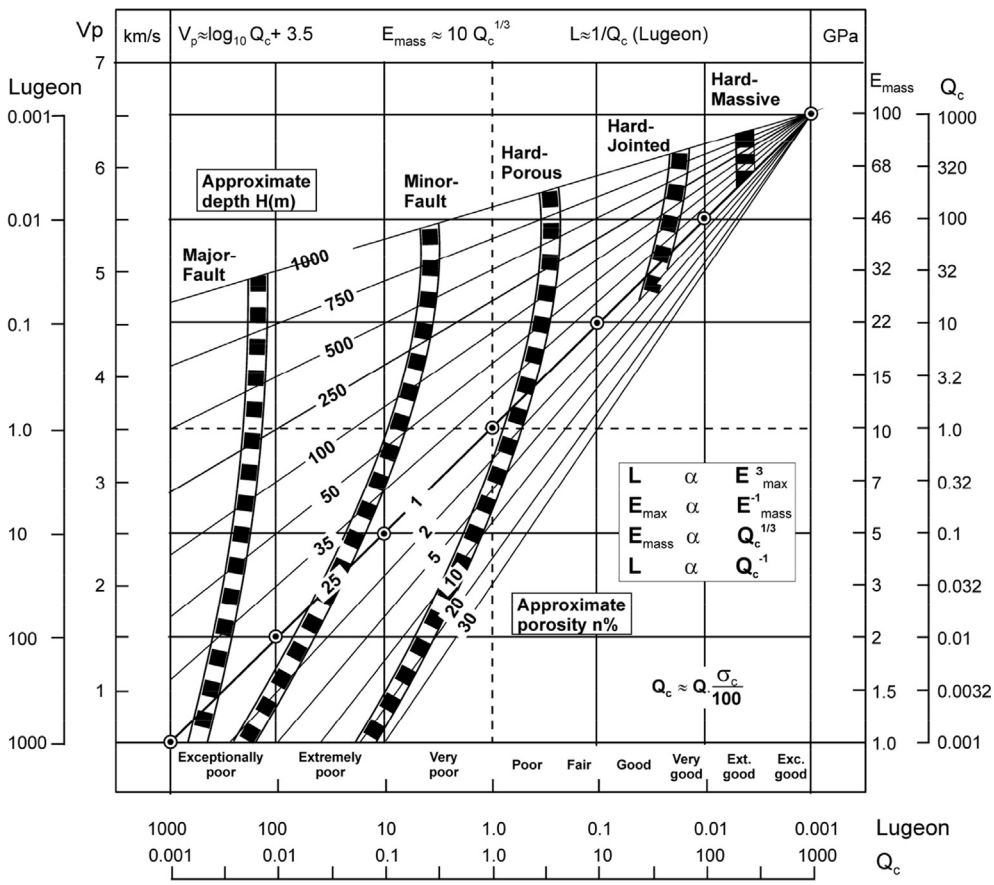

Fig. 24. Type-curves suggested by Barton (2006). The 'proportional' (α) terms in the inset are based on double-Boussinesq load theory, flow theory, and empirical relationships to Q_c. For a more general solution for permeability, the modified term Q_{H2O} is recommended. This is shown in Fig. 25. Note the 'type-curves' in the above figure for e.g. 'massive rock': high rock mass quality Q_c, high rock mass deformation modulus E_{mass}, high P-wave velocity V_P, low Lugeon L-value. Such expectations are logical, but clay-filled discontinuities cause problems, and in that case Q_{H2O} is recommended (Barton, 2007).

Table 3
A set of inter-related geotechnical approximations that are useful when assessing results in the field. Note: $Q_c = Q\sigma_c/100$ as shown in Fig. 23 (Barton, 2002b, 2006).

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

Table 4
Smoothed permeability data from the Laxemar site in Sweden (Hartley et al., 2006). Extreme values, even at 600–800 m depth, sometimes range from at least 10^{-6} m/s to 10^{-11} m/s. The rapid changes in permeability at shallow depth resemble the 'Q-jumping' seen with P-wave velocities. Velocity gradients (km/s/km or s^{-1}) can be as high as 100–200 in the near-surface, but decline rapidly at greater depth where rock mass quality Q-values (and permeability) show slower improvement.

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

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

Brantberger et al. (1998) have given Eq. (4) for estimating the inflow into a pre-injected tunnel. This was used by Bäckblom (2002) in a report for the Finnish nuclear waste authority POSIVA, with predictions of inflow in small tunnels at 450 m depth, which was of interest in POSIVA waste disposal plans at that time.

$$q = \frac{2\pi k_i h}{\ln\left(\frac{R+t}{R}\right) + \frac{k_i}{k}\ln\left(\frac{2h}{R+t}\right) + \xi} \tag{4}$$

where q is the inflow to tunnel per metre of tunnel (m^3/s) (Multiply by 1000 × 60 × 100 for L/min/100 m). k here is the hydraulic conductivity of the back-ground rock mass (m/s), k_i is the hydraulic conductivity of the pre-injected zone (m/s), R is the radius of tunnel (m), t is the assumed thickness of grouted zone 'cylinder', h is the depth, and ξ is the skin factor (assume a 'central value' of 5).

For an ungrouted tunnel, the inflow Q per metre of tunnel in an isotropic medium with a horizontal phreatic surface was actually derived (for an isotropic porous medium) by Muskat (1937), and was 'verified' in a large Hele-Shaw model reported in Barton (1972). The skin factor is included in the version (Eq. (5)) given by Brantberger et al. (1998).

$$q = \frac{2\pi kh}{\ln\left(\frac{2h}{R}\right) + \xi} \tag{5}$$

For instance, with a back-ground permeability (k) of 10^{-7} m/s, a 5 m thick injected zone ($t = 5$ m), and a permeability (k_i) of 10^{-8} m/s (a

Table 5
The two versions of 'Q-permeability' estimation (Barton, 2007). Both methods are obviously approximate because both methods are presently based on limited test data. However, Q_{H2O} has been tested in more recent rail tunnel projects, including multiple TBM with extensive seismic refraction results to help infer Q-values between boreholes. It appears to be sensitive to the higher permeability of fracture zones, and of weathered rock, and also sensitive to the lower permeability with increased depth, where there may be increased Q-values. It was derived empirically from Q-logged and Lugeon-tested boreholes in shales, limestones, schists, and nodular limestones at rail-tunnel sites, and has been applied with apparent success in granites (Norway and South Africa), and in granitic gneisses in Norway.

Case	Equation
(i) $L \approx 1/Q_c$ (1 Lugeon $\approx 10^{-7}$ m/s $\approx 10^{-14}$ m^2 for water at 20 °C) (*hard*, *jointed*, *clay-free* rock masses)	$Q_c = (RQD/J_n)\,(J_r/J_a)\,(J_w/SRF)\,(\sigma_c/100)$ (standard equation, normalised by $\sigma_c/100$)
(ii) General case, with depth/stress allowance, and consideration of joint wall strength JCS	$Q_{H2O} = (RQD/J_n)\,(J_a/J_r)\,(J_w/SRF)\,(100/JCS)$ $k \approx 0.002/(Q_{H2O}D^{5/3})$ m/s

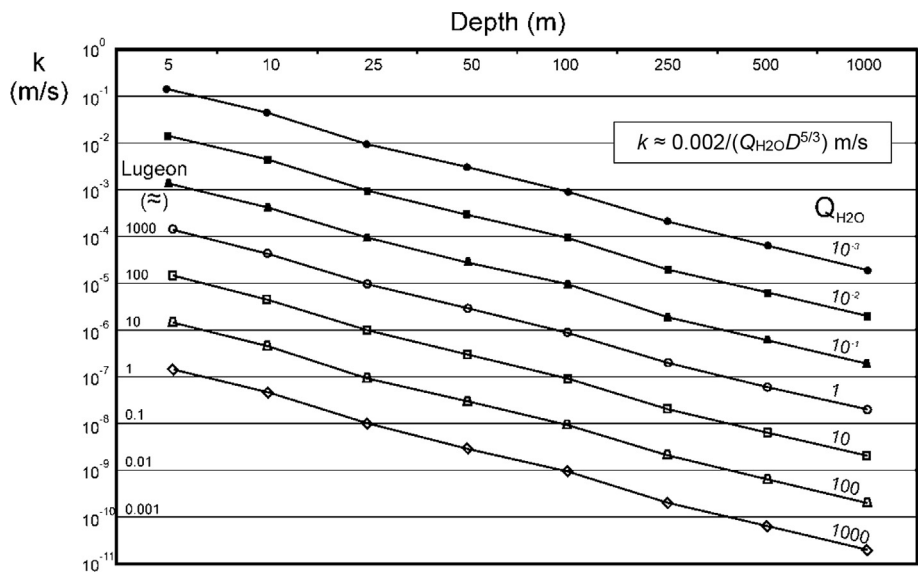

Fig. 25. Depth-dependent permeability estimation using Q_{H2O} concept (Table 5, Eq. (ii)) (Barton, 2007).

relatively poor result for pre-injection), or an alternative and better result of 10^{-9} m/s, also with $t = 5$ m, give predicted inflows of 25.8 L/min/100 m or the much improved 2.8 L/min/100 m, respectively. A groundwater level at surface (distance h) is assumed. In these estimates, Bäckblom (2002) used a skin factor ($\xi = 5$) (accounting for the effect of the rapid pressure drop at the periphery: a hydraulic excavation damaged zone (EDZ) effect is presumed). It is reported from Äspö URL research (Rhén et al., 1997) that this 'correction factor' ranged from 4 to 7. Table 6 shows an illustrative progression of grouting needs, for a wide range of back-ground permeabilities.

We will now repeat the application of Eq. (4) for transport (road or rail) tunnels much closer to the surface, and of larger cross-section, to investigate if the calculations still give numbers (L/min/100 m) that correspond with our expectations, based on more knowledge of such projects than smaller tunnels at 450 m depth, as investigated by Bäckblom (2002). Results for the larger tunnel are given in Table 7.

The sixteen inflow cases, sorted into depths of 50 m and 100 m, with no pre-grouting, or with pre-grouting, have been labelled (a, b, c, d) to represent the probable likelihood of failure to prevent groundwater drawdown. The two examples with estimates of 2.4 L/min/100 m and 3.2 L/min/100 m are in the 'grey area' according to experiences. As noted earlier, even 1–2 L/min/100 m is considered likely to cause settlement damage when conditions are unfavourable: meaning a great thickness of clay and a high initial water-table. We may note that if a pre-grouted zone with a permeability value as low as 10^{-10} m/s was achievable, inflow is reduced to virtually zero. However, reference to Fig. 7 shows a 'limit' of 10^{-9} m/s (approximately 0.01 Lugeon) because *hydraulic* apertures are below 5 μm, and supposedly off the scale of d_{95} values given in Table 1. Here, though *the mistake has been deliberately made* to compare a theoretical parameter (hydraulic aperture e) with a physical reality (d_{95}). With the $4d_{95}$ rule-of-thumb, we will be depending on aperture $E >> e$ due to high $JRC > 10$ (Refer to JRC conversion between e and E, Fig. 8). The concepts illustrated in Fig. 9 may then resolve the dilemma of whether a high-quality, high Q rock mass can be improved by pre-grouting. As with so many other things in hydraulics, it depends.

11. In situ 3D grouting effects

We will end this paper by addressing 3D anisotropy via 3D permeability measurements. Fig. 26 is a compact summary of some unique field tests from Brazil (Quadros and Correa Filho, 1995), which indicates that 3D testing using multiple boreholes can help to prove

Table 6
Quoting from Bäckblom (2002), in order to achieve an inflow of less than 3 L/min/100 m with $h = 450$ m, $R = 2.5$ m, $t = 5$ m and $\xi = 5$. The following sealing efficiencies would be needed, with the three orders of magnitude range of back-ground permeabilities (which might *roughly correspond* with Q-values of 100, 10, 1, 0.1) according to Table 3.

Hydraulic conductivity k of the rock mass (m/s)	Inflow Q to tunnel without pre-injection (L/min/100 m)	Required sealing efficiency (%)
10^{-9}	1.6	Sealing not needed
10^{-8}	16	80.7
10^{-7}	155	98.1
10^{-6}	1560	99.8

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Table 7
Application of Brantberger et al. (1998) equations (Eqs. (4) and (5)) to a larger transport tunnel at (fully saturated) depths of 50 m or 100 m. Assume $R = 6$ m, $t = 6$ m, $\xi = 5$.

Hydraulic conductivity k of rock mass (m/s)	Hydraulic conductivity k_i of pre-grouted cylinder (m/s)	Inflow without pre-grouting (L/min/100 m)		Inflow with pre-grouting (L/min/100 m)	
		$h = 50$ m	$h = 100$ m	$h = 50$ m	$h = 100$ m
10^{-9}	10^{-10}	0.24[c]	0.44[c]	0.03[d]	0.06[d]
10^{-8}	10^{-9}	2.41[c]	4.43[b]	0.32[c]	0.63[c]
10^{-7}	10^{-8}	24.13[a]	44.32[a]	3.19[c]	6.31[b]
10^{-6}	10^{-7}	241.3[a]	443.2[a]	31.92[a]	63.11[a]

[a] Unacceptably high inflow.
[b] Border-line cases.
[c] Acceptable.
[d] Exceptionally low inflow result.

what has been achieved in both successful or unsuccessful grouting. These particular before-and-after-grouting 3D water permeability tests were performed in a permeable dam abutment. The preliminary, conventional interpretation of *individual borehole tests* showed reductions of permeability from 1 to 4 orders of magnitude as a result of grouting (i.e. from 10^{-7} m/s to 10^{-8} m/s, or from 10^{-5} m/s to 10^{-7} m/s,

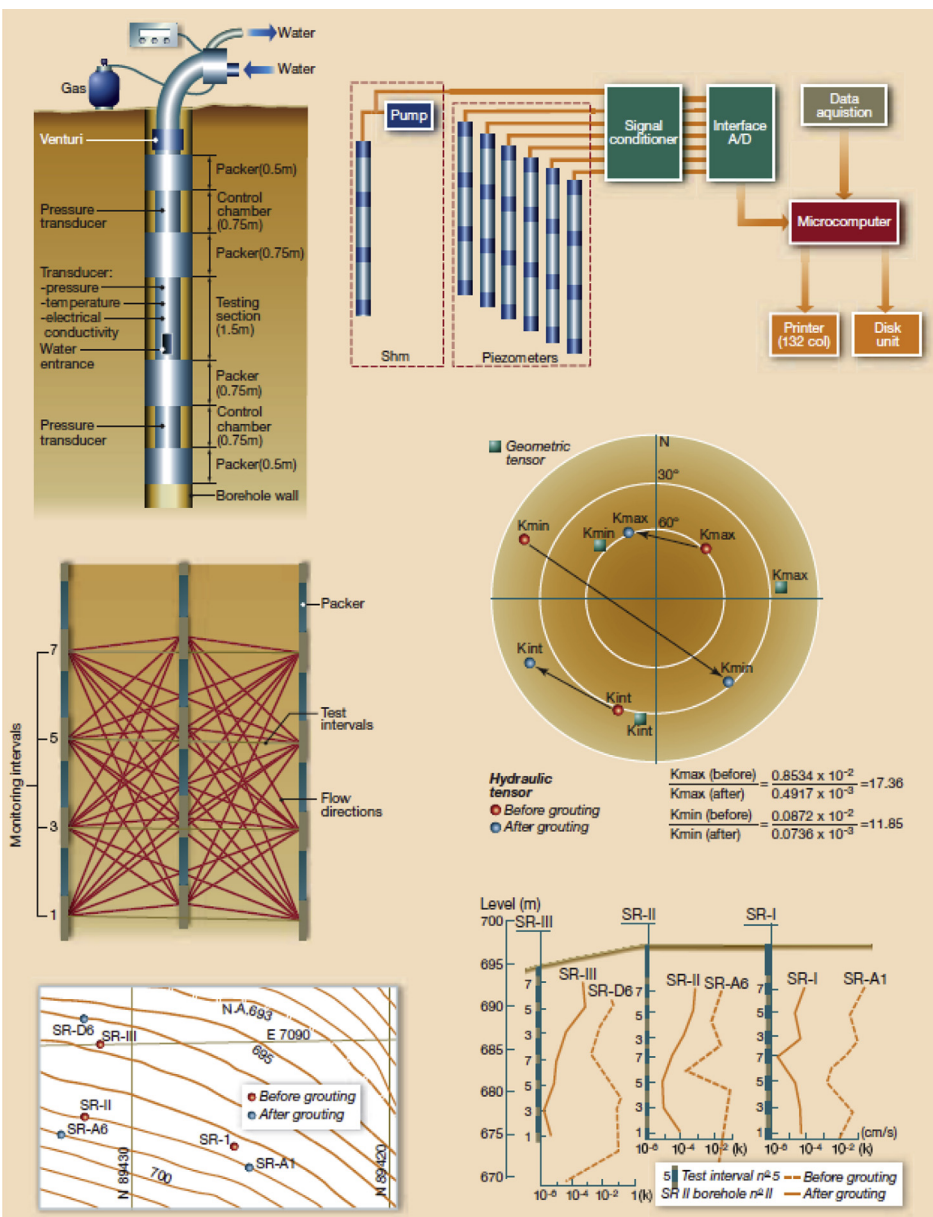

Fig. 26. Three-dimensional permeability testing performed between three boreholes, both before and after grouting, showed rotation and reduction in magnitude of the permeability tensors, and greatly increased bulk modulus. Despite use of industrial cement and bentonite, the permeable rock mass was greatly improved. Quadros and Cruz (1995), reproduced in Barton and Quadros (2003). Further examples of permeability anisotropy are given by Barton and Quadros (2015).

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or from 10^{-4} m/s to 10^{-8} m/s). Clearly, the higher the initial permeability, the greater the potential improvement.

In a 3D sense, the three principal permeability tensors all rotated as a result of the grouting, signifying good or partial sealing of at least three sets of joints. The reductions in k_{max} and k_{min} were more than one order of magnitude, despite the 6–8 m distance between the unusually widely separated boreholes. The bulk modulus increased on average by a factor of almost 8. This suggests that when pre-grouting ahead of a tunnel at much higher pressures, and with much closer hole spacing than here, and when using micro-cements and micro-silica based additives, rather than industrial cement and bentonite (as in the Brazilian tests), then dramatic changes in the *effective* rock mass properties can be expected. As will be seen shortly, even when using conservative assumptions concerning improvements in *Q*-parameters, some dramatic and rather likely improvements in rock mass parameters are indeed predicted.

The hydro-mechanical characteristics of the rock mass can be estimated using the measured values for the hydraulic directional diffusivity, k_d/S_s, where S_s is the storage coefficient of the rock mass and k_d is the directional hydraulic conductivity. For a homogeneous, isotropic medium, $S_s = \gamma g/E_v$, where E_v is the assumed average bulk modulus of the jointed rock mass, γ is the density, and g is the acceleration due to gravity (Laginha, 1968; Franciss, 1985). The calculated results show that for the undisturbed rock mass, the bulk modulus is $E_v = 169$ MPa. For the rock mass after grouting, we have $E_v = 1.35$ GPa, showing that the bulk deformation modulus increased approximately 8 times in the case of this poor-quality rock mass.

12. Estimating improved rock mass properties

The left column of Table 8 was a first demonstration of how the Q-system could be utilised to make (obviously) approximate estimates of the potential effects of grouting. Such possibilities were also discussed in Barton et al. (2002), with Norwegian grouting expert colleague Roald, at the same time that a new Q-system paper was being published, showing potential links of *Q* with other parameters (Barton, 2002b).

We see significant potential increases in *Q*-values, even when very conservative assumptions are made. In fact, it may be assumed that the left column of Table 8 is *too conservative* to be realistic: Larger effects than these must be expected from high-pressure pre-injection, assuming stable non-bleeding and non-shrinking grouts are used. When studying these quite strong predicted effects, it is worth noting that even with the generally lower pressures used in dam-site grouting, and also with the use of industrial cements and bentonite (i.e. typical more economic methods for dam sites), cross-hole velocity measurements indicate from 1 km/s to 2.5 km/s increase in seismic P-wave velocity (Barton, 2006).

The calculated 8-fold improvement in bulk modulus as a result of the above Brazilian dam site grouting, based on the interpretation of the 3D permeability testing, is not quite matched by the *Q*-based estimates of 3-fold–5-fold increase in modulus seen in Table 8. However, it is easier of course to make a large improvement in a poor-quality rock mass, compared to improving a higher quality rock mass. The reference to 'economic methods' (for dams) above should not be used to infer that tunnel pre-injection methods are not economic. Although apparently costly (for instance 5000 US $ per metre for a 100 m^2 tunnel), systematic pre-injection is needed to prevent *extremely* costly environmental damage. High unit prices, in general, usually reduce overall costs in tunnelling.

Table 8
Comparison of a conservative model with a more realistic model of possible improvements in individual *Q*-parameters, and how these might impact on rock mass properties and support needs (Barton, 2011).

Conservative model	More realistic model
RQD increases (e.g. 30%–50%)	RQD increases (e.g. 30%–70%)
J_n reduces (e.g. 9 to 6)	J_n reduces (e.g. 12 to 4)
J_r increases (e.g. 1 to 2, due to sealing of most of set #1)	J_r increases (e.g. 1.5 to 2, due to sealing of most of set #1)
J_a reduces (e.g. 2 to 1, due to sealing of most of set #1)	J_a reduces (e.g. 4 to 1, due to sealing of most of set #1)
J_w increases (e.g. 0.5 to 1)	J_w increases (e.g. 0.66 to 1)
SRF remains unchanged (e.g. 1 to 1)	SRF improves (e.g. 2.5 to 1, due to consolidation of loose material)
Wet conditions	**Wet conditions**
Before pre-grouting	Before pre-grouting
$Q = (30/9) \times (1/2) \times (0.5/1) = 0.8$	$Q = (30/12) \times (1.5/4) \times (0.66/2.5) = 0.2$
$V_P \approx 3.4$ km/s	$V_P \approx 2.8$ km/s
$E_{mass} \approx 9.3$ GPa	$E_{mass} \approx 5.8$ GPa
$k \approx 1.3 \times 10^{-7}$ m/s	$k \approx 5 \times 10^{-7}$ m/s
10 m tunnel: B 1.6 m c/c, S (fr) 10 cm	10 m tunnel: B 1.4 m c/c, S (fr) 13 cm
Dry conditions	**Dry conditions**
After pre-grouting	After pre-grouting
$Q = (50/6) \times (2/1) \times (1/1) = 17$	$Q = (70/4) \times (2/1) \times (1/1) = 35$
$V_P \approx 4.7$ km/s	$V_P \approx 5$ km/s
$E_{mass} \approx 25.7$ GPa	$E_{mass} \approx 32.7$ GPa
$k \approx 5.9 \times 10^{-9}$ m/s	$k \approx 2.9 \times 10^{-9}$ m/s
10 m tunnel: B 2.4 m c/c	10 m tunnel: sb (spot bolts)

Note: B represents the systematic bolting with spacing c/c in meter, and S(fr) represents the fibre-reinforced shotcrete of given thickness in cm.

13. Some effects of grouting on V_p

"The average values for the whole foundation were 3.18 km/s *before grouting and* 4.74 km/s *after grouting"*. This is one of several cases quoted in the extensive review of seismic characteristics of rock masses (Barton, 2006). The above improvements in P-wave velocities could also be interpreted using Fig. 27. This shows a series of curves with the implied ranges of improvement in velocity as a result of curtain grouting at a 270 m high arch dam in Georgia.

Another figure showing the result of some focussed dam foundation grouting, this time from Norway, demonstrates some dramatic improvements in P-wave cross-hole velocities. These are shown in Fig. 28. The cross-hole velocity increases were measured by a former Norwegian Geotechnical Institute (NGI) colleague. An unexpected amount of foundation damage was caused by the immediately adjacent quarry blasting. The quarry was unfortunately the initial source of rockfill for the dam in question. Some un-anticipated shear displacements (and dilation) on foliation planes was suspected as the source of the very low 'disturbed-zone' velocities. Of course, this grout-penetration scenario was probably one of the least demanding, due to the probable size of the sheared and dilated apertures.

14. Conclusions

(1) Settlement damage caused by too much water flowing into newly driven drill-and-blast or TBM tunnels in jointed rock appears to be widespread, knowing no national borders. It follows the consequences of possible or probable water-table draw-down and resultant pore pressure reduction in deformable overlying sediments. If tunnel inflow is allowed to exceed, for instance, 5 L/min/100 m, and in some adverse situations even as little as 1–2 L/min/100 m, then damage to overlying urban buildings may occur, due to the

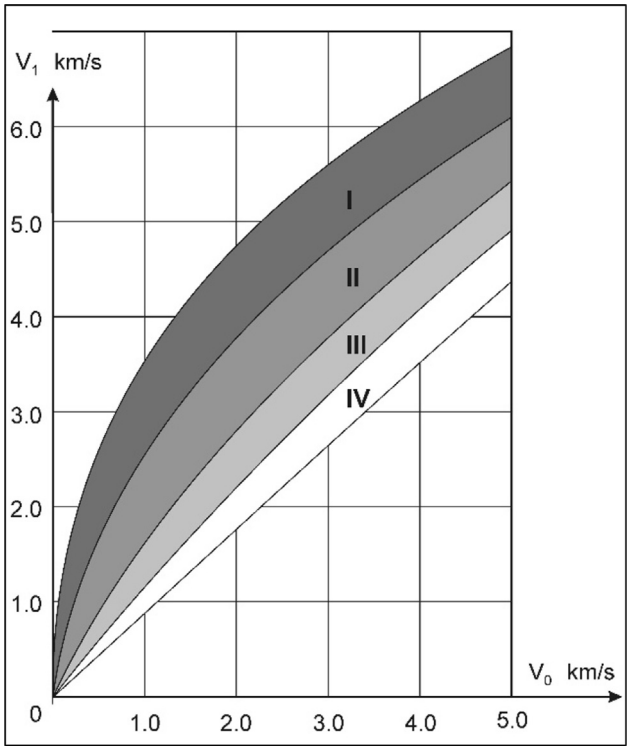

Fig. 27. Classes I to IV of foundation improvement due to grouting: (I – excellent, II – good, III – satisfactory, and IV – unsatisfactory) based on cross-hole velocity monitoring at the Inguri arch dam (Savich et al., 1983).

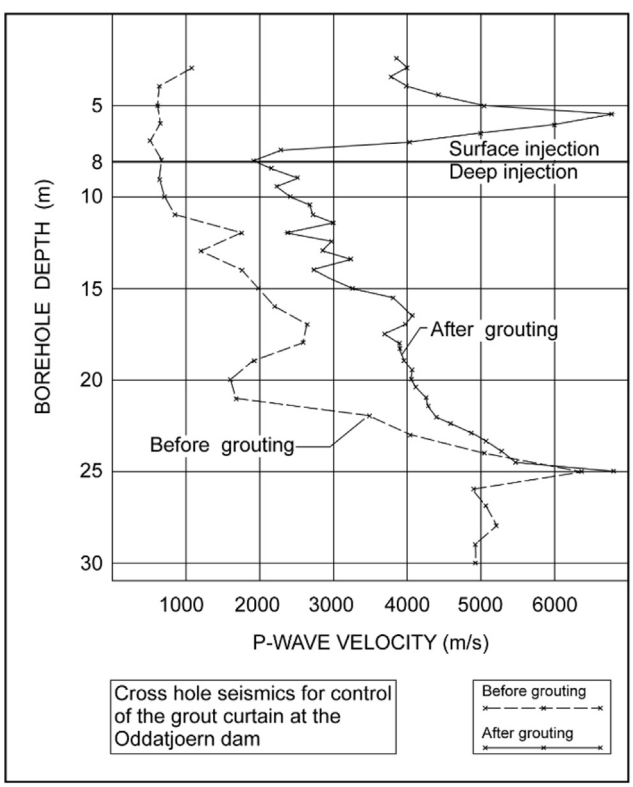

Fig. 28. A dam foundation on gneiss which was damaged by near-by blasting is 'repaired' by both near-surface and deeper injection with cement grout. Shearing and dilation along foliation planes were the suspected cause of the low velocities, and their dramatic improvement by grouting. Note the unprecedented maximum $\Delta V_P > 6$ km/s (By, 1988).

frequency of flat clay-bearing areas tending to have more buildings. The width of a general draw-down trough, or fracture-zone related 'fast drainage track', can reach to 500 m, 1 km, or even 3 km in the case of adverse fracturing at a several hundred metres deep tunnel. Settlement can locally exceed 1 m even when 100 m or more from a leaking tunnel.

(2) Systematic pre-injection of stable, non-shrinking micro-cements and additives is of course the solution. However, due to the *extensional viscosity* of ideally suitable UFC and MFC micro-cements, it is unfortunate that an opinion exists in some quarters that a physical rock joint aperture 8 to 10 times the d_{95} particle size is needed for pre-grouting. This result is due to the use of the filter pump screen to characterise grouts for pre-grouting fine rock joints. The reality, tested with a normal stressed rock joint, and matching experience at large-scale ore-passes, and also matching logic, suggests that $4d_{95}$ is a more practical and defensible criterion concerning minimum physical joint aperture.

(3) For many decades, it has been known that the theoretical hydraulic aperture (e) that can be derived from back-analysis of a flow test using the cubic law is usually considerably smaller than the mean physical joint aperture (E). This is due to the effect of rock joint wall roughness *JRC*, specifically the small-scale JRC_0 relevant to a nominal 100 mm sample length. Available joint apertures, which are mean values due to areas in contact, may nevertheless be very tight, and the use of high injection pressures may become advisable, to allow near-borehole hydraulic jacking.

(4) Contrary to typical 'field instructions', the 5–10 MPa final injection pressure that is achieved by successive reduction of the water-cement ratio should not be held constant when the grout front has apparently stiffened and flow has stopped. At this critical stage, maintaining a high hydraulic pressure, without the usual logarithmic decay while flow is occurring, can damage the pre-injection already done.

(5) Simple methods of estimating permeability from rock mass characterisation using Q_{H2O} are demonstrated. The more preferable performance of systematic Lugeon testing in deviated boreholes, or the more complex 3D permeability testing (hydro-tomography) in the case of critical dam foundations and abutments are also shown. Interpretation of permeability values, however derived, can be made to assist with pre-injection design, using a simplified 'cubic network' approach, initially based on Snow (1968) but with the addition of JRC_0. The recommended selection of no more than two joint sets for flow analysis can also be adjusted for strongly anisotropic permeability, thereby increasing the estimates of joint apertures.

(6) The objective is to simplify the interpretation of permeability, and make it possible to assess pre-grouting needs and which micro-cements to use, including opinion on which pre-grouting pressures may be needed. When flow stops, an immediate reduction of pressure is considered to be an important recommendation for improving the quality of pre-injection.

(7) 3D permeability testing, using multiple simultaneous test holes, has been performed both in dam abutments, in mines, and above metro tunnels to investigate scale effects (possible representative elementary volume (REV)), and anisotropy. In the present context, 3D testing before and after grouting at a permeable dam abutment revealed not only the reduction in magnitude, but also the rotation of the three principal permeability tensors, and the improved modulus of the rock mass as a result of the grouting.

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(8) The structure of the Q-value calculation of rock mass quality, with its emphasis on number of joint sets joint (J_n), and joint properties J_r and J_a, means that it can be used to infer possible, indeed probable, improvements in rock mass properties, such as P-wave velocity and deformation modulus. Grouting at dam sites and in foundations has often demonstrated increases in P-wave velocity as great as 1–2 km/s, indeed sometimes even greater. It is clear that deformation modulus, and hence deformability, over-break, and tunnel support needs are all affected in a positive sense. Designed tunnel support appears too conservative if pre-grouting has been performed, and no credit for this has been taken. Possibly this will change in the future.

Conflicts of interest

The authors wish to confirm that there are no known conflicts of interests associated with this publication and there has been no financial support for this work that could have influenced its outcome.

Appendix. Conversion of hydraulic aperture (e) to physical aperture (E) using the JRC value. The UDEC-BB model of twin-tunnels demonstrates $E > e$, and its reduction with depth following the Barton-Bandis constitutive model.

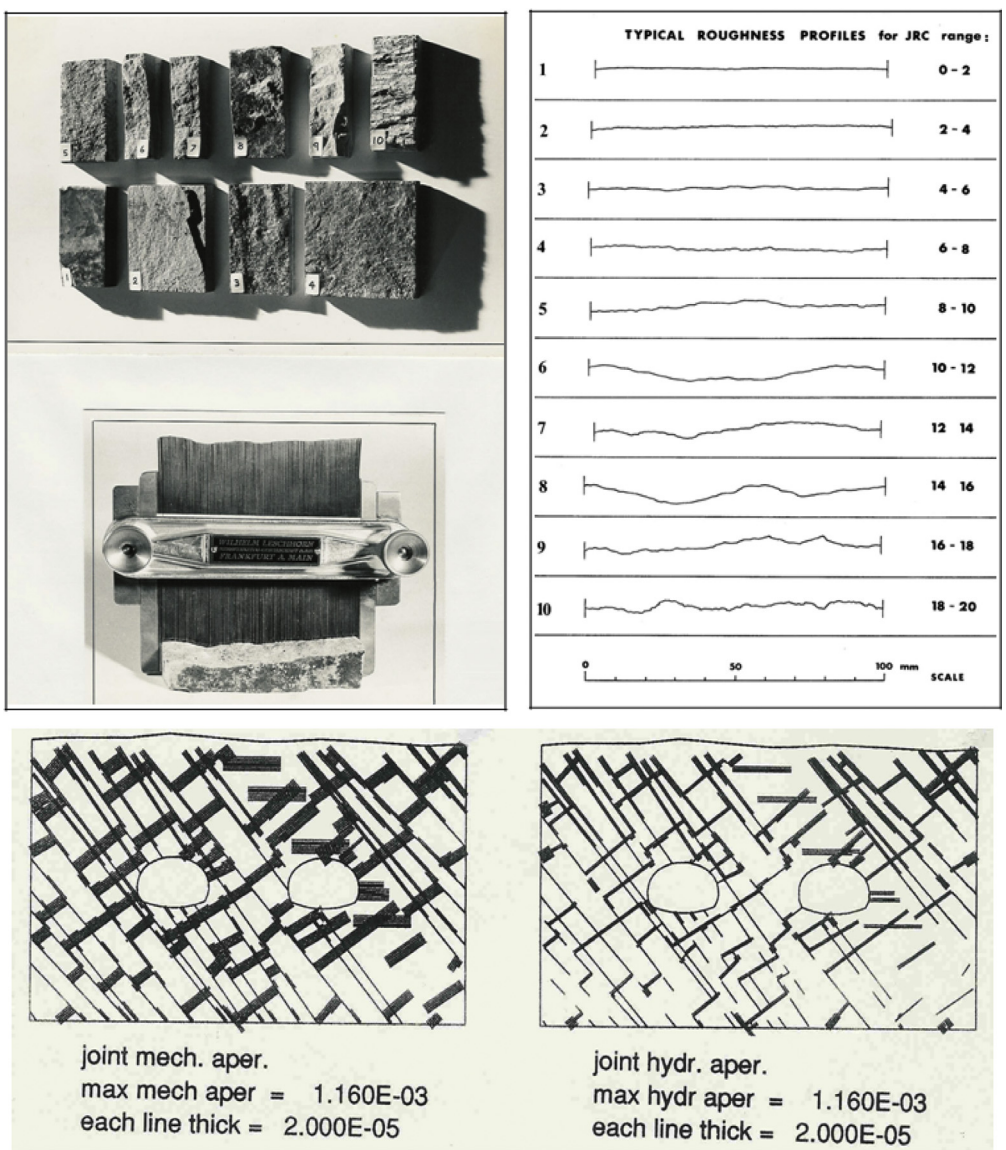

Fig. A1. Top: Examples of joint roughness variation and approximate JRC_0 values (from Barton and Choubey, 1977). Note that Fig. 8 shows how to convert e to E. Bottom: The UDEC-BB model of twin three-lane Oslo Tunnels demonstrates both $E > e$ and the closing effect of normal stress at greater depth (Makurat and Barton, 1988). Note that the maximum (block-corner) channel apertures are large enough (1.16 mm) to give $e = E$ in this example. This can also be seen from Fig. 8. Practical Note: Single-line thicknesses represent 20 μm apertures, suggesting the need for high pressure injection with ultra-fine cements in the case of the dipping sedimentary bedding joints. No pre-grouting was performed, despite advice to the contrary, no steel reinforcement was used in the 'all-under-compression' (?) concrete lining, and post-injection of cracks in the (actually) anisotropically loaded liner was needed. In cold weather, some leakage occurs in the tunnels to this day. If the differentiated apertures of the two sets (cross-joints, bedding) are realistic, a p-q plateau could be expected, the most gapped set being easier to pre-grout.

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ARTICLE IN PRESS

Nick Barton was born in England. He obtained a London University BSc degree in Civil Engineering from King's College in 1966, and a PhD concerning shear strength of fractures and rock slope stability from Imperial College in 1971. He worked for two long periods at NGI in Oslo between 1971 and 2000, eventually as Division Director and then as Technical Advisor. He also worked for four years at TerraTek (subsequently Schlumberger) in Salt Lake City, and was Manager of Geomechanics from 1981 to 1984. Since 2000, he has had his own international rock engineering consultancy, Nick Barton & Associates, based in Oslo and São Paulo. He has consulted on several hundred rock engineering projects in 38 countries during a total of 45 years, has 260 publications as first or single author, and has written two books, one on TBM prognosis, the other linking rock quality and seismic attributes of rock masses at all scales. He has 13 international awards including the 6th Müller Lecture of the International Society for Rock Mechanics (ISRM). He developed the widely used Q-system for classifying rock masses, and for selecting rock tunnel and cavern single-shell support in 1974. He was originator of the rock joint shear strength parameters JRC and JCS and co-developer of the resulting Barton-Bandis constitutive laws for rock joint coupled M-H modelling in 1982, which was incorporated as a sub-routine in UDEC-BB in 1985. He has also developed the Q_{TBM} prognosis method and Q_{SLOPE} for selecting maintenance-free rock cutting and bench-face angles. His chief areas of consulting activity have been in hydropower tunnelling and cavern construction and performance, nuclear waste disposal site characterization, metro tunnels and caverns, and site characterization at high dams. He has given more than 40 keynote lectures in international conferences.

Eda Freitas de Quadros was born in Brazil. She graduated in Civil Engineering in 1967 from the University of Pernambuco. She obtained her MSc and PhD degrees in Engineering Sciences at the Polytechnic School of São Paulo University, in 1982 and 1992, respectively. She is currently the President of the International Society for Rock Mechanics and Rock Engineering (ISRM). In 1975, she joined the Technological Research Institute of the State of São Paulo (IPT), Brazil, where she was Assistant Researcher, Senior Researcher, Head of the Rock Mechanics and Rock Hydraulics Laboratory (1994–1996, 2006–2011) and Head of the Rock Engineering Department (1996–1998). She has worked in the development of equipment and techniques to characterize flow in rock joints and rock masses based on laboratory and field research, including the anisotropic permeability tensors in rock masses applied to dam and mine engineering. From 2003 to 2011, she was Manager for some advanced research and equipment development for long-term elevated temperature creep and hollow cylinder tests in salt rocks for the Brazilian oil company Petrobras. She was Coordinator of the Petrobras Inter-Laboratory Net (2007–2011), a joint research project with IPT for study of off-shore pre-salt cap rocks. From 2011 to the present, Dr. Quadros is Technical Director of BGTech Soil and Rock Engineering, a Brazilian consultancy company in rock mechanics testing and rock hydraulics, located in São Paulo, Brazil. She has taught fluid flow and rock mechanics in short courses at Universities and Institutes in Brazil, as well as given post-graduation lectures. She participated in several PhD and MSc thesis juries. She has taught short courses on Engineering and Flow in Rock Masses (in collaboration with Dr. Nick Barton) in Australia, China, Portugal and Singapore. Dr. Quadros has been President of the Brazilian Committee for Rock Mechanics (1989–1991). Within the ISRM, Dr. Quadros has been a member of the ISRM Technical Commission on Testing Methods since 1985 and was Vice-President for South America during the term 2003–2007, prior to her ISRM president period from 2014 to 2019.

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