An Engineering Assessment of Pre-Injection in Tunnelling

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ABSTRACT

Water is one of the most difficult of the adverse parameters needing control when driving tunnels. If significant inflows are suddenly occurring at the new tunnel face, the needed control is already too late, as post-injection has to be at lower pressure, and even sealing of leaking bolt holes is time-consuming and frustrating work. The water under pressure is drawn down to atmospheric pressure in an irresistible manner, and any soft materials may also be eroded, possibly allowing rock-blocks to fall and sudden in-rushes to be facilitated. Pre-injection of the rock mass some tens of meters ahead of the face, using high pressure if possible, has been shown to 'normalise' progress, largely removing surprises, and making penetration of even serious fault zones possible. This paper addresses successful use of pre-injection, in which the prediction of groutable joint apertures, grout penetration limitations, and possible grout take volumes per cubic metre of rock, can each be estimated, as a result of 5 to 10 MPa pre-injection pressures. Joints are obviously opened more than in the preceding Lugeon tests, and many rock mass properties can apparently be improved if stable, non-bleeding, non-shrinking cement-based materials are used. The one day delay for each grouting screen, when planned for, proves a good investment in overall tunnelling progress.

Keywords: Tunnelling; Pre-injection; Joints; Apertures; Q-parameters; Seismic velocity; Cost

1. INTRODUCTION

Norwegian unlined HEP pressure tunnel designs took many years to reach heads of 1000m, after eventually learning to trust in the larger minimum rock stress that prevents leakage. It has also taken many years to reach 10 MPa injection pressures when pre-grouting ahead of tunnels, where inflows need to be controlled to between 1 and 5 litres/min/100m, or where tunnel stability needs improvement, or both of the above. Three recent high-speed rail tunnels, driven through variable geology under built-up areas towards the capital city Oslo, have benefited from a total of 12 km of systematic pre-injection. These experiences have demonstrated the possibilities for pre-injection prognosis, and most importantly have shown that rock mass properties are improved, and support needs are reduced. Progress is a constant 15 to 20 m per week for the completed tunnels.

The pre-injection performed in the first tunnel was focussed on the natural (above-tunnel) environment, and different classes of inflow were pre-designed, according to assumed sensitivity to ground-water draw-down. The last tunnel was injected more strictly, with emphasis also on the long-term tunnel environment. Completely dry arches (observed), dry walls (observed) and dry inverts (presumed), seem to have been achieved in 99.9% of the typical limestone, shale and igneous-dykes geology. Inflows as low as 1 litre/min./100 m were achieved, roughly equivalent to 10^{-9} m/s permeability. Overbreak was greatly reduced, and support needs also reduced.

Table 1 - Approximate costs of pre-injection needed to achieve various levels of ‘dryness’ in 90 m^2 tunnels.

Inflow (approx.)	Cost
20 l/min/100 m	1,400 US $ /m
10 l/min/100 m	2,300 US $ /m
5 l/min/100 m	3,500 US $ /m
1-2 l/min/m/100 m	≈ 5,000 US $ /m

Do we know the actual effects of this high pressure injection on the rock mass? Can effects be quantified in any way? The answers are yes to both questions, because it has been found from recent Norwegian tunnelling projects that high pressure pre-injection may be fundamental to a good result: i.e. much reduced inflow (usually zero), improved stability, little over-break, and an obvious need for less support. Part of the reason for a good result is that the injection pressures used ahead of Norwegian tunnels are far higher than have traditionally been used. Even at dam sites, where, maximum grouting pressures for deep dam foundations have been limited to about 0.1, 0.05 and 0.023 MPa/m depth in Europe, Brazil and USA respectively: Quadros and Abrahão (2002). Increased seismic velocity is seen as one of the results, plus at least some of the desired reduction in permeability. Various results of pre-injection have been reviewed in Barton (2006), and estimations of improved rock mass properties were presented in Barton (2002).

Fig.1 **-** Left: drilling next pre-injection screen. Right: preparing for primary robotic layer of S(fr), prior to bolting

In Figure 1, the typical appearance of pre-injected tunnels is shown. The foreign visitors with yellow reflection vests, far out-number the specialist tunnel workers. In the left photo, the second (final) layer of S(fr) covers the systematic CT (corrosion protected) bolting, and drilling for the new pre-injection screen has begun in the right invert. In the right photo, the first layer of shotcrete (still curing) has been followed by systematic CT bolting. Due to the lack of overbreak despite the limestone and shale, the permanent support of B + S(fr) appears to be, and indeed is very conservative. However this is for twin-track rail use, and must be dry.

The tunnels described in this introduction were driven in Cambro-Silurian schists, calcitic schists, then shales and nodular limestones, and included about one hundred igneous dykes from a later period during the Oslo graben development. The typical range of Q-values from a total of several kilometres of core logging (Q-histogram method, and JRC) performed by the writer prior to tunnel start-ups, was from 0.01 to 100.

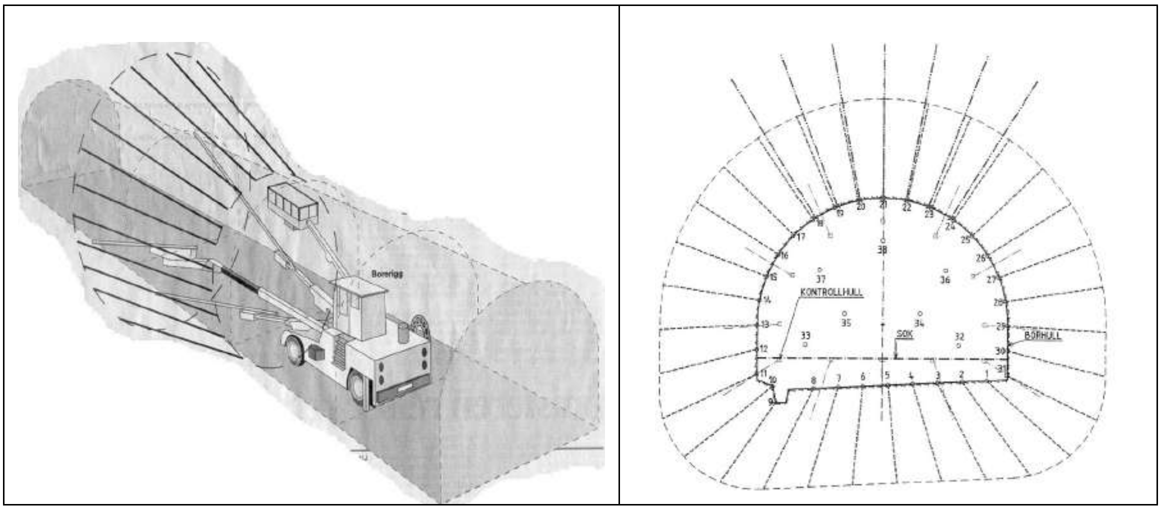


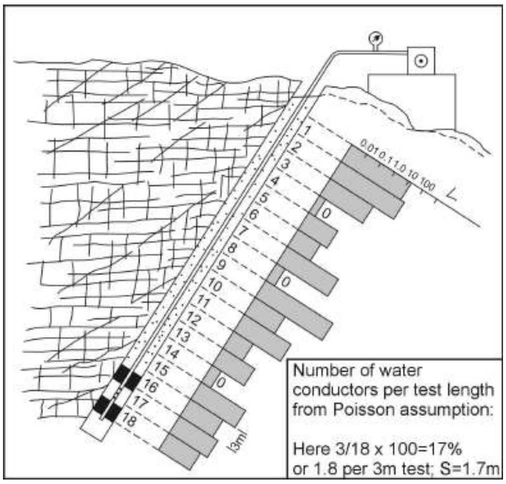
Fig. 2 - Conceptual pre-injection screens, which may vary in length from 20 to 30 m, and have from 30 to70 holes depending on tunnel cross-section.
Hole spacing is from 0.5-1.0 m c/c.

According to a recent Norwegian report by Klüver (pers. comm.), a shallow tunnel in phyllite with 5m of cover was injected at invert level to a final pressure of 6.5 MPa, and to 5 MPa even at the shallow depth of the arch, only 5m below the surface. However, establishment of an outer screen was advised by Klüver in such extreme situations. The reality is that while grout is still flowing , there is such a steep pressure gradient away from the injection holes (from logarithmic to linear depending on joint intersection angle) that 'damage' to the rock mass is limited to local, near-borehole joint aperture increase. These aspects will be discussed later in the paper.

The presumed effects of local high pressures causing joint aperture increase, are probably in the region of small fractions of a mm in competent rock, judging by the local grout take of the rock mass, which may be about 1 to 6 litres/m^3 of rock mass, as shown later. Needless to say, in deeply weathered terrain, grouting pressures need to be limited, as grout-takes may be significantly higher. Careful observation is needed in such cases, to check that the tunnel face stability is not compromised by too high pressure. Besides use of lower pressure, packers can be located one or two meters deeper behind the tunnel face.

2. INTERPRETING LUGEON TESTS FOR GROUTABILITY

Figure 3 shows how one can make a preliminary estimate of the mean spacing of water-conducting joints, using Lugeon tests and the assumption of their Poisson distribution down the borehole, following Snow (1968). A key simplifying assumption is that the water conductors can be roughly represented by a cubic network of parallel plates, i.e. the conductors only, as shown in Figure 4. There are many more joints found in cores through most rock types, due to limited connectivity. The writer has added these between the hypothetical conducting planes, as in Barton et al. (1985).



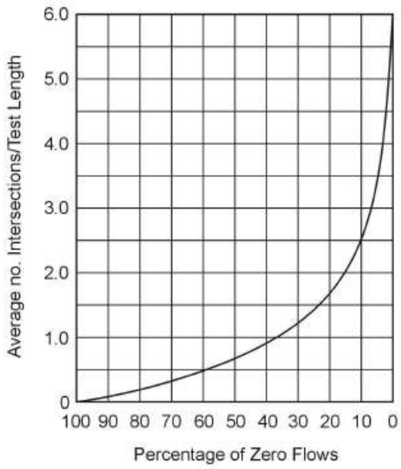


Fig. 3 - Left: Lugeon testing and zero flow sections as a percentage of the total. Right: Poisson distribution for interpreting average number of water conductors. (17% zeros:1.8 conductors/test length: S=1.7m). Snow (1968).

Figure 4 shows a simplified attempt to represent 'reality', using the isotropic model of Snow (1968), with some modifications added by the writer. The reality may obviously be anisotropic and will be much less homogeneous. Because of stress transfer across joints and therefore points of rock-to-rock contact, there will tend to be tortuous flow between the joint walls. The *average* physical aperture (E) of individual joints and joint sets which are potentially groutable, is usually larger than (e) the hydraulic aperture, and depends on JRC, the joint roughness coefficient of Barton and Choubey (1977).

2.1 Basic elements of Snow's method

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

$$K_1 = e^2/12 \times e/S \quad (1)$$

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

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

Making further 'engineering' simplifications that 1 Lugeon ≈ 10^{-7} m/s ≈ 10^{-14} m^2, therefore 1 Lugeon ≈ 10^{-8} mm^2, we can finally write the simplified relation:

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

where (e) and (S) are in mm, and L is the average number of Lugeon. (Each of the above apply to a given structural domain, to the whole borehole, or to a specific rock type).

From equation 3, five example-curves of e-against-S are derived, as shown in Figure 4, assuming a typical range of conductor spacing S = 0.5 to 3.0 m. Although hydraulic aperture (e) is not strictly a 'groutable aperture', it is easy to imagine the likely difficulties of grouting rock masses of less than 1.0 Lugeon, unless we can argue for E > e, or can increase E by using much higher pre-grouting pressures than in the Lugeon test.

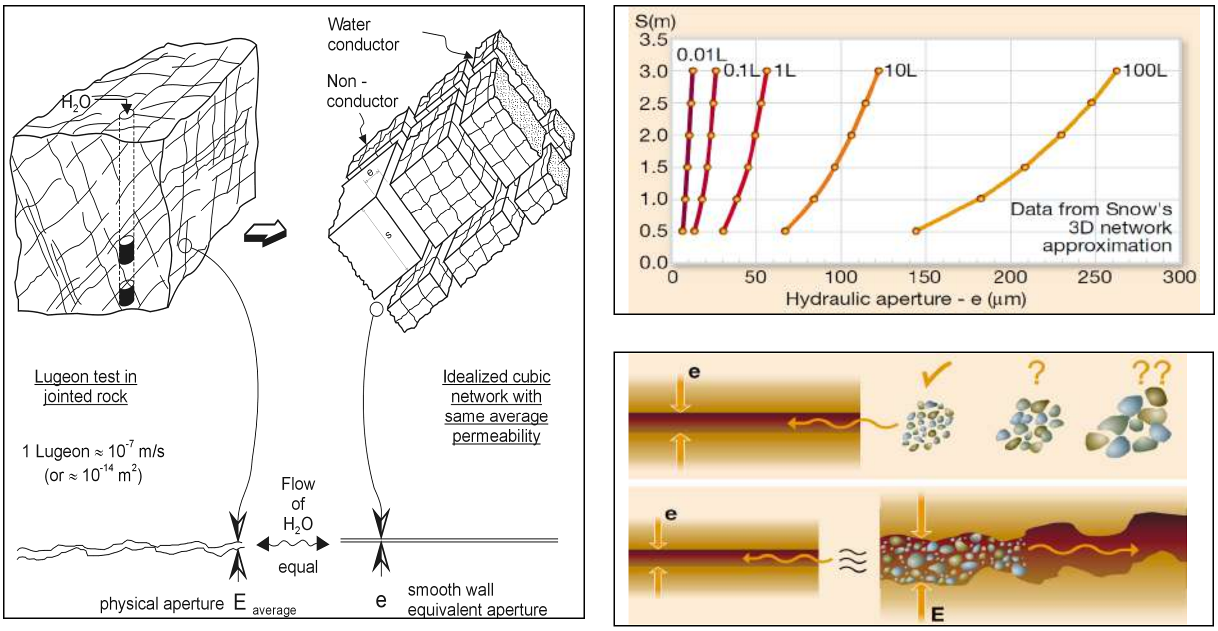


Fig. 4 - Left: Representing a regularly-jointed rockmass with a cubic network of hydraulic conductors of mean aperture (e) and mean spacing (S), based on Snow (1968). Right: Estimates of (e) and (S) from equation 3, and the aperture inequality E ≥ e, which allows grout particles to penetrate real joints (E) even when (theoretical) hydraulic apertures (e) are apparently too small.

3 ROUGHNESS, APERTURES AND PARTICLE SIZES

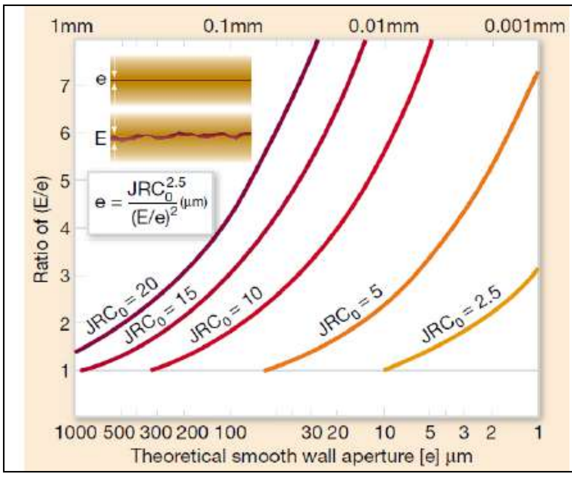
The potential difference between joint aperture (E) and hydraulic aperture (e) has been shown to be dependent on the joint roughness, as shown in Figure 5 and a simple rearrangement of the empirical equation:

$$E \approx (e \times JRC_0^{2.5})^{1/2} \qquad (4)$$

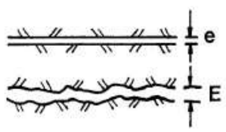
The groutable porosity for three assumed sets of joints as in Figure 4 can, in principle, be written as 3E/S, when assuming an average cubic network, and that (E) gives the average joint space available for flow and for grouting. Clearly this is a tenuous assumption, as the real aperture available for water flow has a distribution of apertures, and as contact points are approached, larger grout particles will be blocked. This is another reason for increasing injection pressures.

We can note that 1.0 litre of grout per m^3 of rock mass could be estimated from average grouted apertures (E) of 333μm at 1m intervals in three perpendicular directions (the cubic model). It is therefore clear that joint deformation is taking place (most likely on most of the water conducting sets), since typical pre-injection quantities in Norwegian tunnels, amount to about 1 to 6 litres/m^3 of rock mass, based on the assumption of an approximate 6 m thick cylindrical annulus of grouted rock around a 90-100m^2 tunnel.

The value of JRC_0 in equation 4 can be estimated from (a/L) x 400 (at 100mm length scale), using profiling. Here (a) is amplitude of roughness over a measurement length of (L), from Barton et al. (1985). A broad selection of joint roughness measurements, made during Q-logging of 1000m of core, prior to construction of the first rail tunnel described in the introduction, revealed a very approximate relationship between JRC_0 and Jr ('joint roughness number') from the Q-system: $JRC_0 \approx 7Jr - 3$. This logging was repeated for the third tunnel.



Depth zones	S(m)	e(μm)	E(μm)
5-15m	0.3	150	218
15-25m	0.4	110	186
25-45m	0.6	80	159
45-60m	0.7	60	138



$$E = \sqrt{e.\ JRC^{2\cdot5}}$$

Grout-Take Estimates / 1m^3 rockmass

Depth zone	5-15m	15-25m	25-45m	45-60m
Grout (litres)	2.2	1.4	0.8	0.6

Fig. 5 - Left: The inequality of (E) and (e) for mated joints under normal closure (or opening) is a function of joint roughness coefficient JRC_0. Right: an example of application of the above methods (e, S, JRC_0 and E), from 1978, at a permeable dam site in Surinam, where joints in the core were roughness-profiled. Barton et al. (1985)

Barton and Quadros (1997) showed that JRC_0, which is proportional to amplitude of roughness (a) divided by length of profile (L_n), is equivalent to the classic 'relative roughness' used in hydraulics. From equation 4 we see some of the possible solutions for hydraulic apertures (e) equivalent to E = 50 μm, when roughness JRC_0 is varied. Examples of JRC_0 (100 mm scale) are given in Figure 6.

Table 2 - Equivalence of (e) and (E) with respect to varied joint wall roughness JRC_0 (from smooth slightly undulating to very rough and undulating).

JRC_0	E (μm)	e (μm)
5	50	44.7
10	50	7.9
15	50	2.9

Joint entry by the grout particles was depicted schematically in Figure 4. Remarkably, a micro cement with d_{95} = 30 microns may well penetrate a joint with e = 25 microns – it is a question of roughness, because E may be >>25 microns. Secondly, there is a certain logic (boundary layer theory) and experimental evidence (Bhasin et al. (2002), for blocked entry (i.e. filtering) if $E < 3 \times d_{max}$ (if there were sufficient numbers of d_{max} particles this would be the 'correct equation, with stationary particles on opposite joint walls).

A modified rule-of-thumb for joint entry limits that is easier to use, as d_{95} is easier to measure, is that:

$$E \geq 4 \times d_{95} \qquad (5)$$

When for instance, d_{95} = 12 μm, and d_{max} = 16μm (as for a typical ultra-fine cement), these relations both suggest great difficulty when E ≈ 50 μm. However a very high water/cement + filler ratio can 'over-rule' here, just as an analagous *busy city street* could easily allow all vehicles to pass fast, if they came 'one-at-a-time'. This would be no way to 'block the street' however – the objective here. If the city street was very 'curving' ('rough' at the kerb) it would need to be much wider to pass the same amount of traffic, especially with a lot of parked cars on each side. Roughness effects, 'slow' particles along the walls, and the need to satisfy equation 4 has also been noted at much larger scale, in ore-passes in mines, where E is replaced by shaft diameter D, and 'particles' may be as large as 1 m diameter.

The above suggests that joint roughness assessment is fundamental to the interpretation of Lugeon tests, as it may help not only to decide upon which types of grout (ultrafine, microfine, industrial cement), but also whether high pressures will be needed. For example, from Figures 4 : if L = 1.0, S = 1.5m and e = 45μm (average values for a given domain) and further, if JRC_0 is only 3 or 4 (or Jr ≈ 1), we would be unlikely to get a successful grouting result even with ultrafine (d_{95} = 12μm), unless we deformed the joints using high injection pressures.

We fail, due to equation 5 size limitations. For typical ultrafine, micro- and industrial cements, E ≥ 50, 100 and 400μm are simple-to-remember approximate limits. (More accurate might be: 0.04, 0.09 and 0.35 mm).

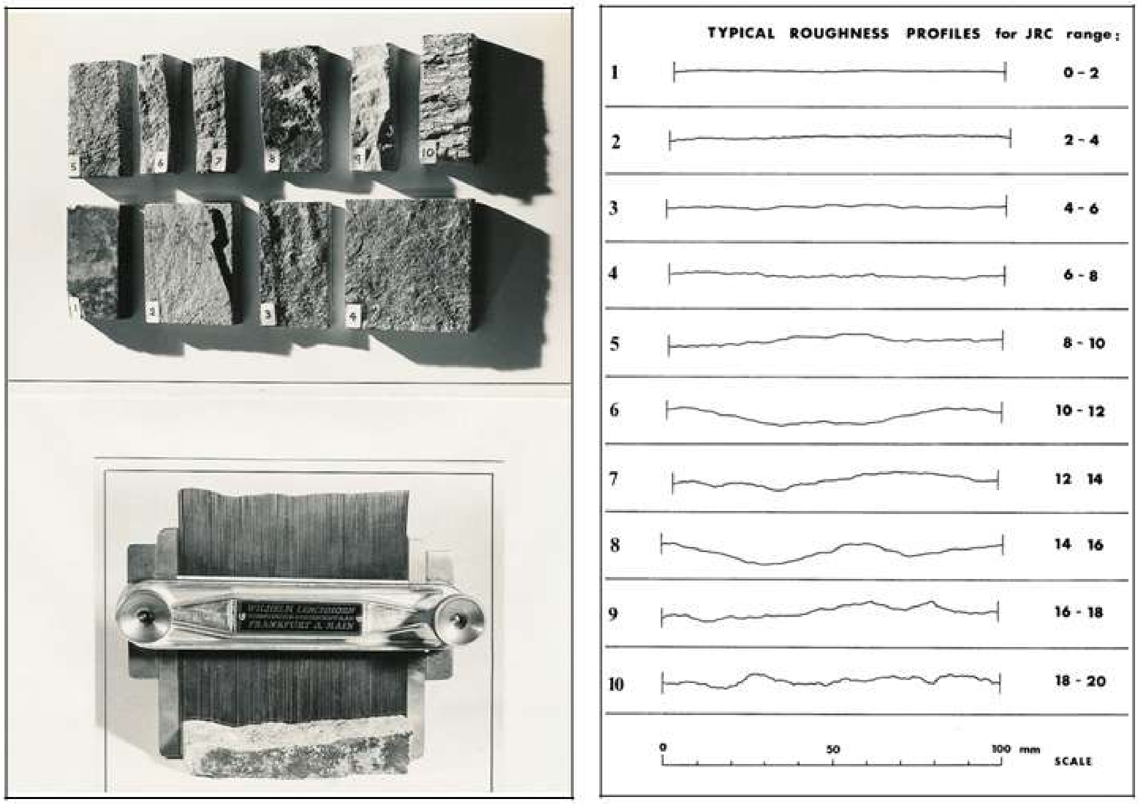


Fig. 6 - Examples of joint surfaces that provided the given ranges of JRC_0 (100 mm scale) roughness. These values of JRC_0 allow an approximate conversion from e to E (Barton and Choubey, 1977)

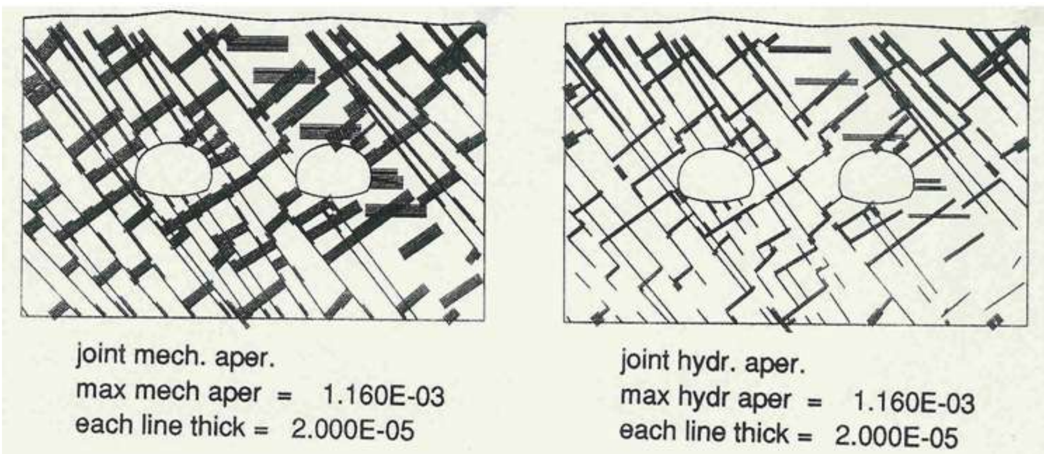


Fig. 7 - Examples of joint apertures E and e in an NGI UDEC-BB model of twin tunnels (The maxima are equal due to corner-of-block channel apertures exceeding 1mm) (Pers. comm.. A. Makurat, 1988).

4. JOINT APERTURE INCREASES DUE TO HIGH INJECTION PRESSURES

In Figure 8, the most fundamental aspect of successful pre-grouting, using elevated grout pressures such as 5 to 10 MPa, is demonstrated by means of the Barton-Bandis normal closure/opening model. The experimental 4th load-unload cycle, following Bandis et al. (1983), is assumed to (almost) represent in situ conditions, following especially the first *'hysteresis-cycle',* when a sampled joint is first re-loaded.

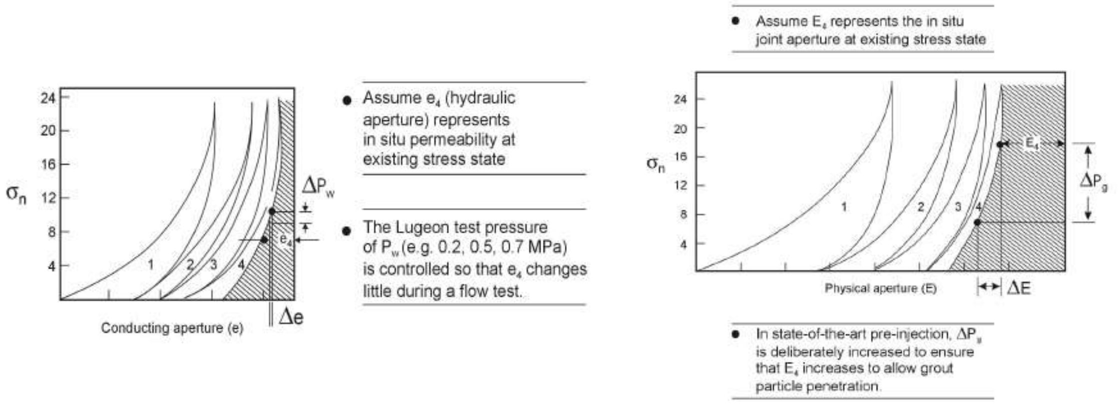


Fig. 8 - The secret of successful pre-grouting, besides grout particle technology improvements, such as use of micro-silica, is to make $\Delta P_g >> \Delta P_w$, so that $\Delta E >> \Delta e$. (Barton-Bandis joint normal closure/opening model)

Conversion between $\sigma_n - \Delta E$ curves and $\sigma_n - \Delta e$ curves shown in Figure 8 is made with equation 4. The Lugeon test with $\Delta P_w \approx 1$ MPa (max.) causes only a small Δe (and also a relatively small ΔE), while a high pressure injection with $\Delta P_g \approx 5$ to 10 MPa, will achieve a significant ΔE (say 10 to 50 μm) depending on distance (R) from the injection hole. This increase may be the difference between success and failure, but sometimes (often?) hydraulic 'jacking' or local loss of contact points, may be the only alternative.

In Figure 9 the different potential pressure- drops away from an injection borehole are schematically illustrated, as joints from different sets are intersected at widely different angles. Pressure decay will vary from logarithmic to linear. Depending on whether laminar or turbulent flow, theory suggests some 40 to 80% pressure loss in the first 1m radius (while flow is still occurring). This is the security against unwanted deformation. One must immediately remove the pressure when flow stops, and have 'stop criteria' such as maximum quantity of grout per hole. If necessary a new round of injection in such holes may be needed, after some setting delay.

If injection pressures are limited and particle sizes are too large in relation to equation 5, and if the available ($E + \Delta E$) physical apertures are too small, then 'water sick' rock may be the result. Thin, individual 'lenses' of badly filtered grout (Figure 9, right-hand diagram) may fail to make contact with adjacent 'lenses', and the rock mass will be wet (maybe even more wet than before) following the grouting. There is a prominent example from Scandinavia where the designer failed to recognise the importance of using higher pressure, and even prevented the contractor from using finer grout, despite analyses as outlined above that showed that average joint apertures were too small for the designer-selected grout.

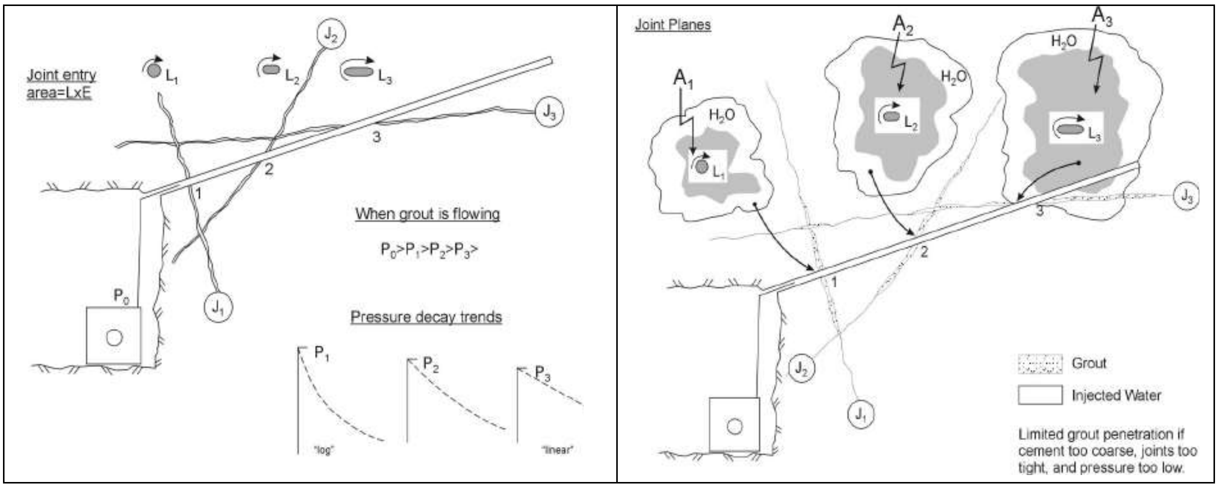


Fig. 9 - Left: Sources of pressure drop and joint entry problems. Grout entry into the differently oriented joints 1, 2 and 3 becomes easier, as local deformation (and a longer elliptical joint entry) is available in case 3 compared to case 1. Finer grouts might be needed if case 1 dominates ahead of the tunnel face. Right: ‘Coffee filter’ effect if grouts are unstable and ‘bleed’, and if too coarse for the joint apertures. If in addition too low injection pressures are used, a disastrous result is guaranteed. This is called ‘water-sick’ rock in Norway, as there is more water in the rock mass after the grouting than before.

The result was wet shotcrete, and leaking bolt holes that needed post-injection, and a one year delay in completing the project, with huge cost over-run. When such a project is also under a city with areas of clay, the added consequences of settlement damage can give tunnelling a bad reputation.

4.1 Some pre-grouting results

From recent compilations of practical experiences, we can derive from Åndal et al. (2001) the following quantities of grout, as used in successful, high pressure pre-injection. Values in parentheses signify presumed ‘escape’ of grout in these two cases, and break-down of the ‘6m grouted cylinder’ assumption. A low percentage of leaking bolt holes of 4 to 5m length is the logic behind an average choice of a 6m cylinder. We can see from Table 3 that *1 to 6 litres* of grout per cubic metre of rock mass is a typical range, for projects where post-grouting water leakages were mostly in the desired range of 1 to 4 litres/minute/100m of tunnel. Tunnel cross-sections were mostly 65 to 95m^2.

Note that an average pre-grouting screen of 25m length, with 30 holes of 50 mm diameter will require at least 1,500 litres of grout just to fill the holes. When distributed through a grouted 6m thick cylindrical volume of 25m length, this nevertheless represents only about 0.1 litre/m^3 of rock mass, so hardly affecting the above ‘rule-of-thumb’ result of 1 to 6 litres/m^3 of rock mass. Tunnels with poor grouting results may typically lie below 1 litre/m^3 in injected volume, resulting in poor connection between the grout ‘lenses’ and possible (continued) wet conditions as a result. Most important are stable non-bleeding, non-shrinking grouts.

Table 3 - Pre-grouting of tunnels data derived from Åndal et al. (2001)

Rock type	kg/m^2 tunnel surface	$\approx kg/m^3$ ++	$\approx litres/m^3$ ++
gneiss	11.0 to 16.5	1.8 to 2.8	1.2 to 1.9
granite	12.0 to 52	2.0 to 8.7	1.3 to 5.8
phyllite	26	4.3	2.9
rhomb porphyry	28 to (99)	4.7 to (16.5)	3.1 to (11.0)
syenite (dike)	30 to (186)	5.0 to (31)	3.3 to (20.7)
fracture zone	19 to 50	3.0 to 8.3	2.0 to 5.5

++ *An average cylindrical annulus thickness of 6m of grouted rock mass has been assumed. A grout density of 1.5 gm/cc is also assumed. This of course varies with the w/c ratios used during the grouting, and is approximate.*

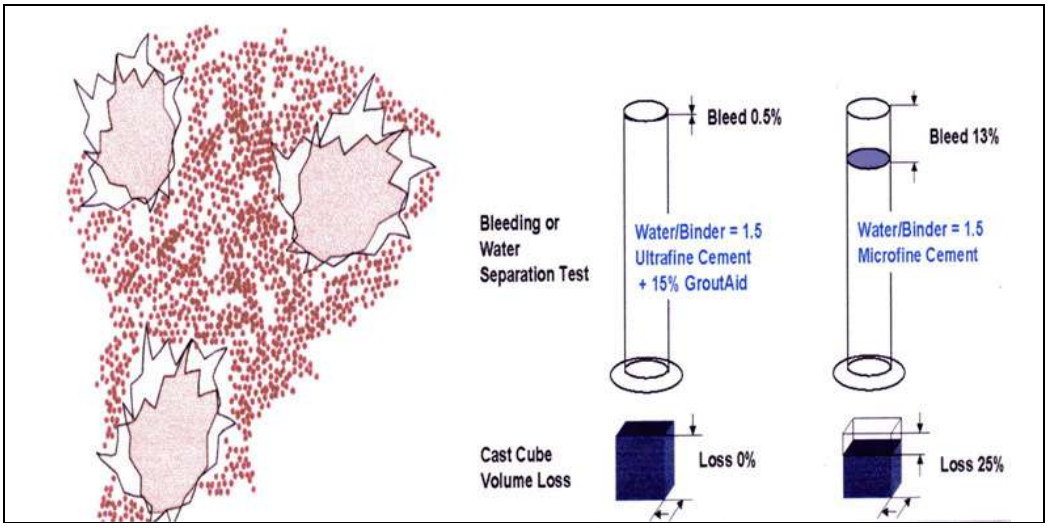


Fig. 10 - Stable non-bleeding, non-shrinking grouts are essential for preventing ‘water-sick’ rock and poor pre-injection results. It is a false ‘economy’ to reject grout additives because of high unit prices. The left drawing contrasts cement particles with micro-silica particles, which are as fine as smoke. (Priv.comm., S. Roald /Elkem)

5. THREE-DIMENSIONAL EFFECTS OF GROUTING

Figure 11 is a compact summary of some unique field tests from Brazil, which indicate that three-dimensional testing using multiple boreholes can help to prove what has been achieved in both successful or unsuccessful grouting. In these particular before-and-after-grouting 3D water permeability tests, which 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 (i.e. from 10^{-7} to 10^{-8} m/s, or from 10^{-5} to 10^{-7} m/s, or from 10^{-4} to 10^{-8} m/s).

In a three dimensional 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 to 8 m, 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 rock mass properties can be expected. As will be seen shortly, even when using conservative assumptions about improvements in Q-parameters, some dramatic improvements in rock mass parameters are indeed predicted.

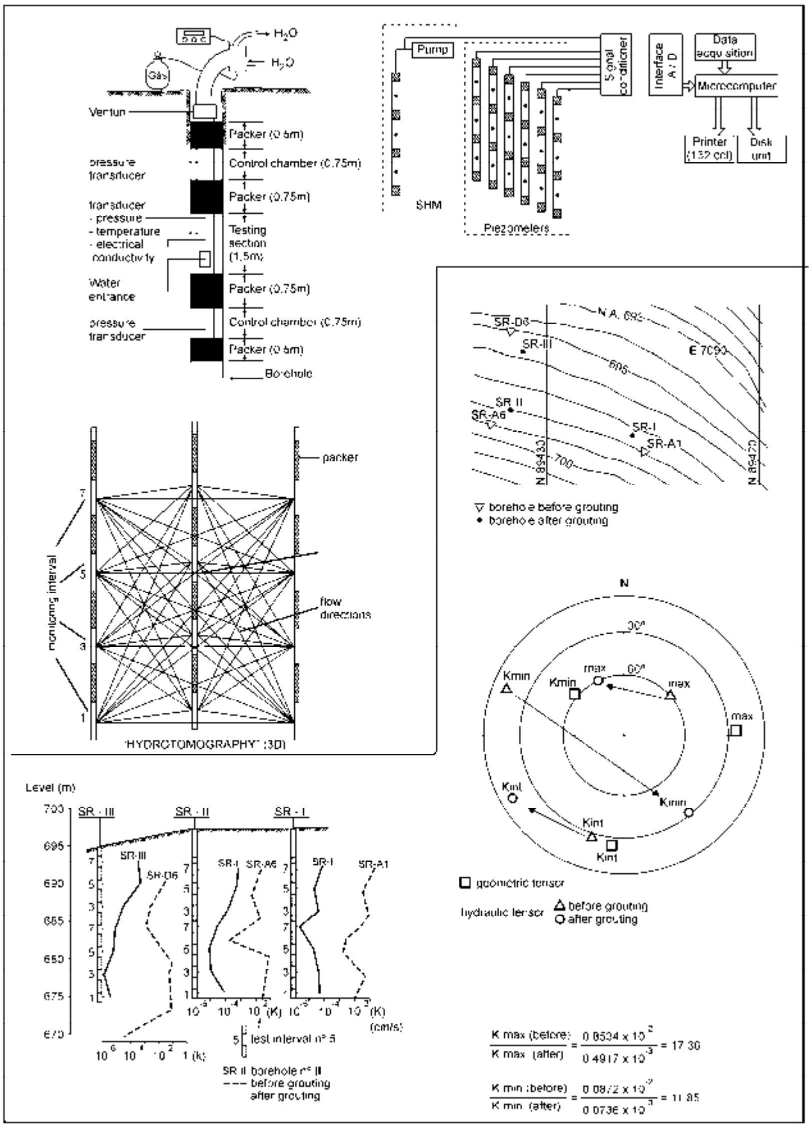


Fig. 11 - Three-dimensional permeability testing performed between three boreholes, both before and after grouting, showed rotation and reduction of permeability tensors, and greatly increased bulk modulus. Despite use of industrial cement and bentonite, the permeable rock mass was greatly improved. Quadros et al. (1995)

6. IMPROVEMENT OF ROCK MASS PARAMETERS BY PRE-INJECTION

Table 4 is a demonstration of how the Q-system can be used to make (obviously) approximate estimates of the potential effects of grouting. Such possibilities were first discussed in Barton et al. (2002), and given in more detail in Barton, (2002). As expected these arguments were criticised by those who like to criticise what they have not produced themselves.

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 in Table 4 is too conservative to be realistic: bigger effects than these must be expected from high-pressure pre-injection, assuming stable non-bleeding and non-shrinking grouts are used. At the bottom of the table, in both columns, the potential changes in rock mass properties caused by the assumed effects of pre-grouting are shown, based on empirical links between Q and these parameters, which were detailed in Barton (2002).

When studying these quite strong predicted effects, it is worth noting that even with the lower pressures used in dam-site grouting, and also with the use of industrial cements and bentonite (i.e. typical traditional methods), cross-hole velocity measurements indicate from 1.0 to 2.5 km/s increase in seismic P-wave velocity. The 8-fold improvement in bulk modulus as a result of the above dam site grouting, based on 3D permeability testing, is not quite matched by the Q-based estimates of 3-fold to 6-fold increase in modulus seen in Table 4.

Table 4 - An illustration of possible effects of pre-injection on the rock mass properties that are described in the Q-system. The V_P and M (deformation modulus) estimates (assuming UCS = 100 MPa) can be checked from Figure 12. The support estimates for 10 m span, ESR=1 tunnels, are NMT (single-shell) based, using Figure 13.

CONSERVATIVE MODEL	MORE REALISTIC MODEL
RQD increases e.g. 30 to 50%	RQD increases e.g. 30 to 70%
Jn reduces e.g. 9 to 6	Jn reduces e.g. 12 to 4
Jr increases e.g. 1 to 2 (due to sealing of most of set #1)	Jr increases e.g. 1.5 to 2 (due to sealing of most of set #1)
Ja reduces e.g. 2 to 1 (due to sealing of most of set #1)	Ja reduces e.g. 4 to 1 (due to sealing of most of set #1)
Jw increases e.g. 0.5 to 1	Jw increases e.g. 0.66 to 1
SRF unchanged e.g.1.0 to 1.0	SRF improves e.g. 2.5 to 1.0 due to consolidation of loose material
WET WET WET WET WET WET WET	WET WET WET WET WET WET WET
Before pre-grouting *Q = 30/9 x 1/2 x 0.5/1 = 0.8*	*Before pre-grouting* Q = 30/12 x 1.5/4 x 0.66/2.5 = 0.2
Vp ≈ 3.4 km/s $E_{mass} \approx 9.3$ *GPa* $K \approx 1.3 \times 10^{-7}$ m/s **10 m Tunnel: B 1.6 m c/c, S(fr)**	*Vp ≈ 2.8 km/s* $E_{mass} \approx 5.8$ *GPa* $K \approx 5.0 \times 10^{-7}$ m/s **10 m Tunnel: B 1.4 m c/c, S(fr) 13**

10 cm	**cm**
DRY DRY DRY DRY DRY DRY DRY	DRY DRY DRY DRY DRY DRY DRY
After pre-grouting **Q = 50/6 x 2/1 x 1/1 = 17**	***After pre-grouting*** **Q = 70/4 x 2/1 x 1/1 = 35**
Vp ≈ 4.7 km/s ***E_{mass} ≈ 25.7 GPa*** ***K ≈ 5.9 x 10^{-9} m/s*** **10 m Tunnel: B 2.4 m c/c**	***Vp ≈ 5.0 km/s*** ***E_{mass} ≈ 32.7 GPa*** ***K ≈ 2.9 x 10^{-9} m/s*** **10 m Tunnel: sb (spot bolts)**

"The average values for the whole foundation were 3.18 km/s before grouting and 4.74 km/s after grouting" which imply an effective Q-value increase from (very approximately) 0.5 to 17, or a Lugeon value reduction from perhaps 2 to 0.06. Such quotations as these can be found in the big review of seismic measurements by Barton (2006). In Figure 14, two figures from this review are shown: one a specific result of grouting on velocity increase at a blast-vibration-damaged dam site, the other a series of curves showing the implied ranges of improvement in velocity as a result of grouting at a major dam site in Russia.

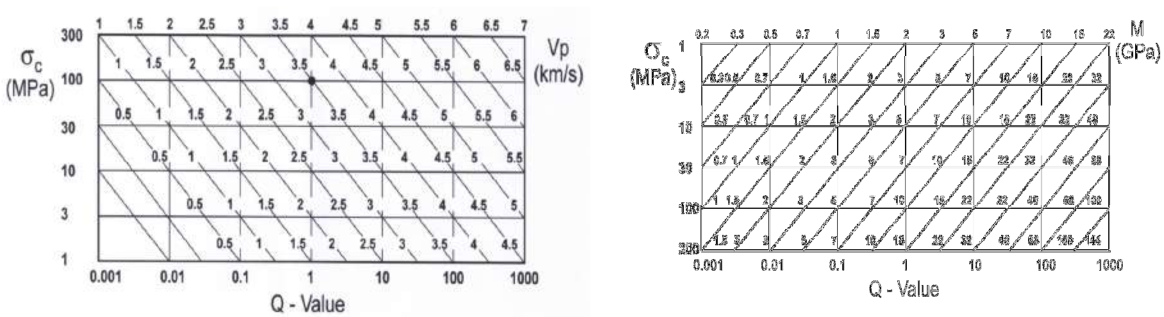


Fig. 12 - Empirically-based links between Q-value, UCS V_P, and M (static deformation modulus). Derivation of these diagrams is explained Barton (2006)

The lack of shotcrete that is suggested from Figure 13, following presumed successful pre-grouting, appears at present to be a radical suggestion, just as 1000 m head unlined pressure tunnels and 10 MPa pre-grouting pressures, also appeared radical in Norway many years ago, and obviously appear radical to all those who have never used such designs. Let us see what happens in the future both in Norway, and in other countries.

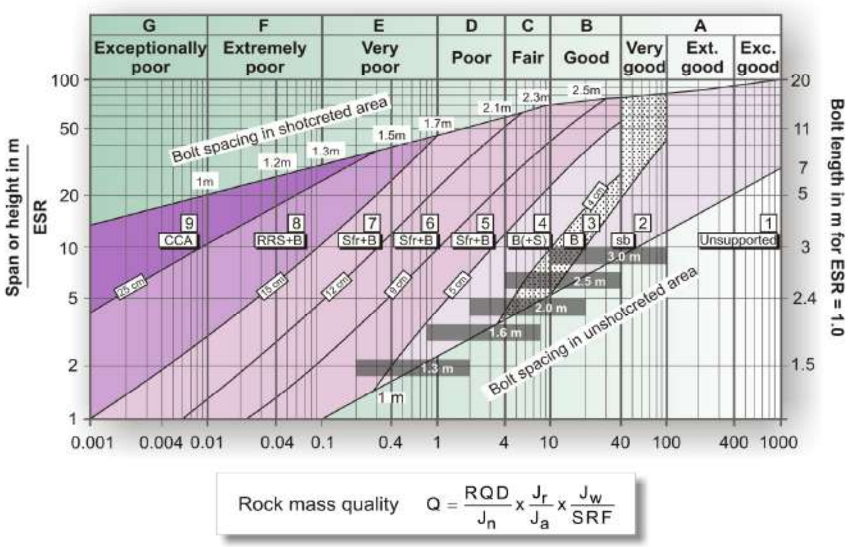


Fig. 13 - Q-system based estimation of permanent support needs for tunnels and caverns, assuming NMT (single-shell) principles for fast economic tunnelling (Grimstad and Barton, 1993)

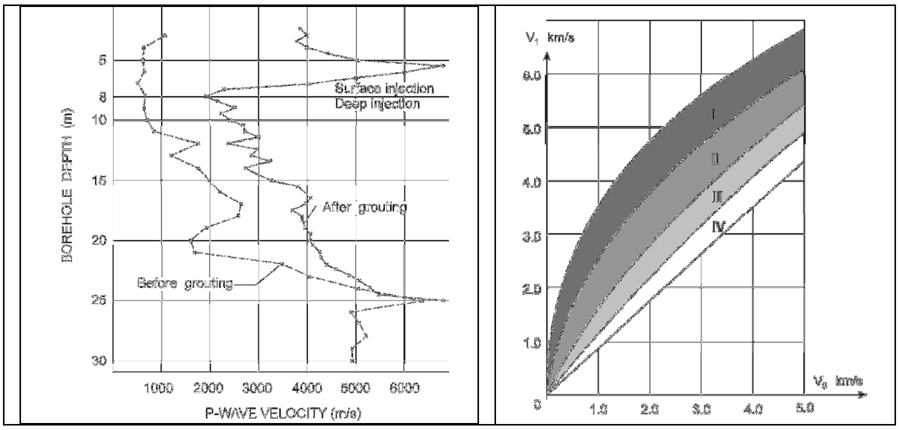


Fig. 14 - Left: Before and after grouting effects on cross-hole velocity at the Norwegian Oddatjørn dam abutment. By (1988). Right: Grouting efficiency (I excellent, II good, III satisfactory, IV unsatisfactory) based on velocity monitoring at the Inguri arch dam (Savich et al.,1983)

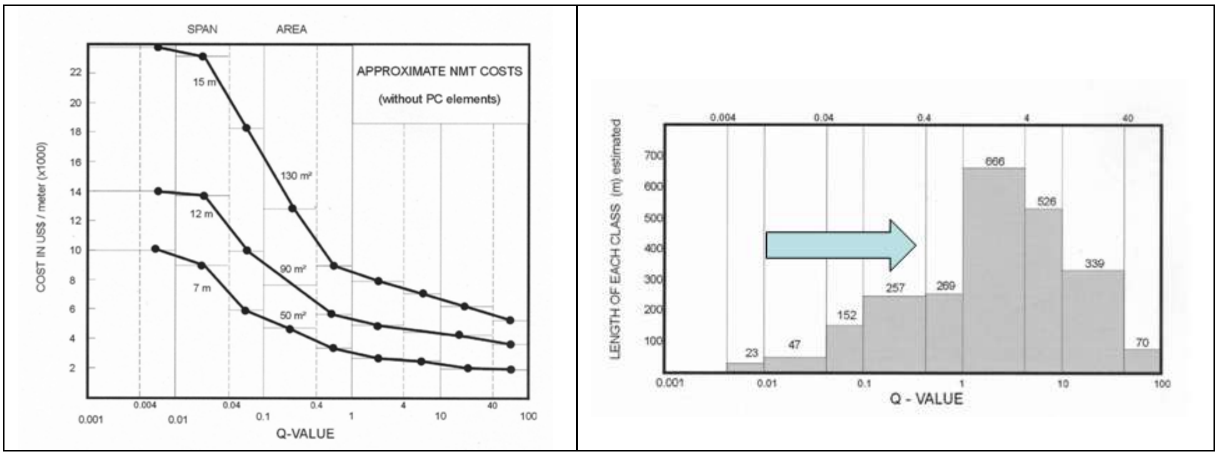


Fig. 15 **-** Because Q-parameters and therefore Q-values seem to be significantly improved even by regular dam grouting, and especially by high-pressure pre-injection, then costs (and time) for tunnelling, which may vary by factors of 10-12 across the spectrum of Q-values, can be expected to benefit also, making the investment in the 'delay' for pre-injection *a very good investment*. (NMT tunnelling cost estimates, and Q-logging of core (see no. of m) drilled along a motorway, from NB&A report, 2002). Arrow suggests possible 'removal' of bad rock.

7. CONCLUSIONS

- Pre-injection can be 'designed' using an analysis of Lugeon testing, and conversion of hydraulic to physical apertures. High pressures, use of additives, and efficient drilling equipment are needed.
- Q-parameters, Q-values, moduli, velocities, permeabilities are each improved, plus reduced support.

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