FJELLSPRENGNINGSTEKNIKK BERGMEKANIKK/GEOTEKNIKK 2023 PRE-GROUTING OF TRANSPORT TUNNELS IN JOINTED ROCK FOR SUCCESSFUL CONTROL OF WATER Forinjisering av transport tunneler i ....

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

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

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SUMMARY

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

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

SAMMENDRAG

Forinjisering er en effektiv måte å fortrenge vann og sterkt begrense tilsiget til tunneler, dersom det praktiseres riktig. Sprekkesett forsegles suksessivt, og permeabilitetstensorer er kjent for å rotere og redusere i størrelse for hvert sett. Dette er målt under 3D-permeabilitetstester. Behovet for tunnelsikring reduseres faktisk ved vellykket forinjisering, men ikke når våt sprøytebetong eller lekkende boltehull sees etter mislykket forinjisering. Muligheten for tørre tunneler avhenger av bruk av stabile ikke-krympende sementfugemasser med mikro-silika tilsetninger. På grunn av ekstensjonsviskositet blir sistnevnte fravalgt hvis man bruker den uønskede filterpumpen som er foretrukket i enkelte land. Partikkelstørrelser bør passe til estimatene for gjennomsnittlige fysiske sprekkeåpninger (E). Hydrauliske åpninger (e) beregnet fra permeabilitetstesting er idealiserte glatte parallelle plater. De er mindre, matematisk avledede åpninger, så de representerer fysisk ikke-eksisterende mål for å bestemme sementpartikkelfinheten, ved bruk av enten ultrafin eller mikrosement, eller industriell Portland-sement. Tommelfingerregelen for at E trenger å være større enn circa 4 x d95 er blitt bevist eksperimentelt i naturlige sprekkeprøver. Sprekkeåpningforskjellen E ≥ e skyldes hydrauliske tap på grunn av ruhet. Disse åpningene er omtrent like når de er større enn circa 1,0 mm. Et dårlig forinjeksjonsresultat som våt sprøytebetong og lekkende boltehull kan også skyldes for lavt injeksjonstrykk. Lokal nærborhulljekking er nødvendig, med begrenset risiko når flyt av injiseringsmasser fortsetter fortsatt. Det er et uunngåelig logaritmisk til lineært trykkfall fra injeksjonsborehullet ut i de kryssede sprekker, med minst 50 % trykktap innen 1 m for Newtonske væsker, og åpenbart mer for rue sprekker ved bruk av sementholdige masser med deres Bingham-fluid kohesjon og friksjon. Trykket må imidlertid ikke holdes når strømmen har stoppet. Injeksjonstrykket må åpenbart senkes når det ikke er nødvendig i tilfeller med store strømninger nær overflaten eller i permeable knuste soner på dypet. Dersom man av en eller annen grunn ikke bruker stabile sementer med nødvendig mikrosilikatilsetning, vil det uansett være nødvendig å bruke lavere trykk, men man må da forvente dårligere penetrering og volumreduksjon ved herding, altså sannsynlighet for våt sprøytebetong. Forfatterne vil trekke på sine erfaringer fra konfidensielle sakkyndige og rettserfaringer fra flere pre-og post-injeksjonsprosjekter i Norge og i utlandet.

Nøkkelord: forinjisering; settningsskader; høytrykk; mikro-silika; sprekkeåpninger

Introduction

Pre-grouting *ahead of tunnels* has three main functions: to control inflow into the tunnel, to make tunnelling progress more predictable in case of poor-quality rock masses, and to limit groundwater drawdown above the tunnel. This helps to avoid settlement damage caused by consolidation of clay deposits beneath built up areas. Towns tend to be built where terrain is flat, due to the same clay deposits. Green areas are also largely protected if groundwater levels are maintained. 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, even football stadium owners (Stockholm), and sometimes farmers and forest tourgoers (Oslo).

Tunnelling may trigger ground-water drawdown and damage to infrastructure, even multiple 1m subsidence to roads near metro tunnels (São Paulo) with resulting damage and even abandoned houses. Furthermore, and distant from cities, TBM need not get stuck in mountains (multiple instances) with good probe drilling and timely impermeabilization (pre-injection) routines. This has become increasingly possible on more recent TBM. It has taken TBM manufacturers decades to achieve, as the realization of potentially more challenging ground is more widely accepted.

Some relevant back-ground information

During extensive testing of a large number of cements for their grouting abilities twenty years ago, *Roald and Saasen, (2004)* discovered that ideal stable non-shrinking grouts with micro-silica slurry additive have *extensional viscosity*. Such ideal grouts tend to be excluded from selection when tested in filter- pumps with their artificial screens that force flow-separation, and which subject the tested grout to a maximum pressure gradient in one artificial test plane, quite different from grout flow in rock joints and fractures, and different also from the NES test. These findings will be discussed in more detail later.

In the Bærum Tunnel west of Oslo, where inflows were in the range 1 to 2 litres/min/100m and locally lower than this, the *first author* had previously logged all core and related it to Lugeon data, to Q-parameters and to seismic velocities on behalf of Jerbaneverket, today's BaneNor. The Q_{H2O} method was developed on the same occasion *(Barton, 2007)* using more than 1.4km of Lugeon test results, with lower permeability at increasing depth not only due to improved Q-values.

Figure 1. The Bærum Tunnel west of Oslo with systematic pre-grouting of 5km length and 1 to 2 litres/min/100m result with no need for the pre-installed infiltration wells. This project is a good example of the gradual improvement in achievable inflow limits, which has reduced in Norway from about 30litres/min/100m some 40 years ago, to about 10litres/min/100m about 30 years ago, and down to best results of about 1-2litres/min/100m in the last 10 - 20 years. Note the dry shotcrete in the current last 50-60m of this tunnel, in contrast to the occasional small wet patches. Filter-pump discrimination against stable grouts with micro-silica has been shown to cause 'reversal' of the dry/wet shotcrete experience – meaning mostly wet shotcrete and a minimum of dry patches. This is an undesirable and totally unnecessary result.

As will be seen later, a slightly altered Q-rating method is used, reversing Jr/Ja to Ja/Jr. In two earlier tunnels along the same Oslo-Asker line (Yong-Asker) the core-logging and permeability interpretation methods to be outlined in this paper were also used, enabling predictions of the different grout particle (injection cement) needs. Unfortunately, the main consultant chose three different pre-injection strategies for different 'milieu classes', taking care of the external environment but not the internal tunnel environment, so there remained some unnecessary and unwanted minor drips of water.

Settlement damage due to groundwater drawdown in overlying sediments

There is 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, especially metro lines, is often needed. This is exactly where great care is essential (pre-injection), in preference to the optimistic: 'We do not expect settlement damage'.

The most remarkable draw-down due to tunnelling known to the first author reached to 2.9 km. It was apparently transmitted from a single TBM tunnel major-inflow event and was presumably transmitted along fractured zones beneath two intersecting valleys in Sri Lanka. Thousands of houses were damaged, and hundreds of wells dried up – each in the neighbouring valley. A small river used by farmers disappeared. There was no other tunnel within tens of kilometers to explain this costly phenomenon.

Unfortunately, double-shield TBM, even with (or because of) reliance on bolted and gasketted PC-element liners, have seen instances of draw-down up to 1km distant, due to mis-interpretation of permeability, sometimes in the form of *sub-horizontal* fractures and unexpected connectivity in otherwise good quality ‘vertically-fractured’ gneiss and amphibolite. The problem is the continuously repeated ‘delay’ of PC-element installation in the tail-shield, 15 to 20m distant from the advancing tunnel face. The usual advance represents a normal half, one, or two days ‘delay’ for the liner to be in place. This ‘delay’ can be extended in time if a troublesome weakness zone, or fault *(Barton, 2013)* or hard mixed-face is involved, the latter causing unexpected cutter damage and delays in the maintenance cycle *(Macias and Barton, 2022).* The temporarily unsupported length behind the cutter-head may suffer over-break. This occurred in troublesome sub-sea kilometers (ch 20 -24km) in the UK/France Channel Tunnel, where three joint sets caused unexpected wetness – with the benefit of hindsight - an expected result since 30 to 60m under the seabed.

Pre-grouting effectively reduces EDZ to prevent draw-down

Figure 2 illustrates some coupled-process discontinuum modelling using the Itasca Inc./Dr Peter Cundall code UDEC-BB. It was performed by Dr. Karstein Monsen for a disputed case of draw-down and leakage at a metro tunnel that was not pre-injected – but should have been. The left diagram shows an example of nearly complete drawdown of the groundwater table. On the right is the flow of water in principally the rougher and more permeable sub-horizontal joints at a more limited drawdown stage. The smoother, tighter sub-vertical joint set suffered the most shearing.

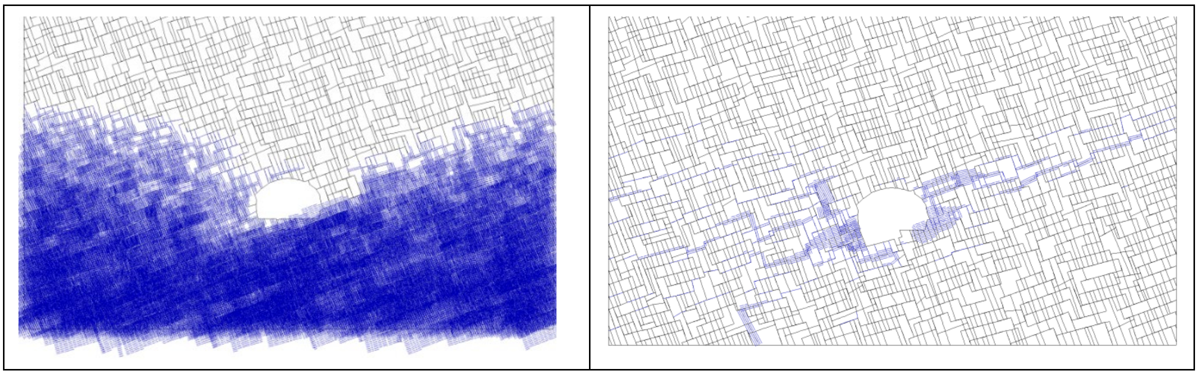


Figure 2. Coupled UDEC-BB tunnel model of groundwater drawdown and inflow along the joints when no pre-injection is performed – but should have been.

The question arises of not only how to perform pre-injection to prevent such potentially damaging drawdown (how many holes, what injection pressures, which grouting materials, which starting water/cement ratios), but also of what is actually happening when injecting cement-based grouts into the active (one diameter thick?) load-bearing rock ‘cylinder’ that will later surround the new tunnel.

Figure 3 shows a diagrammatic attempt at representing the roughly ‘cylindrical’ surrounds of a tunnel to represent EDZ_1 , EDZ_2 and EDZ_3 which are each defined at the top of the figure. The idea with the symbolic and *right-side and left-side* sketches is to represent the potential effect of pre-injection, *or not*. EDZ_2 represents rock joint adjustments. Permeability here is likely to be minimized by pre-injection, but most important is the EDZ_3 representing blast damage. This is formed *after the pre-injection* when the tunnel is excavated, so represents a more permeable zone (assisting drainage if still needed) on the inside of the grouted cylinder.

Note that in the 'boxes' at the top of Figure 3, the Q-value is assumed to increase as also predicted in *Barton, Buen and Roald (2002)* and in *Barton (2003).* This was later confirmed by direct observation of reduced support needs in well-designed Norwegian tunnels (such as the Bærum Tunnel) that had been systematically and successfully pre-injected for a continuous 5km.

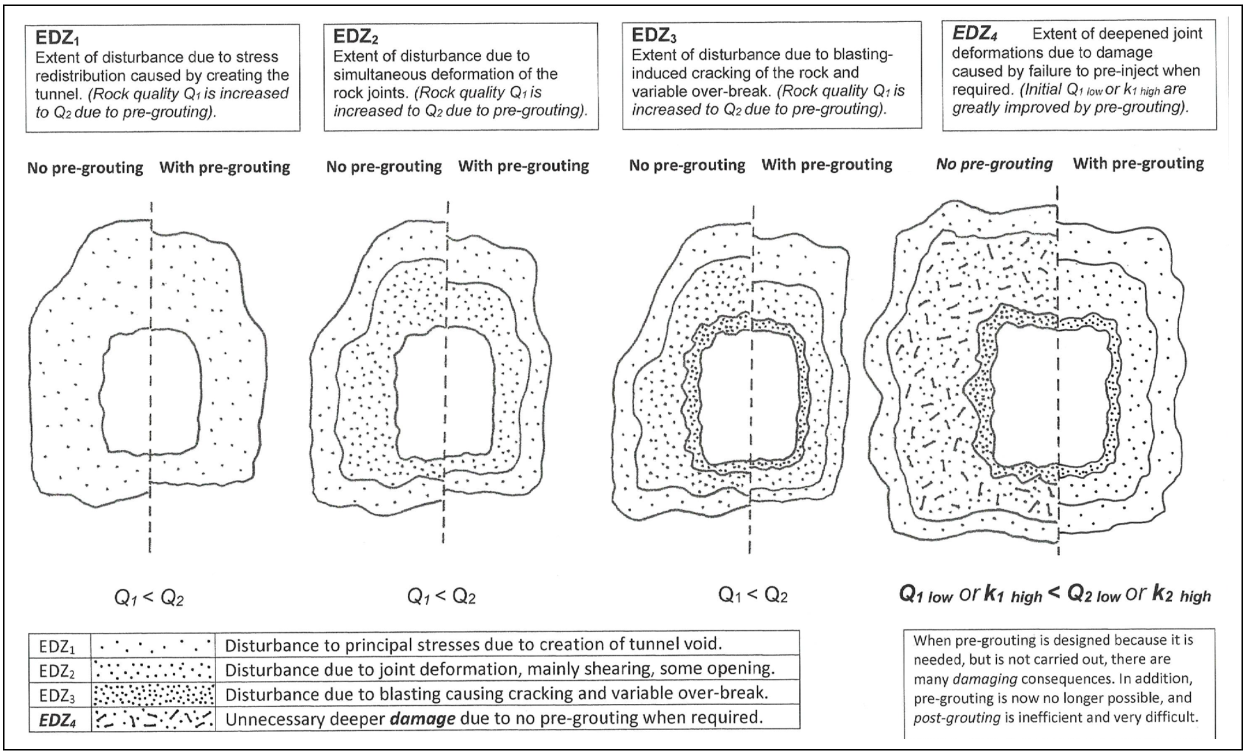


Figure 3. The concept of tunnel $EDZ_{1\ to\ 4}$ and their reactions to pre-injection, or no pre-injection.

The benefits of micro-silica additives and evidence of the 4 x d_{95} rule

Roald, Nomeland and Hansen (2002) and *Roald and Saasen (2004)* described extensive measurements of the penetration properties of various cement-based injection materials in a thorough analysis of some 20 manufacturer's grouting cements. The most important physical quality-control result of this research is that bleeding and volume loss can be reduced to negligible amounts by using micro-silica slurry additive. Figure 4 is a graphic summary of the diverging consequences of not using or using such additives.

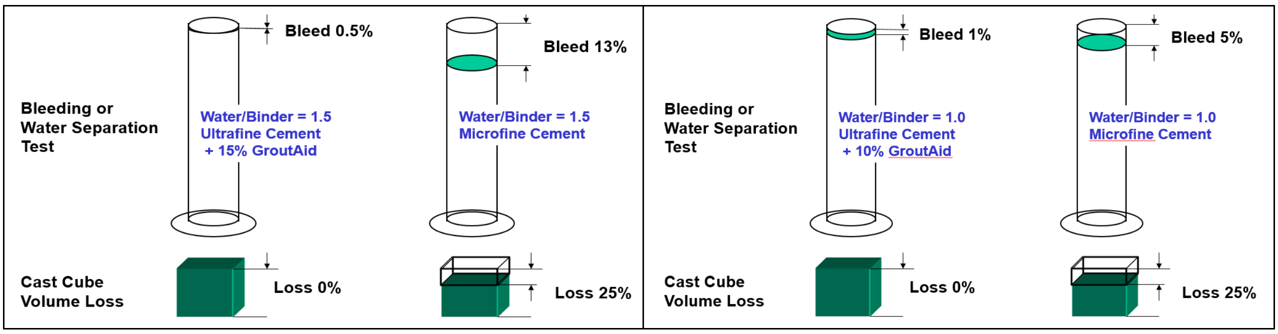


Figure 4. Micro-silica additives to eliminate bleed and volume loss when grouting, (Roald, Nomeland and Hansen, 2002).

A major practical problem is that unstable grouts *without* micro-silica do not tolerate high-pressure injection as well as stable grouts, and they may shrink when setting due to filtration and bleeding.

The filter pump was also investigated during the studies of 20 years ago. *Roald and Saasen (2004)* found that it gave inconsistent results, but worst of all it caused *stable grouts to be rejected* because of their resistance to flow separation due to an *extensional viscosity* much higher than the conventional shear viscosity. Those using the filter pump seem to be unaware of this.

Filter pumps are apparently in regular use in Sweden and were therefore used at the very large Stockholm Bypass twin-motorway project *(Creütz and Osterman, 2019).* This seems to have caused the exclusion of stable grouts at this project. This in turn increases the risk that the cement-only grouts will not tolerate high injection pressures. The incomplete grouting of a limited number of joint sets, perhaps only one set, as very frequently observed, may be the unintended result. This has allowed small but environmentally damaging inflows especially in northern parts of this major project. These have been protested by environmental groups and by affected property owners.

Opinions flowing from filter pump use include illogical 'rules-of-thumb' (see opinions given in *Creütz and Osterman (2019* and in *Dalmalm and Janson (2001).* Equipment-caused rejection of the well-known and rock joint tested '4 x d_{95}' physical joint aperture groutability limit is described below and in some following figures. The numerous grouting tests performed in Elkem's Materials Laboratory twenty years ago included the NES apparatus, with successively reduced apertures (100, 75, 50µm) until grout mixes with successively reducing water/cement ratios (2.0, 1.3, 1.0, 0.7) were finally experiencing blockage. Just four (4) of the cement manufacturers more than twenty (20) cements were successful in penetrating the 50µm NES artificial 'smooth fracture' opening, and these had 10 to 15µm d_{95} particle sizes, as advertised by their manufacturers. They were therefore demonstrating '3 to 5' times d_{95} penetrability.

In tests at NGI during our extensive UK Nirex nuclear waste project (1990-1996), it was found by *Bhasin et al. (2002)* that 4 x d_{98} was the mean physical joint aperture that could be grouted. The estimate of aperture was made using the JRC conversion method *(Barton, Bandis, Bakhtar, 1985)* shown later, utilizing the interpreted hydraulic aperture (e). It was found possible to inject this rock joint (a large-diameter joint sample of welded tuff from Sellafield) using a stable super-fine grout while the joint was under a normal stress of 3.5MPa. It had a water-flow interpreted hydraulic aperture (e) of 25µm. The grout had a re-checked d_{98} particle size of 12µm. With JRC = 6, the e = 25µm aperture converts to a mean physical (E) aperture estimate of 47µm. In this case E ≈ 4 x d_{98}.

Interestingly, and significantly, this 'particle-size' rule also applies to the many-orders-of-magnitude-larger ore-passes in mines. In this case the 'slow' particles (blocks of ore) next to the walls define a very approximate parabolic velocity distribution. Blockage results when the d_{95} block size causes 4 x d_{95} to exceed the ore-pass diameter as seen in Table 1. These empirical results from *Hambley, Pariseau, Singh (1983)* are well-known in the mining industry. On occasion, explosive 'bombs' are rolled down on trolleys to release blockages in the often 5m to 6m diameter ore-passes.

Table 1 Guidelines for preventing blockage (and the need for 'bombing') in ore passes (Hambley, Pariseau, Singh, 1983). Figure 6 provides some visual images – of larger block flows.

Ratio of ore-pass dimension (D) to block dimension (d)	Relative frequency of blocking	Flow probability
D/d > 5	Very low	Almost certain flow
5 > D/d > 3	Often	Variable
D/d < 3	Very high	Almost certain not to flow

The joint aperture inequality E > e and its advised use in grout selection

The fundamental mismatch of the mean physical (E) and theoretical hydraulic aperture (e) of joints was already graphed in 1972 *(Barton, 1972)* and confirmed again following coupled *in situ* tests in subsequent work in the USA *(Barton et al. 1985).* The aperture E ≥ e and change of aperture ΔE > Δe) joint flow data gradually being collected was updated by Quadros in *Barton and Quadros (1997)* as shown in Figure 5. The concept is by now widely accepted following numerous PhD studies. Experience of ΔE > Δe was specifically recorded at the unique $8m^3$ in situ flatjack-loaded block test in 1980-1981 performed by TerraTek, where the mismatch of joint closure and change of hydraulic aperture was confirmed *(Barton, 1982).* As must already be clear, 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. It was a surprise to find that Swedish grouting designers have apparently focussed just on the *(non-physical* so not actually existing*) hydraulic aperture,* at their largest project (by length > 2x18km with spans 20-30m): the surprisingly narrow-pillar Stockholm Bypass, as described by *Creütz and Osterman (2019).*

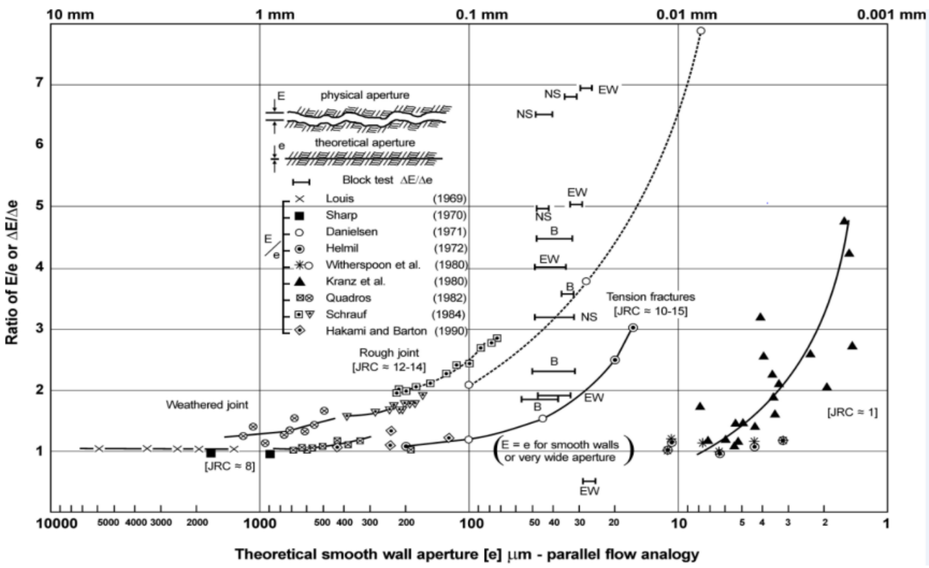


Figure 5. Experimental evidence for the mismatch of hydraulic and physical apertures started more than 50 years ago (Barton, 1972) and was assembled in Barton et al. 1985 and updated in Barton and Quadros, 1997. The E ≥ e mismatch should not be ignored when designing grout.

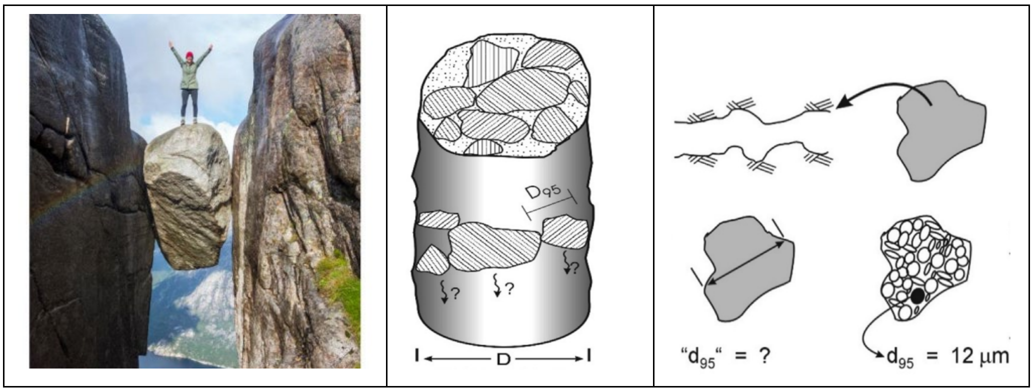
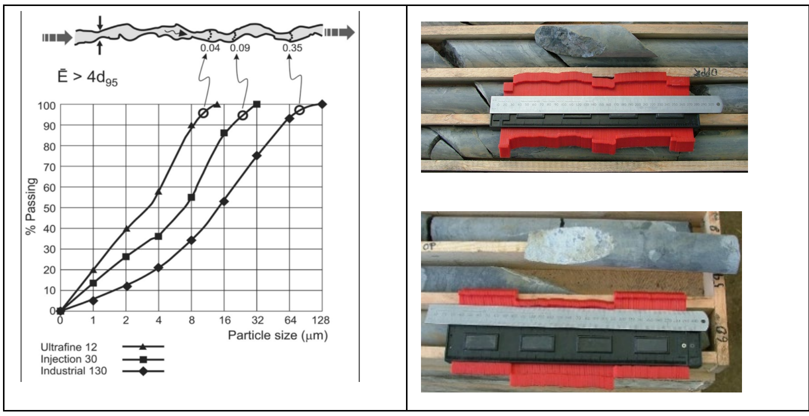


Figure 6. The flow of blocks down rough-walled ore-passes in mines follows the rule-of-thumb principles for flow of the ten-thousand-times smaller cement particles in rock joints. In the case of the latter the selection of finer grouts (ultrafine instead of micro-cement) together with locally increased joint apertures due to carefully controlled hydraulic jacking may solve the problem of grout penetration.



JRC_{mean}	3.5 Jr = 1	7.5 Jr = 1.5	12.5 Jr = 2	17.5 Jr = 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

Figure 7. Left: the rule-of-thumb for grout particle penetration with Swedish cements represented. Centre: estimating joint roughness JRC on core at rail tunnels west of Oslo. Right: for a range of JRC (and Jr from the Q-system) and selected hydraulic apertures, coloured numbers represent estimates of mean physical apertures. Green, through yellow, orange and red need ultrafine, micro, and industrial cement, respectively. Using 'e and not E' for grout selection and ignoring roughness JRC (or Jr) has been a costly omission at recent major projects.

Filter pump testing is non-representative of the penetration of stable grouts

Ideal grouts consisting of micro- or ultrafine cement with micro-silica additive to ensure stability and no shrinkage do not take kindly to filter-pump inquisition in an artificial screen taking all the pressure drop. Ideal stable grouts have extensional viscosity that is many times larger than shear viscosity. A desirable grout with lower water/(cement+filler) ratio gets blocked on the screen, and results in illogical opinions (Dalmalm and Janson, 2001 and also in a Norwegian-Swedish text book) that apertures 8 to 12 times larger than d_{95} are needed. Stable grouts are rejected by 'filter-pumping'.

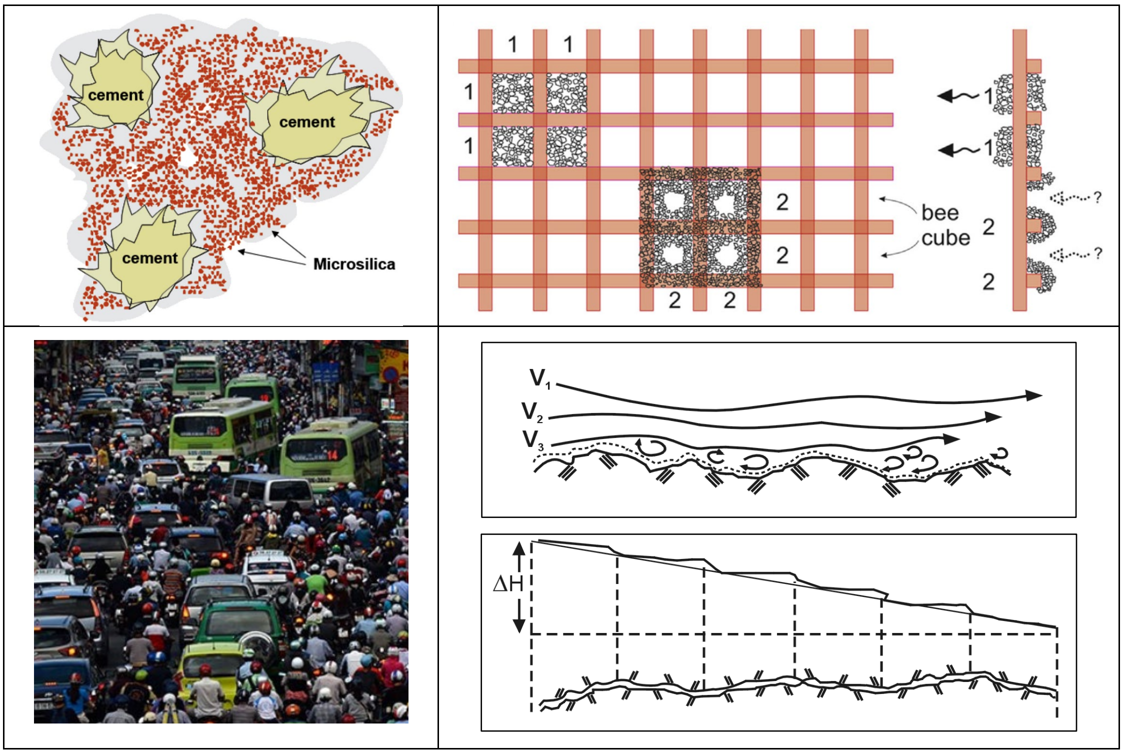


Figure 8. The extensional viscosity of ideal stable grouts causes them to be de-selected in the most artificial and extraordinary testing method – the filter pump screen, as pointed out by Roald and Saasen, 2004. The reality is flow losses over a significant flow distance, and a more or less parabolic velocity distribution (Barton and Quadros, 1997 and 2019).

The real situation, like the *analagous crowded road* in Figure 8, is that there are ‘roughness-induced’ wall losses (slower speed due to parked cars) giving a parabolic-style of velocity distribution, caused in the micro-world by small pressure losses due to grout flowing around interlocking joint-roughness asperities. Widening the road (or local hydraulic joint jacking) gets more grout and traffic further, and if this can occur on intersecting roads even better, but the widening is clearly limited. Anyone who proposes a toll-plaza (the analogy to a filter-pump screen) will cause artificially delayed flows of zero relevance to normal traffic (and grout) flow. *The filter-pump screen that symbolizes the traffic plaza delay does not exist or belong in jointed rock masses.* Imagine a joint aperture of 0.1mm and flow of grout for the desired several meters in each joint intersected. In our analogy the traffic flow equivalent is *tens of kilometers*, not the 10m in the traffic plaza. Local hydraulic jacking of the joints (or local addition of new lanes) gives desirable benefits for locally increasing the speed and reach of the grout (and traffic).

High injection pressures for local joint widening

Some 10 to 15 years ago, three rail tunnels west 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 injection pressures used (5 to 10MPa), will have significantly and deliberately exceeded assumed local minimum rock stress, an unfortunate limit suggested by *Gustafson and Stille (1996)* which apparently is still influencing pre-grouting practice in some countries/institutions. It is almost a perfect recipe for injecting only the most permeable joint set. In Norway higher pressures are traditionally chosen to hydraulically jack the joints, *but this only occurs in the immediate neighborhood of the injection holes*. The radial reduction in pressure, linear to logarithmic, is rapid and very important as it undermines (takes energy from) the action of too low injection pressures. It is indicated in Figure 9.

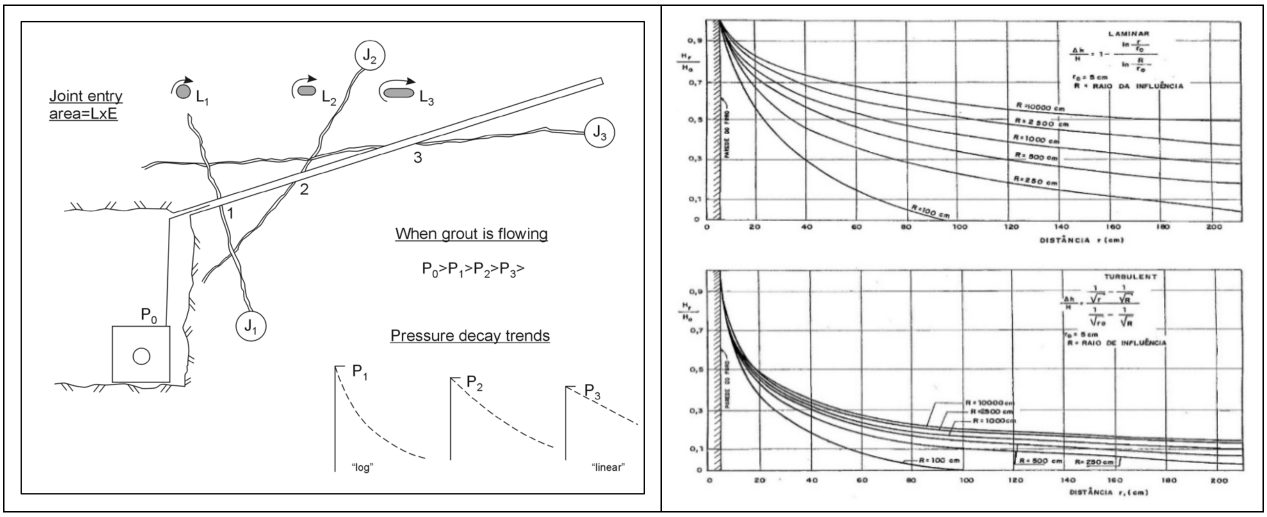


Figure 9. The rapid pressure decay within the joint planes while flow is occurring means that higher pressure pre-grouting is needed. There will be greater difficulty in injecting joint intersection L_1 than L_2 or L_3. The theoretical curves show pressure decay as a function of radial distance (0-2m) across a perpendicularly intersected joint plane. The two right-hand diagrams apply to Newtonian laminar or turbulent flow. More than half the pressure is lost within 1m of the injection holes (Cruz, Quadros and Correa Filho, 1982). It will be worse with Bingham fluids.

To emphasize the role of pressure-drop-while-flowing the following tunnelling example is illuminating – also illustrating the advantage of a *blocker-screen* in some cases. Refer to following Figure 10 from *Roald, Nomeland and Hansen (2002)*. A shallow urban tunnel in phyllite, with 5m of rock cover, was injected at invert level to a final pressure of 6.5MPa, and to 5MPa even in the shallow depth of the arch *(Barton, 2003, Klüver and Kveen, 2004).* However, the establishment of an outer ‘blocker’ screen as illustrated in Figure 10 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 seen in Figure 9. Without this pressure dissipation mechanism, cases such as the above would obviously be damaging to over-lying rock and soil. Application of such high pressures is discounted in most countries, maybe because of incorrect practice (e.g. seeing the damage that may occur if holding high pressure when flow has stopped?) and failing to appreciate the above flow-dependent pressure decay. This is a real safety valve against unwanted hydraulic fracturing, as opposed to the frequently desirable joint jacking.

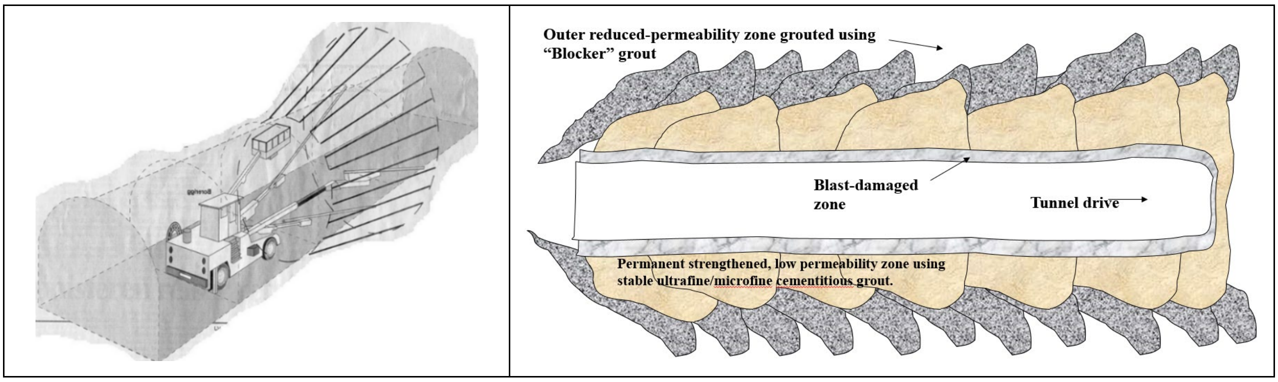


Figure 10. The fast-setting blocker grout principle illustrated by Roald et al. (2002). Three key advantages are a shortening of the setting time so that tunnelling can continue, 'protection' against unnecessary grout loss, and provision of containment when high-pressure injection is needed but the surface (or parallel excavations) are too close for comfort.

In this connection it can be noted that 1 to 5 litres of grout per m^3 of rock mass is a typical result for pre-injected tunnels, based on the assumption that a roughly 5 to 6m thick cylinder surrounding the typical 12m span road tunnel is grouted. A rock mass with three perpendicular sets of joints and a mean 1.0mm aperture would take a theoretical 3 litres of grout per m^3. An 'active porosity' of 0.3% would have been injected in this case. This is far higher than most natural rock mass porosities. In view of the need for stress transfer in a rock mass, *and much tighter joints prior to grouting*, significant joint jacking must be assumed.

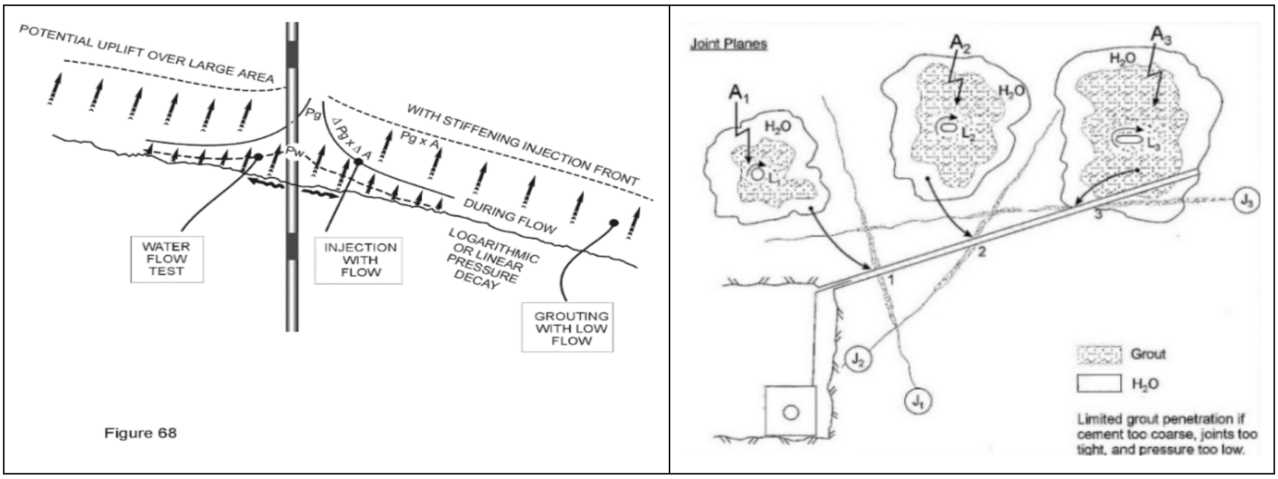
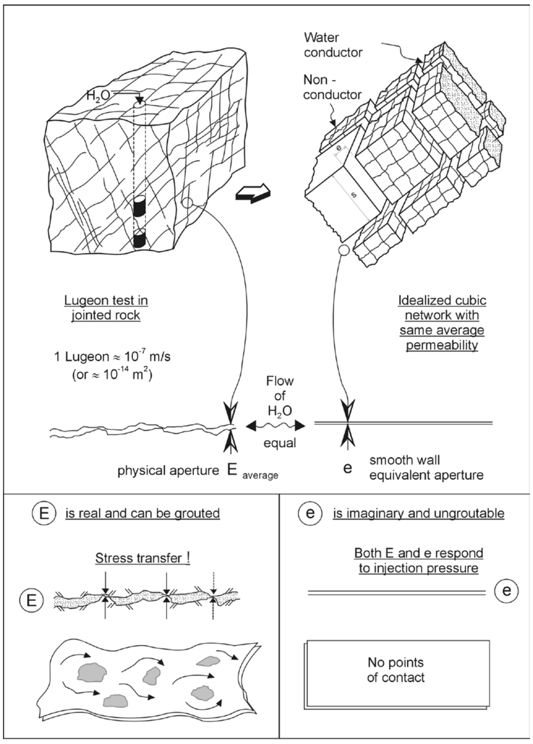
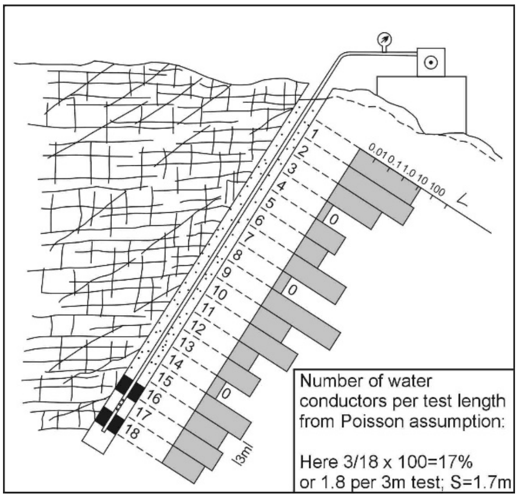
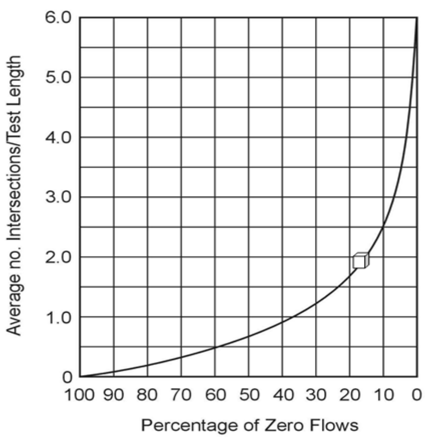


Figure 11. Left: Contrasting joint deformation during a Lugeon test, or during possible joint jacking, or if pressure is maintained when flow has stopped giving potential and damaging uplift. This must be avoided. Right: Too low pressures and too coarse grout without micro-silica combine to make 'coffee-filter' water-sick rock with more water after grouting than before.







Assume:

1) $K = e^2/12$ for one parallel plate

For one set of plates/joints:

2) $K_1 = e^2/12 \times e/S$,

where (S) is the mean spacing of water conducting joints (see rotated 'cube' of conductors).

From Snow [20] and Louis [21]: Rock mass permeability on average, is estimated by flow along ***two** of the three sets* of parallel plates:

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

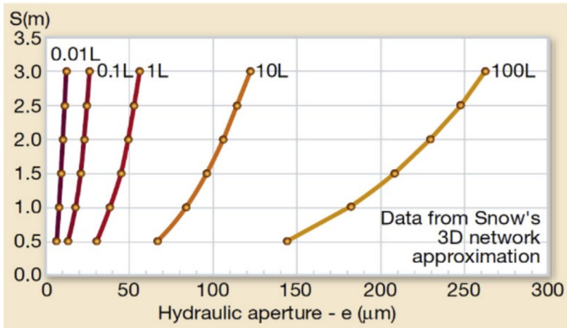
Making further 'engineering' simplifications that 1 Lugeon ≈ 10^{-7} m/s ≈ 10^{-14} m^2, we obtain:

4) 1 Lugeon ≈ 10^{-8} mm^2.

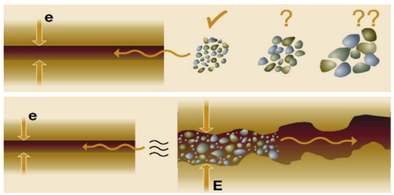
Therefore, the simplified relation as follows:

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

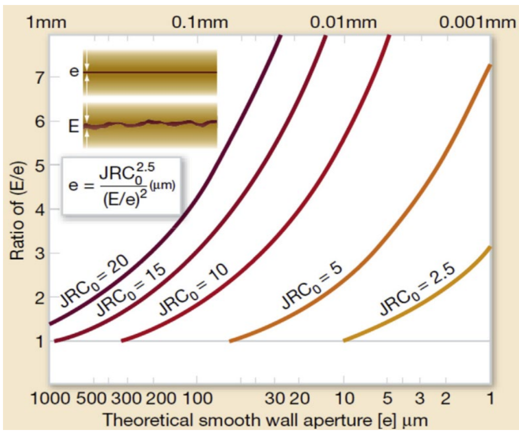
where (e) and (S) are in mm, and L is the average number of Lugeon. See the **five (5) Lugeon** curves:



Each of the above apply to a structural domain, to the whole borehole, or to a specific rock type.



Pre-injecting the lowest permeabilities relies on the aperture difference E > e illustrated above.

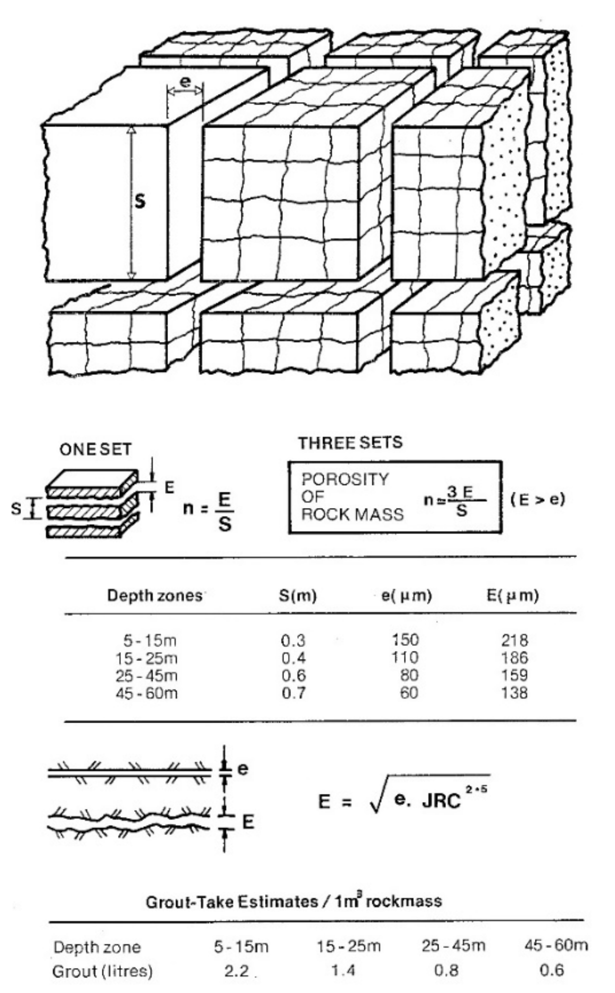


Converting e to E_{mean} using JRC

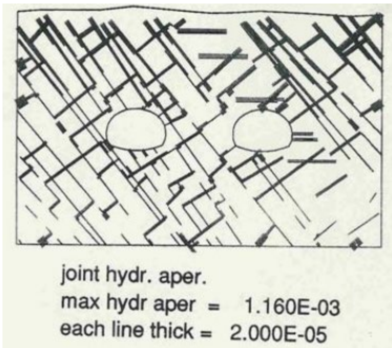
Figure 12. Representation of the rock mass as a cubic network of conductors, <u>but a maximum of only two sets can contribute to flow</u> (equation 3). The % of zero flow Lugeon stages used in a Poisson distribution gives mean spacing (S) of conductors following Snow (1968). Groutable E_{mean} from e and JRC (Barton and Quadros, 1997). The procedure was presented in detail in Barton (2003).

A simplified approach to pre-grouting design – deriving mean physical joint apertures

In Figure 12 on the previous page a suggested 'work-flow' logic is suggested to enable something more than guesswork, or worst still: 'filter-pump design' of pre grouting strategies. Appropriately it starts with an analysis of the Lugeon testing, using a simple statistical 'mean aperture' approach and simplified 'cubic network' assumptions. It is based on a modification of *Snow (1968).* As an example of the proposed method's application: at the Jong-Asker project's 2.7 km long Tanum Tunnel, analysis of the systematic permeability measurements, using the methods summarized in Figure 12, indicated tight hydraulic apertures (e) of 25 to 45 μm, but due to the roughness correction (E > e) most physical apertures were estimated to be from 45 to 150 μm. The *in situ* rock mass porosities (n ≈ 3E/S: see Figure 13 below) varied from 0.004 (shales) to 0.12 (nodular-limestones). See analyses in *Barton (2003).*



A 1988 UDEC-BB tunnel model illustrating the mismatch of modelled joint apertures e and E *(Makurat and Barton, 1988).* Note depth effects. Maximum e and E are block-corner channels and both have apertures > 1mm, and therefore are given equal magnitudes (1.16mm).



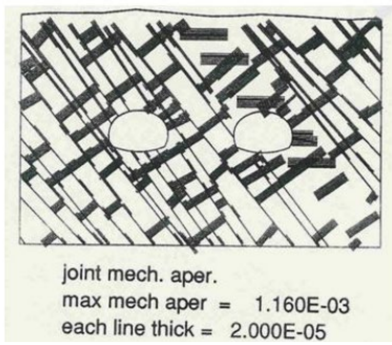


Figure 13. Left: A relatively early (1978) application of the Snow (1968) 'cubic network' method, together with the introduction of the new JRC method of distinguishing between e and E. This was applied at a permeable dam site in Surinam. Note the permeability (aperture) reductions with depth.

Note the assumed (three sets) *physical* joint porosity = 3E/S. There were also reducing permeabilities and reducing aperture estimates at depth. This logic extends to the estimates of S the mean spacing of conducting joints. In other words there are likely to be more 'zero-flow' stages at increasing depth. Note that the estimated grout take, without any assumed jacking effects is as low as 0.6 litre/m^3 with S (at 40-60m depth) a mean 0.7m and E_{mean} = 0.14 mm. Typical pre-grouted tunnels in Norway suggest approx. 1 to 5 litres/m^3 grout volumes *(Barton, 2003)* which implies active joint jacking effects (and an assumed need of this for a good result).

Figure 14. Predominantly wet dark-looking shotcrete (and leaking bolt holes) may be seen when too low pressures and incorrect grout selection is made. It also will not help to delay bolting and have too narrow pillars between large pre-injected tunnel spans in the case of motorways. In the left-hand photo two light-coloured noses are seen in the arch – where shotcrete was able to dry. The wet shotcrete may be assisting groundwater drawdown along the motorway corridor.

Estimating permeability using Q_{H2O}

At the Bærum Tunnel shown in Figure 1, the first author was asked to analyse the initially controversial permeability test results obtained in four long (400m) inclined boreholes deliberately set to intersect dominant sub-vertical structure in the folded shales, nodular limestones and numerous sub-vertical igneous dykes. The assessment included extensive shallow seismic refraction measurements, and detailed Q-parameter histogram logging of all the core from the five boreholes BH1 to BH5, a total 1,460m of core. Measured Lugeon values varied from 0.1 to 394, the highest values in the numerous igneous dykes. Figure 15 illustrates the method of permeability prediction that was developed in 2006 – an empirical development made directly from these quality controlled (partly repeated) Lugeon tests.

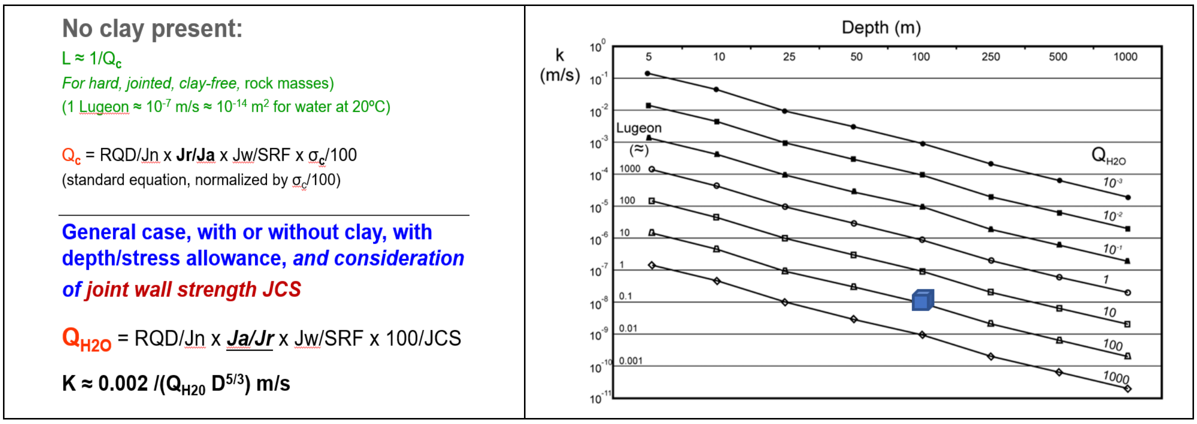


Figure 15. Two options for approximate estimation of permeability from the Q-value: either clay-free, or with clay-filled joints. Depth D in meters. Note that both RQD and Jr/Ja ('least favourable') are potentially anisotropic, as also permeability. Example: 100m depth, regular Q = 50/9 x ***1.5/4*** *x 0.66/1 = 1.4 ('poor'). Assuming weak joint walls and JCS = 10MPa, Q_{H2O} = 50/9 x* ***4/1.5*** *x 0.66/1 x 100/10 = 98. Therefore $K = 0.002/(98 \times 100^{5/3}) = 9 \times 10^{-9}$ m/s (approx. 0.1 Lugeon, see cube). (Barton, 2007).*

Conclusions

1. Good pre-injection results are possible with appropriate grouts and additives, appropriate pressures, and appropriate control routines to prevent hydraulic fracturing but *allow local joint jacking* if needed due to tight but leaking joints. Start with higher w/c ratios and reduce to build pressure. Reduce pressure when flow stops to prevent hydraulic fracturing over a wider area.
2. The typical range of grouting quantities used in pre-injected tunnels in Norway range from 1 to 5 litres/m^3 assuming that on average a 5m thick 'cylinder' of surrounding rock is injected. While inevitably a gross simplification as grout may 'escape'this suggests that a degree of joint jacking is occurring and is needed due to too low mass porosity.
3. Due to the rapid drop in pressure in the first 1 meter from each injection hole that intersects joint planes, it is imperative to use significant injection pressures in hard rock with tight joints, even up to 5 to 10MPa. Limiting pressures to 'minimum rock stresses' *(Gustafson and Stille, 1996),* or to the level of 'confining stresses' (recent SINTEF), to prevent jacking may cause environmental damage if a limited number of the joint sets become grouted.
4. Only one joint set may be injected when pressure is limited and if the apertures are small but still conduct water. This phenomenon has been seen in many tunnelling locations outside Norway.
5. When rock masses are permeable or weaker, lower injection pressures are of course sufficient.
6. Three-dimensional permeability measurements in multiple holes before and after grouting have shown both rotation and magnitude reduction of permeability tensors as each set is grouted (Quadros PhD summary in *Cruz et al. 1982,* see Eda Quadros figure in the Appendix).
7. Grouts can be chosen (ultrafine, micro, industrial) based on Lugeon testing and core logging methods as outlined in this paper. Micro-silica additives are needed to ensure stable non-shrinking grouts, and *one* injection cycle. Several injection cycles to achieve required tightness indicates incorrect technique and/or incorrect materials and has obvious consequences for tunnelling cost and time.
8. It is strongly advised never to use the filter pump to test potential grout mixes. The artificial screens disqualify the best grouts, as these have marked *extensional viscosity*. Except for flowing through sand they have no practical similarity to grout flowing in rock joints.
9. In the case of pre-grouting ahead of shallow tunnels it may be necessary to use a blocker grout that sets fast and provides confinement for the following higher-pressure pre-injection.
10. When there is a strict demand of maximum inflows in the range 1 to 3 litres/min/100m one should start with 'too many holes' (especially if large spans) and use the finest cements and micro-silica. One can then optimize, perhaps reducing the number of holes, and even use a coarser cement if the necessary results are being readily achieved and documented. All holes must be injected. Do not start with widely-spaced holes in an effort to reduce costs.
11. Install up-to-the-face shotcrete support and bolt reinforcement especially if a parallel tunnel is to come (too) close. Large motorway tunnels should be separated by a pillar width appropriate to the general rock mass quality, with conservatism when both tubes need pre-injection.
12. Groundwater drawdown is likely to be at least 'doubled' by twin-tubes especially if pillars are too narrow and bolting is delayed. Extra pre-grouting effort should be expected in each tunnel. In the case of overlying clay only the best possible results will suffice to prevent environmental damage. Remember the importance of the in-tunnel environment also.

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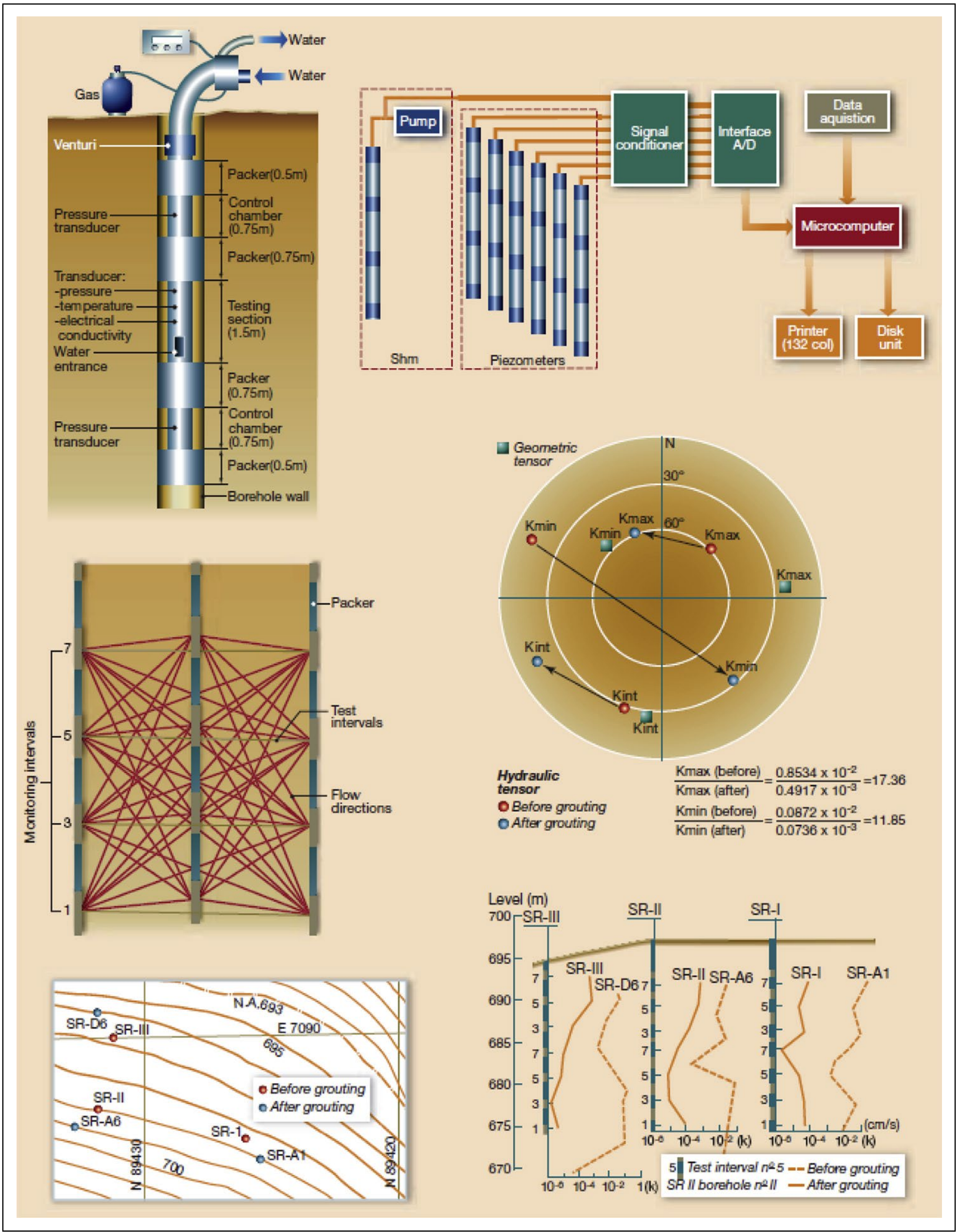
APPENDIX (see figure on last page)

Three-dimensional permeability testing performed between three boreholes, both before and after grouting, showed rotation and reduction in magnitude of the eigenvalues (principal permeability components), and greatly increased bulk modulus (Quadros and Correa Filho 1998). This unique test has helped to interpret the desirable grouting result: sealing of (almost) all the joint sets, instead of just one joint set when using too low injection pressures where such is the norm. The latter (grout apparently into only one joint set) has regrettably been seen countless times when reviewing a major Swedish tunnel, using hundreds of high-quality tunnel-face photographs.



Eda Quadros and Diogo Correa Filho, 1995