Underground pressures

N Barton and E Quadros offer an understanding of high pressure pre-grouting effects for tunnels in jointed rock

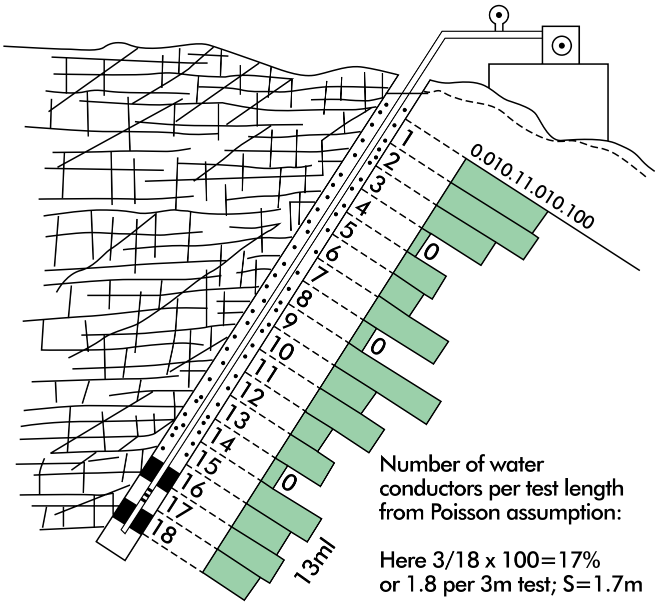


Fig 1: Lugeon testing and zero flow sections as a percentage of the total

NORWEGIAN UNLINED pressure tunnels took many years to reach heads of 1000m. 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 say 1 or 5 l/m/100m, or where tunnel stability needs improvement (and both of the above).

Do we know the actual effects of this high pressure injection on the rock mass? Can effects be quantified in any way? 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 and improved stability. The pressures used are far higher than have traditionally been used at dam sites, where in Europe, Brazil and the US, maximum grouting pressures (for deep dam foundations) have been limited to about 0.1, 0.05 and 0.023MPa/m depth respectively (Quadros and Abrahão, 2002).

According to a recent report by Klüver (2000), 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. On at least one joint set there may be local shear and dilation. Each of these effects are probably in the region of small fractions of a mm, judging by the local grout take of the rock mass, which may be about 1 to 5litres/m^3 of rock mass.

Basic elements of Snow's method

Figures 1 and 2 show how, using Snow (1968), 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. A further key simplifying assumption is that the water conductors can be roughly represented by a cubic network of parallel plates, i.e. the conductors only. There are many more joints found in cores through most rock types, due to limited connectivity.

In Figure 3 a simplifying attempt to represent 'reality', using the isotropic model of Snow (1968) is illustrated. The reality may be anisotropic and less homogeneous. It is further emphasised that in reality, stress transfer across the joint walls is required. Because of points of contact, and tortuous flow, and actual rough joint walls, the average physical aperture (E) which is potentially groutable, is usually larger than (e) the hydraulic aperture.

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, while

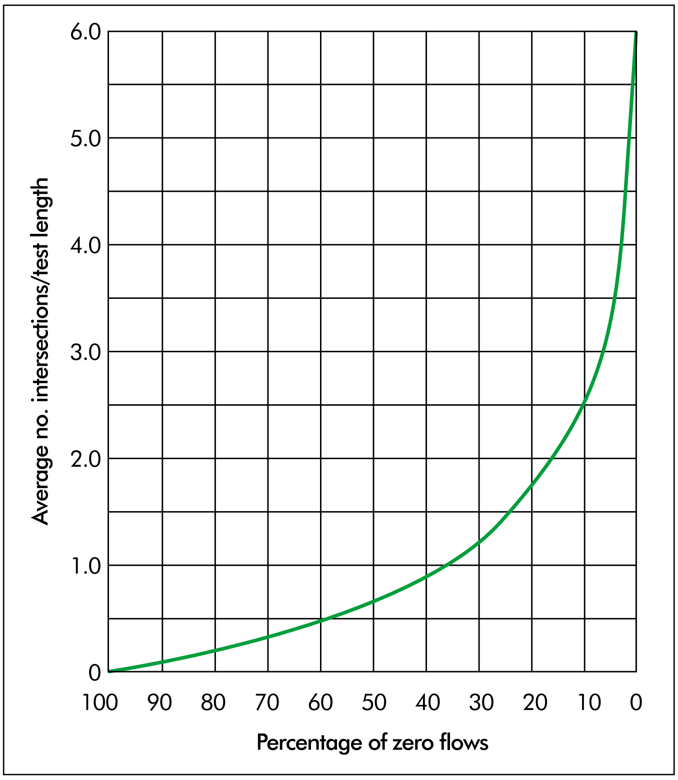
$$K_1 = {}^{e^2}\!/_{12} \times {}^{e}\!/_{S} \qquad (1)$$

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 simplifications that 1 Lugeon ≈ 10-7m/sec ≈

Fig 2: Poisson distribution for interpreting average number of water conductors (Snow 1968)



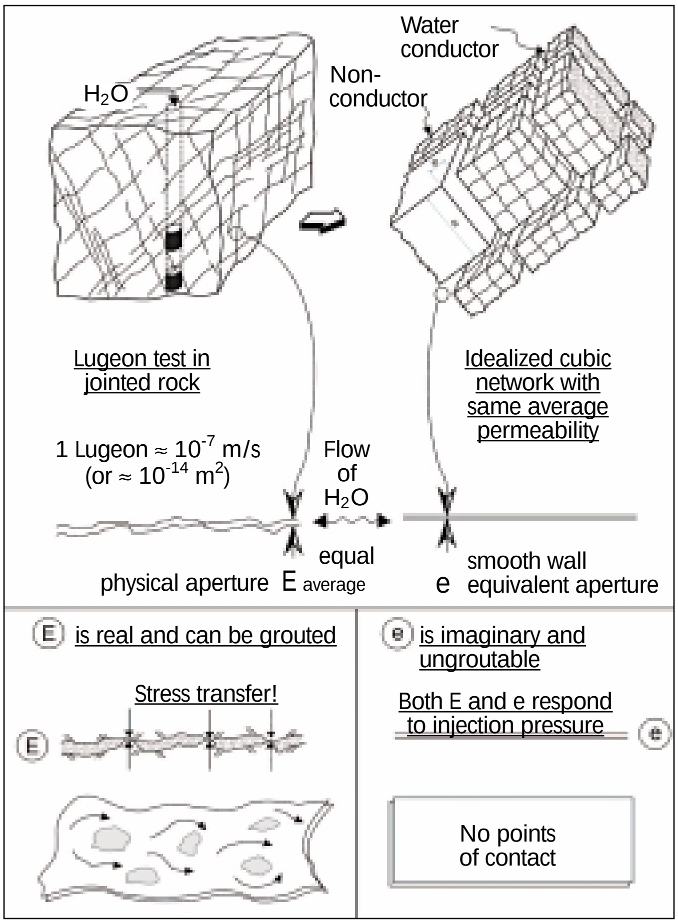


Fig 3: Cubic network model of Snow 1968 and definitions of e and E, from Barton et al, 1985 and Barton 2003

$10^{-14}m^2$, therefore 1 Lugeon ≈ $10^{-8}mm^2$, we can finally write the simplified relation

$$e \approx \sqrt{L \times 6 \times S \times 10^{-8}} \tag{3}$$

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 examples are derived, as shown in Figure 4, assuming a typical range of 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 higher pre-grouting pressures than in the Lugeon test.

Roughness and apertures

An example application of equation 3 to a set of permeability (Lugeon) tests at a permeable dam site is demonstrated in Figure 5. Here we show the potential difference between (E) and (e), based on the joint roughness dependence shown in Figure 6 and a simple rearrangement of the empirical equation:

$$E \approx \sqrt{e \times JRC_0^{2.5}} \tag{4}$$

At the bottom of Figure 5 the assumption was made (in 1985) that grouting would not further deform the joints (i.e. ΔP_g = 1MPa assumed, as in the Lugeon test where ΔP_w = 1MPa). The groutable porosity can, in principle, be written:

$$n \approx 3E/S \tag{5}$$

assuming the average cubic network and that (E) will give 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.

Joint entry by the grout particles is depicted schematically at the top of Figure 7. There is a certain logic for blocked entry (i.e. filtering) if E ≤3 x d_{max} (if there are sufficient numbers of d_{max} particles). A modified rule-of-thumb that is easier to use, as d_{95} is easier to measure, is that

$$E \leq 4 \times d_{95} \tag{6}$$

may cause blockage (a filter cake). When d_{95} = 12µm, and d_{max} = 16µm (as for a typical ultra-fine cement), these relations both suggest great difficulty when E ≈ 50µm.

The next critical question is which hydraulic aperture (e) will be approximately equivalent to E ≈ 50µm? The answer is 'many possible apertures', because of joint wall roughness JRC_0. Barton and Quadros (1997) showed that JRC_0, which is proportional to amplitude of roughness (a) divided by length of profile (Lp), is equivalent to the classic 'relative roughness' used in hydraulics. From equation 4 we see in table 1 some of the possible solutions for hydraulic apertures (e) equivalent to E = 50µm.

Table 1:
Equivalence of (e) and (E) with respect to varied joint wall roughness JRC_0 (from smooth slightly undulating to very rough and undulating)

JRC0	E (µm)	e (µm)
5	50	44.7
10	50	7.9
15	50	2.9

The value of JRC_0 can be estimated from (a/L_p) x 400 (at 100mm length scale), using profiling. A broad selection of joint roughness measurements in 1000 m of core by Barton (2002a), revealed an approximate relationship between JRC_0 and J_r (joint roughness number) from the Q-system is as follows:

$$JRC_0 \approx 7J_r - 3 \tag{7}$$

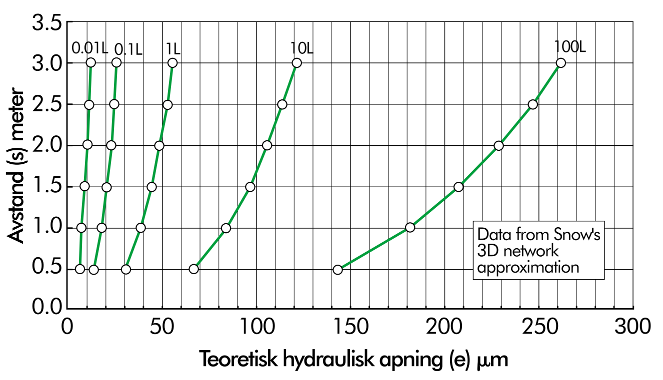
This can be used prior to more accurate profiling methods.

The above considerations suggest 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 etc.), but also whether high pressures will be needed. For example, from Table 1 and Figure 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 J_r ≈ 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 6 size limitations.

Joint aperture increases due to pressure

In Figure 9, 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 of Bandis is assumed to (almost) represent in situ conditions, following especially the first 'hysteresis-cycle', when a sampled joint is first re-loaded.

Conversion between $\sigma_n - \Delta E$ curves and $\sigma_n - \Delta e$ curves shown in Figure 9 is made with equation 4. Our Lugeon test with ΔP_w ≈ 1 MPa (max.) causes only a small Δe (and also a relatively small ΔE), while a high pressure injection ΔP_g ≈ 5 to 10MPa, will achieve a significant ΔE (say 10 to 50µm) depending on distance (R) from the injection hole. This increase may be the difference between grouting success and grouting failure, but sometimes hydraulic



0.01 Lugeon		0.1 Lugeon		1 Lugeon		10 Lugeon		100 Lugeon	
S (m)	e (μm)	S (m)	e (μm)	S (m)	e (μm)	S (m)	e (μm)	S (m)	e (μm)
0.5	6.7	0.5	14	0.5	31	0.5	67	0.5	144
1.0	8.4	1.0	18	1.0	39	1.0	84	1.0	182
1.5	9.7	1.5	21	1.5	45	1.5	97	1.5	208
2.0	10.6	2.0	23	2.0	49	2.0	106	2.0	229
2.5	11.5	2.5	25	2.5	53	2.5	114	2.5	247
3.0	12.2	3.0	26	3.0	56	3.0	122	3.0	262

Fig 4: Derivation of mean hydraulic apertures (e) and mean spaings (s) from Snow 1968 equations

'fracturing' (local loss of contact points) may be the only alternative.

In Figure 7 we depicted the different potential pressure- drops away from an injection borehole, 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, hydraulic theory suggests some 40 to 80% pressure loss in the first 1m radius (while flow is still occurring).

If, despite this fundamental fact, injection pressures are still limited and particle sizes are too large in relation to equation 6 and the available (E + ΔE) aperture, then 'water sick' rock as depicted in Figure 8 may be the result. Thin, individual 'lenses' of badly filtered grout 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 are costly examples of this from poor practice.

PRE-GROUTING RESULTS

From recent compilations of practical experiences, we can derive from Åndal et al 2001 the following quantities of grout 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 certain 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 2 that 1 to 5 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 1500 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 0.1litres/m^3, so hardly affecting the above 'rule-of-thumb' result of 1 to 5litres/m^3. Tunnels with poor grouting results may typically lie well below 1litres/m^3 in injected volume, resulting in poor connection between the grout 'lenses' and suffer possible (continued) wet conditions as a result.

We can also note that average grouted apertures (E) of 333μm at 1m intervals in three perpendicular directions (the cubic model) are suggested by 1litres/m^3 of grout. It is therefore clear that joint deformation is taking place (most likely on all conducting sets). Shear and dilation is also a likely, local mechanism, for at least one of the joint directions depicted in Figures 7 and 8.

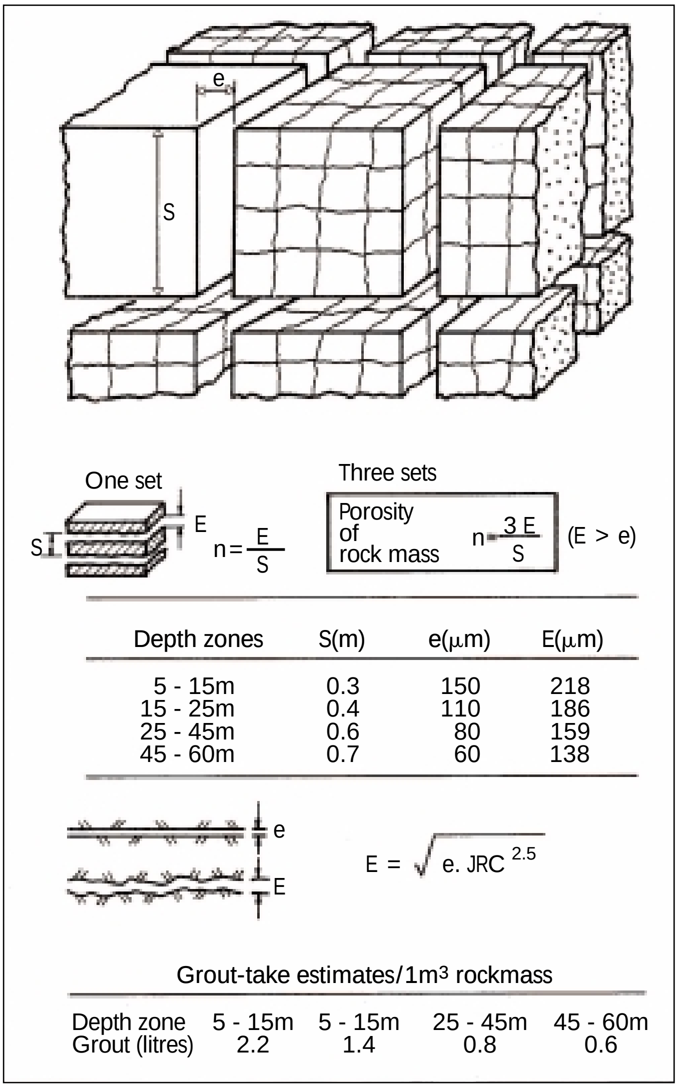
A compact summary of some unique field tests from Brazil, indicates that three-dimensional testing using multiple boreholes can help to prove what is going on in both successful and unsuccessful grouting. In these particular before-and-after-grouting water permeability tests, which were performed in a permeable dam abutment, the preliminary, conventional interpretation of individual borehole tests showed reductions of permeability from 1 to 4 orders

Table 2:
Pre-grouting data derived from Andal et al., 2001.

Rock type	kg / m^2 tunnel surface	≈kg / m^3 ‡	≈litres/m^3 ‡
gneiss	11.0 to 16.5	1.8 - 2.8	1.0 - 5.0
granite	12.0 to 52	2.0 - 8.7	1.1 - 5.0
phyllite	26	4.3	2.5
rhomb porphyry	28 to (99)	4.7 - (16.5)	2.7 - (9.4)
syenite (dike)	30 to (186)	5.0 - (31)	2.9 - (17.7)
fracture zone	19 to 50	3.0 - 8.3	1.8 - 4.7

‡ An average 'cylinder' thickness of 6m of grouted rock mass has been assumed. A grout density of 1.75 gm/cc is also assumed.

Fig 5: Application of Snow (1968) and Barton et al 1985 to a permeable dam site. In this early example, grouting pressure, was assumed equal to the Lugeon test pressure



of magnitude (i.e. from 10^{-7} to 10^{-8}m/sec, or from 10^{-5} to 10^{-7}m/sec, or from 10^{-4} to 10^{-8}m/sec).

In the three dimensional 'hydrotomography', using multiple borehole pumping tests, the three principal permeability tensors all rotated, signifying good or partial sealing of at least three sets of joints. The reductions in K_{max} and K_{min} were more than 1 order of magnitude (between the widely separated boreholes), and deformability (the bulk modulus) also reduced on average by a factor of almost 8. The lesser reduction in permeability in '3D' space compared to that in single holes, emphasises the need for thoroughness in the first (and therefore only) round of grouting, provided high injection pressures are used (e.g. 5 to 10MPa), with successive reduction in the w/c ration, from perhaps 1.0 to 0.5 following Klüver, 2000.

On the basis of all the following : measurement of P-wave veloc-

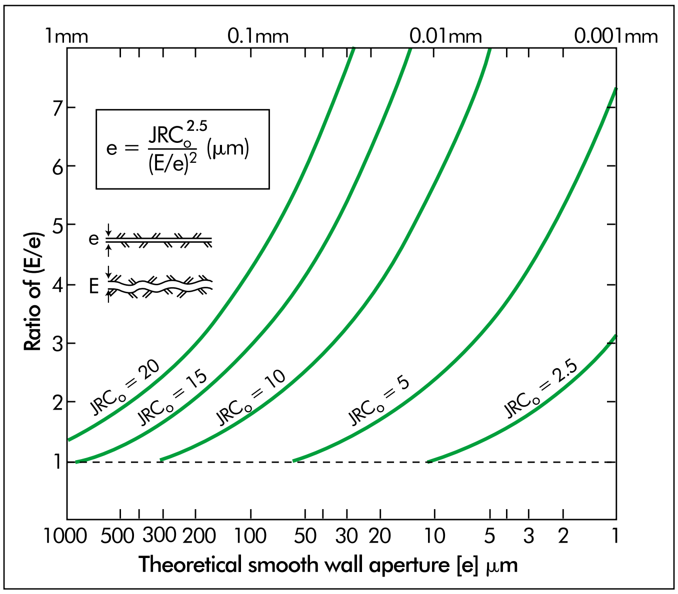


Fig 6: The inequality of (E) and (e) for mated joints under normal closure (or opening) is a function of joint roughness coefficient JRC_0 Barton et al 1985

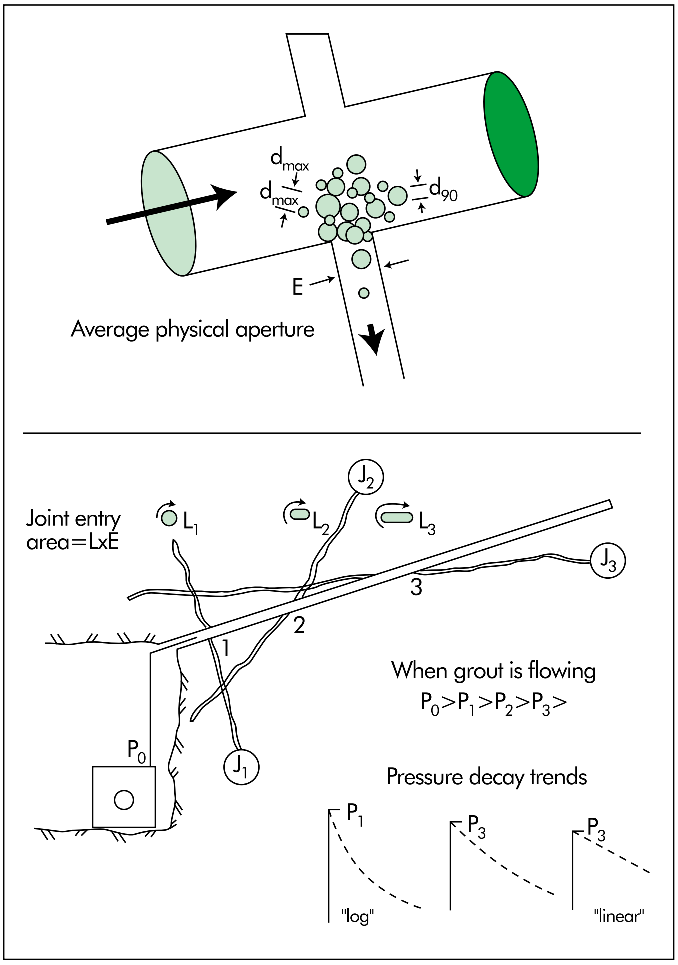
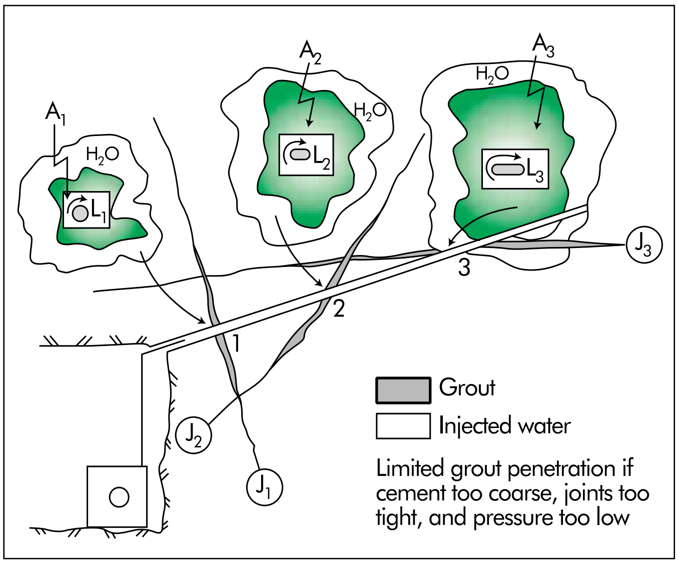
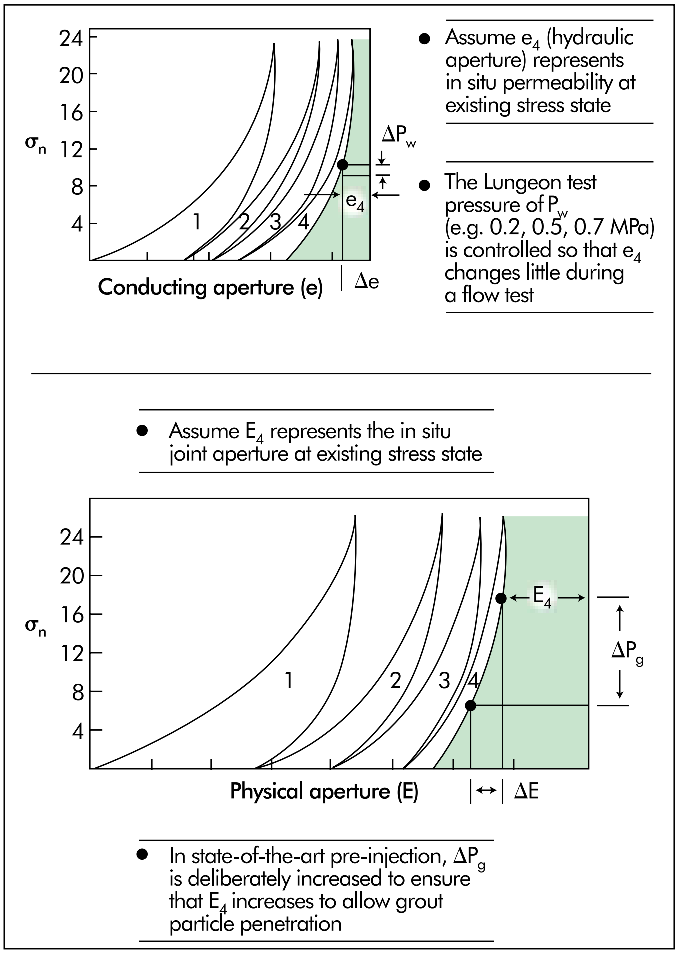


Fig 7: Sources of pressure drop and joint entry problems, Barton 2003



Above - Fig 8: 'Water sick' rock, due to too coarse cement particles, too tight joints, and too low injection pressure. Below - Fig 9: The secret of successful pre-grouting (besides grout particle technology improvements, such as the use of microsilica) is to make $\Delta P_g >> \Delta P_w$ so that $\Delta E >> \Delta e$ (Barton-Bandis joint model)



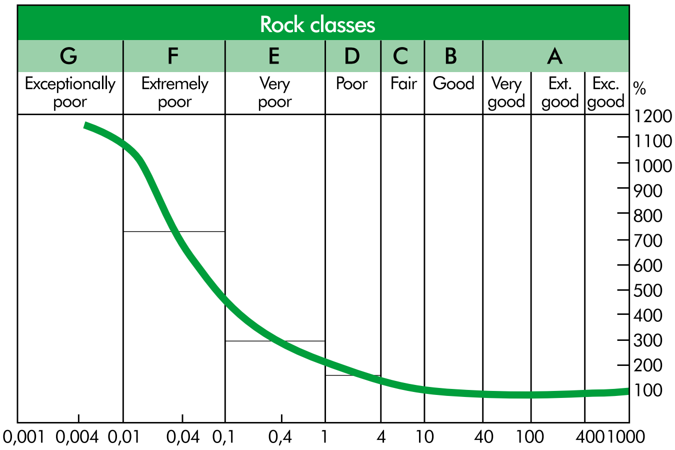


Fig 10: Relative cost in relation to Q-value, for a major rail tunnel (Barton, Buen & Roald, 2001/2002

ity increase in dam foundations, modulus increases, deformation reduction, reduced tunnel rock support requirements, and of course reduced water inflows to tunnels, it is reasonable to assume that successful pre-grouting improves various rock mass properties. In the following we will assume that Q-parameters can form the basis of a 'quantitative' understanding of the potential effects of grouting.

We will assume that in a certain rock mass, pre-grouting may cause moderate, individual effects like the following:

RQD increases e.g. 30 to 50%, J_n reduces e.g. 9 to 6, J_r increases e.g. 1 to 2 (due to sealing of most of set No. 1), J_a reduces e.g. 2 to 1 (due to sealing of most of set No. 1), J_w increases e.g. 0.5 to 1 (even with $J_W = 1$, tunnel ventilation air may contain moisture), SRF (might increase in faulted rock with little clay, or if under low stress i.e. near-surface).

Before pre-grouting $$Q = \frac{30}{9} \times \frac{1}{2} \times \frac{0.5}{1} = 0.8$$

After pre-grouting $$Q = \frac{50}{6} \times \frac{2}{1} \times \frac{1}{1} = 17$$

Such results could give significant improvements in rock mass parameters from empirical methods described by Barton, 2002b.

The potential reduction in tunnel support needs with improved effective Q-values is illustrated in Figure 10, by the reduced relative cost, and similar advantages for time of construction. (Data given by Roald for a major rail tunnel, see Barton et al, 2001/2002). A moderate shift in effective Q-value due to pre-grouting will clearly give significant cost and time savings, especially in the steeper parts of the curve, where pre-grouting may be most needed.

Of course, pre-grouting delays tunnel driving every third or fourth round, but the 24 hour 'delay' is an investment in trouble free advance for the next rounds, and water inflow restrictions at environmentally sensitive locations are usually solved in the process – by one thorough high-pressure pre-grouting cycle.

Conclusions

- High pressure pre-injection of micro-cements at 5 to 10 MPa excess pressure will generally cause local joint opening, and probably local shear and dilation on inclined joint sets. Since average grouted apertures may be as much as 0.5mm, it is clear that the Lugeon testing will 'fail' to produce realistic apertures on two counts.
- The mean hydraulic apertures (e) derived from Snow's cubic network assumptions and from the cubic law – which are useful first steps in the estimation process – will first need conversion to average physical apertures (E) using the joint roughness coefficient JRC0 (Barton et al, 1985). These apertures will vary from domain to domain, and from rock type to rock type.
- Effective-stress-reduction modelling is then required to derive estimates of the increased apertures, bearing in mind the rapid pressure decline at increased radii from the injection holes.
- In situ stress estimation for modelling undisturbed joint aperture conditions may need to account for different stiffnesses in interbedded rocks like shale and limestone (Barton 2002a).
- The Barton-Bandis model for predicting increased apertures from normal-opening or from shear-dilation (Barton et al, 1985), apparently provides realistic mean physical apertures, judging by application to recent tunnelling projects where different sized micro-cements and micro-silica were in use.
- An important step in this judgement is the comparison of ΔE (the increased physical aperture) to an 'E' ≥ 4 d95 particle size joint entry limit, which has its origin in the rule-of-thumb 'E' $\geq 3d_{max}$. These generally give similar predictions.
- Three-dimensional permeability tests ('hydrotomography') performed simultaneously between several boreholes, gives evidence of principal value (tensor) rotation, reduction and homogenization, as a result of grouting. The presumed successive sealing of different sets resembles the pressure plateaux recorded when pre-grouting, as observed by Klüver 2000.
- If several sets of joints are sealed or partly sealed, some modest improvements in many Q-parameters can be envisaged, which can potentially be used to support observations of various rock mass improvements (Barton 2002b). IWP&DC

The authors are Nick Barton, Nick Barton & associates, Oslo, Norway; and E Quadros, IPT, Sao Paulo, Brazil. This article was based on a paper presented at the ISRM 2003 congress, held from 8-12 September 2003 at the Sandton Convention Centre, Gauteng, South Africa. With thanks to the South African Institute of Mining and Metallurgy for kind permission to reproduce this paper

References

1. BARTON, N., BANDIS, S. and BAKHTAR, K. Strength, deformation and conductivity coupling of rock joints. Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 22(3), 121-140, 1985.
2. BARTON, N. and QUADROS, E. Joint aperture and roughness in the prediction of flow and groutability of rock masses. Proc. of NY Rocks '97, editor Kim, K., 'Linking Science to Rock Engineering', Int. J. Rock Mech. Min. Sci. 34(3-4),907-916, 1997.
3. BARTON, N., BUEN, B. and ROALD, S. Strengthening the case for grouting. Tunnels and Tunnelling International, 34-36, Dec. 2001, 37-39, Jan. 2002.
4. BARTON, N. Theoretical and empirical understanding of pre-injection and possibility of reduced support quantities in selected lengths of tunnel (in Norwegian). Miljø- og Samfunnstjenelige Tunneler, Norwegian State Road Authority, Report 29, 2003, 140p.
5. BARTON, N. An analysis of rock mass quality, Lugeon tests, and the effect of pre-injection with respect to sealing and stability of the tunnels between Jong and Asker (in Norwegian). Contract report for Jernbaneverket (Norwegian State Rail Authority), 2002a
6. BARTON, N. Some new Q-value correlations to assist in site characterization and tunnel design. Int. J. Rock Mech. Min. Sci., 39, 185-216, 2002.
7. KLÜVER, B. H. Rock injection. Project C, Sealing Techniques. (in Norwegian). Miljø- og Samfunnstjenelige Tunneler, Norwegian State Road Authority, Int. Rept. 2151, 2000.
8. QUADROS, E. and CORREA FILHO, D. Grouting efficiency using directional (3-D) hydraulic tests in Pirapora Dam, Brazil. Proc. of 8th ISRM Cong., Tokyo, editor Fujii, Balkema, 823-826, 1995.
9. QUADROS, E. and ABRAHÃO, R. Percolation and grouting in basaltic rocks – some Brazilian experience. Proc. EUROCK 2002, Workshop on Volcanic rocks, Madeira, Portugal, 2002.
10. SNOW, D. T. Rock fracture spacings, openings and porosities. Jour. Of Soil Mech. And dound. Div., Proc. Of ASCE, SMI, 73-91, 1968.
11. Å, T., ANDERSSON, H. and AASEN, O. Injection experiences from selected tunnel projects. (in Norwegian). Miljø- og Samfunnstjenelige Tunneler, Report No. 2, Norwegian State Road Authority, 2001.