The why's and how's of high pressure grouting - Part 1

Nick Barton, of Nick Barton & Associates, explains the theory behind high pressure pre-injection of stable particulate grouts ahead of tunnel faces in jointed, water-bearing rock. Although opinions differ about the need for high pressure, its effectiveness has been proved in numerous projects. Conversely, the use of insufficient pressure has often led to wet and less stable tunnels

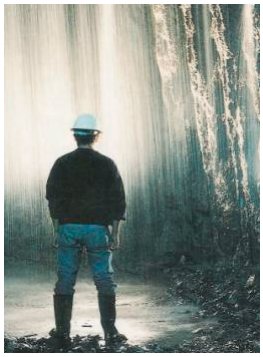
A recent consultant's article (Pells, *T&TI* March, p34[10]) and an experienced contractor's reactions (Garshol, *T&TI* May, p36[8]) has again put focus on the results obtained by pre-grouting ahead of tunnels. Garshol emphasised the need to use high pressures to get an acceptable result. The proviso is of course that the work is done ahead of the face (as in most of Garshol's excellent grouting case records) and not from behind the face where lower pressures have to be used (as in most of Pell's case records).

With a long-standing rule for injection pressure gradients of 0.23 bars/m depth for dam foundation grouting in the US, but usually higher elsewhere, it is clear that there will be reactions when 50 to 100 bars is recommended by an experienced contractor for a stretch of tunnel whose 20m depth suggests only 5 bars.

The reasons for performing high pressure (50 to 100 bars) injection when pre-grouting ahead of tunnels is that inflows have to be controlled, perhaps down to 1 to 2lt/min/100m. Permeabilities lower than 10^{-8}m/s or lower than 0.1 Lugeon are implied – and these are also achieved when owners and consultants become aware of what is achievable today, with stable ultrafine and microcements and the vitally necessary additives like microsilica and plasticizers.

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[12]).

According to a recent report by Klüver (2000[9]), a shallow tunnel in phyllite with 5m of cover, with



Drilling holes for successful high-pressure pre-injection at the Jong-Asker rail tunnel west of Oslo
Left: What can happen without these measures!

severe environmental requirements for low inflow, was injected at invert level to a final pressure of 6.5MPa and to 5MPa 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.

Measurements in Norway have also shown that there can be a pressure drop of 1MPa up to the injection point for these low w/c ratio, frictional-and-cohesive fluids (Åndal et al, 2001[14]). So with at least 40% to 80% pressure drop in the first 1m radius from an injection hole, even for Newtonian fluids like water, it is clear that at the very minimum, an excess pressure of 25 bars may be needed to inject significant volumes of grout into connected joint planes and flow channels.

On at least one joint set there may be local shear and dilation. Each of these effects is probably in the region of small fractions of a mm, judging by the average grout take of numerous rock masses. About 1 to 5lt/m^3 of rock mass is typical, if we assume that a 6m thick cylinder of rock around the tunnel has been thoroughly injected (Barton, 2003[ref?]).

Some pre-grouting results

From recent compilations of practical experiences, we can derive from Åndal et al, 2001[14] the following quantities of grout used in successful, high pressure pre-injection.

Table 1: Pre-grouting data derived from Åndal 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	1.0-1.6
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.-8.3	1.8-4.7

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

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 4m to 5m length is the logic behind an average choice of a 6m cylinder. We can see from Table 1 that 1-5lt of grout per m^3 of rock mass is a typical range, for projects where post-grouting water leakages were mostly in the desired range of 1 to 4lt/min/100m of tunnel. Tunnel cross-sections were mostly 65m^2 to 95m^2.

Note that an average pre-grouting screen of 25m length, with 30 holes of 50mm diameter will require at least 1,500lt of grout just to fill the holes.

When distributed through an assumed 6m thick cylindrical volume of 25m length, this nevertheless represents only about 0.1lt/m^3, so hardly affecting the above 'rule-of-thumb' result of between 1 and 5lt/m^3. Tunnels with poor grouting results may typically lie a long way below 1lt/m^3 in injected volume, resulting in poor connection between the grout 'lenses' and possible (continued) wet conditions as a result.

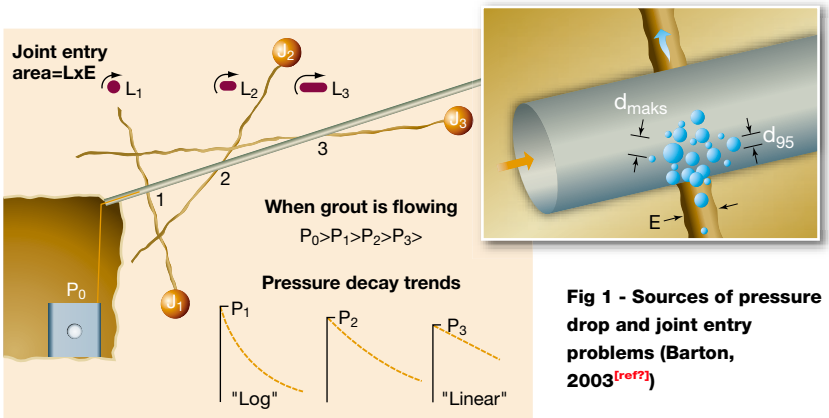


Fig 1 - Sources of pressure drop and joint entry problems (Barton, 2003[ref?])

A simplified model for visualising grout penetration

Because the author and readers of this magazine are engineers, a practical, calculable model is required for first pass estimates of grouting needs. For this reason a simple, classical model from Snow (1968[13]) has been applied, with two important additions, namely the differentiation of hydraulic (e) and physical (E) average apertures, and the recognition that deformation (local joint opening) can occur. Figure 2 shows the cubic network concept and the E > e concept.

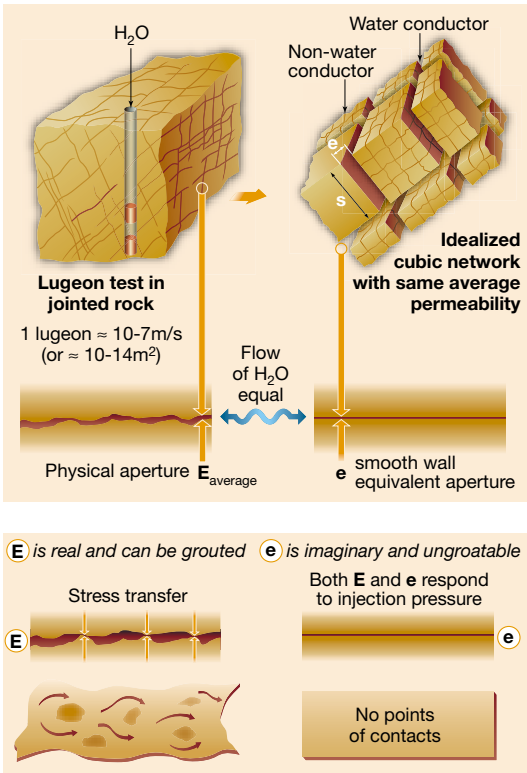
Assuming the cubic law is sufficiently valid for engineering purposes that we can ignore non-linear or turbulent flow, we can write permeability K = e^2/12 for one parallel plate, while

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

for one set of parallel plates of mean spacing (S). Snow (1968[13]) further assumed that the 'rock mass permeability' would be constituted, on average, by flow along two of the three sets of parallel plates. Thus:

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

Making further simplifications that 1 Lugeon ≈ 10^{-7}



Left and below left: Fig 2 - Cubic network model of Snow (1968[14]), and definitions of e and E, from Barton et al (1985[1])

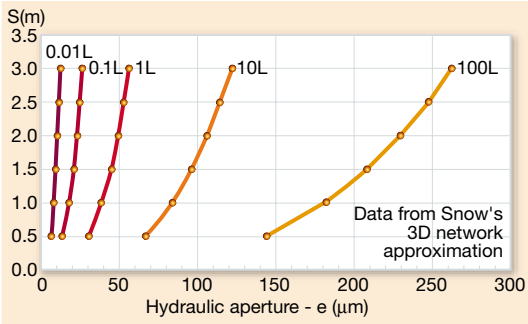
Part 2 of 'The why's and how's of high pressure grouting' will appear in next month's T&TI and will feature the complete reference table.

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

$$e \approx \sqrt[3]{L \times 6 \times S \times 10^{-8}} \quad (3)$$

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 of (e) and (S) are derived. These are shown in Figure 3, assuming a typical range of (average) S = 0.5m-3.0m. 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. We will return to both these important aspects in a moment.



Right: Fig 3 - Derivation of mean hydraulic apertures (e) and mean spacings (S) from Snow (1968[14]) equations

Below: 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(_m)	e(_m)
5	50	44.7
10	50	7.9
15	50	2.9

Roughness, apertures and particle sizes

The potential difference between (E) and (e) has been shown to be dependent on the joint roughness, as shown in Figure 4 and a simple rearrangement of the empirical equation:

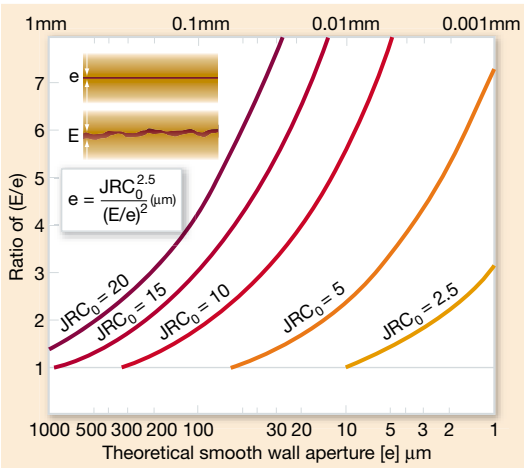
$$E \approx \sqrt{e \times JRC_0^{2.5}} \quad (4)$$

The groutable porosity for three assumed sets of joints in Figure 2 can, in principle, be written:

$$n \approx \frac{3E}{S} \quad (5)$$

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 average grouted apertures (E) of 333µm at 1m intervals in three perpendicular directions (the cubic model) are suggested by 1.0lt/m^3 of grout. It is therefore clear that joint deformation is taking place (most likely on most of the water conducting sets). Shear and dilation is also a likely, local mechanism, for at least one of the inclined joint directions depicted in Figure 1.



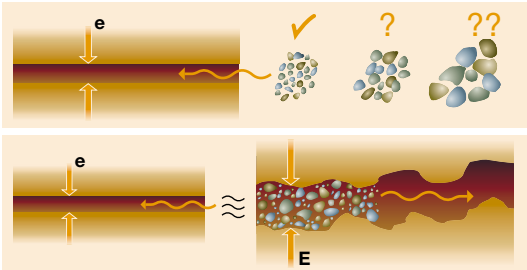
Right: Fig 4 - The inequality of (E) and (e) for mated joints under normal closure (or opening) is a function of joint roughness coefficient JRC_0. (Barton et al, 1985[1]). The hydraulic aperture (e) is not however smaller than the physical aperture (E ≈ 2m)

The value of JRC_0 can be estimated from (a/L) x 400 (at 100mm length scale), using profiling. A broad selection of joint roughness measurements in 1000m of core by Barton (2002[4]), revealed an approximate relationship between JRC_0 and J_r ('joint roughness number') from the Q-system. This can be used prior to more accurate profiling methods.

$$JRC_0 \approx 7J_r - 3 \quad (6)$$

Which hydraulic aperture (e) will be approximately equivalent to, for instance E ≈ 50 ?m. The answer is 'many possible apertures', because of joint wall roughness JRC_0. Barton and Quadros (1997[2]) 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 ?m.

Joint entry by the grout particles is depicted schematically in Figure 5. Firstly, a micro cement with d_{95} = 30 microns may well penetrate a joint with e = 25 microns – it is a question of roughness,



Far right: Fig 5 - The inequality of (e) and (E) and the E > 4 d_{95} entry limit, due to particles 'delayed' on the rough walls of the joint. A higher w/c ratio allows this rule to be broken, but gives an incompletely blocked end product, unless w/c is reduced at the end of grouting, and applied under high pressure

because E may be >>25 microns. Secondly, there is a certain logic (boundary layer theory) and experimental evidence (Bahsin et al, 2002[7]), 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 for joint entry limits that is easier to use, as d_{95} is easier to measure, is that:

$$E \leq 4 \times d_{95} \quad (7)$$

When for instance, d_{95} = 12 ?m, and d_{max} = 16µm (as for a typical ultra-fine cement), these relations both suggest great difficulty when E ≈ 50 ?m. However a very high water/cement+filler ratio can 'over-rule' here, just as a 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.

The above suggests 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 Figures 3 and 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 7 size limitations. T&T