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The theory behind high pressure grouting Pt 2

Here Nick Barton, of Nick Barton & Associates, concludes his two part article explaining the theory behind high pressure pre-injection of stable particulate grouts ahead of tunnel faces in jointed, water-bearing rock

Effects of high pressure on joint deformation

In Figure 7, the most fundamental aspect of successful pre-grouting, using elevated grout pressures such as 5MPa to 10MPa, can be demonstrated by means of the Barton-Bandis normal closure/opening model. The experimental 4th load-unload cycle of the Bandis part of the model 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 7 is made with equation 4. In a Lugeon test with $\Delta P_w \approx$ 1MPa (max), only a small Δe (and also a relatively small ΔE) is experienced. In contrast, a high pressure injection with $\Delta P_g \approx$ 5MPa 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 success and failure for initial joint entry, but sometimes (often?) hydraulic 'fracturing' (local loss of contact points) may be the only alternative.

If injection pressures are limited and particle sizes are too large in relation to equation 7 and the available (E + ΔE) aperture, then 'water sick' rock 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 examples of this where designers have failed to recognise the importance of using higher pressure.

Three dimensional effects

Some unique 3D field tests using multiple boreholes, reported from Brazil (Quadros et al.1995[11]), indicate what may be 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 of magnitude (i.e. from 10^{-7}m/s to 10^{-8}m/s, or from 10^{-5}m/s to 10^{-7}m/s, or from 10^{-4}m/s to 10^{-8}m/s).

In a 3D sense, the three principal permeability tensors all rotated (Figure 8), 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.

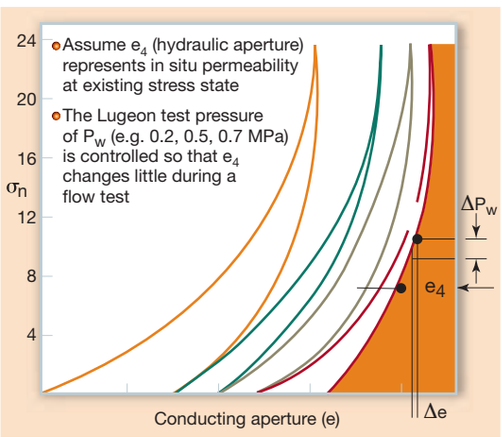
Improvements due to high pressure pre-injection?

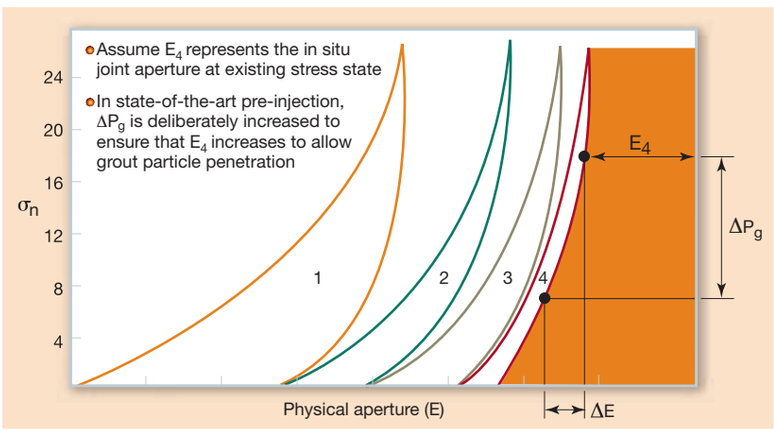
In *T&TI* recently (June 2004, p14-16), Moen summarised experiences from the 6.3km Jong-Asker, 114m^2 rail tunnel, where systematic pre-grouting was used throughout. He states that stability problems in shales and schists proved to be almost non-existent, and that rock quality had definitely improved due to the grouting. It seems reasonable to assume that successful pre-grouting improves various rock mass properties, because measurements of P-wave velocity show increase during grouting of dam foundations, reduced deformation is measured in tunnels, there are reduced tunnel rock support requirements, and of course reduced water inflows. Garshol 2004 suggested from 10^{-2} to 10^{-3} improvement in permeability in 'highly jointed rock masses with predominantly very fine fissures', and from 10^{-5} to 10^{-6} improvement for 'widely spaced and very open large joints'.

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 $Q = \frac{50}{6} \times \frac{2}{1} \times \frac{1}{1} = 17$.





Bottom left and right: Fig 7 - An illustration of grouting pressure effects on joint aperture changes. Δe during a Lugeon test is supposed to be small, while ΔE during high pressure grouting is supposed to be large. But this is only locally around each hole due to logarithmic-to-linear pressure decay

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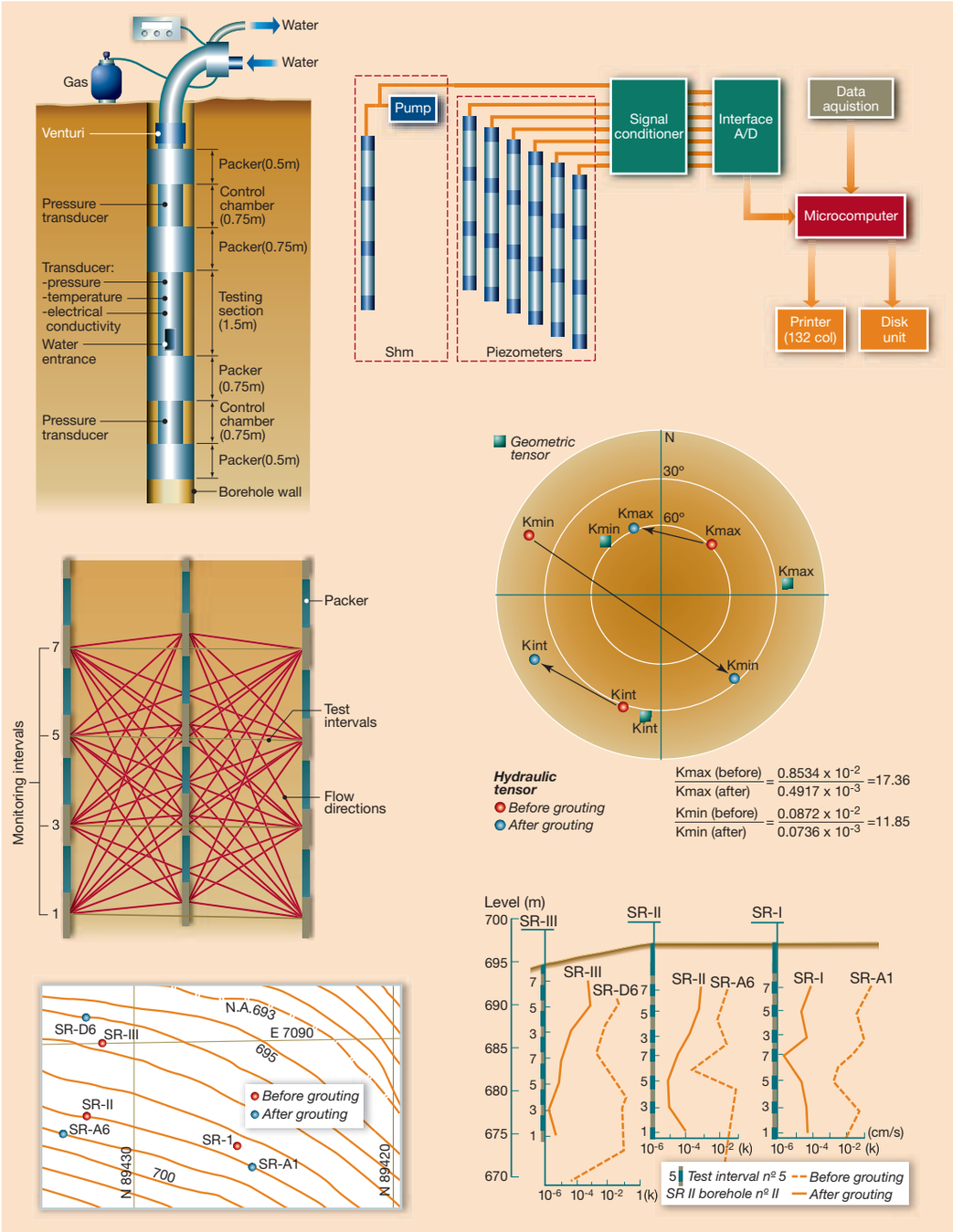

Above: Fig 8 - Before and after grouting 3D permeability testing, showing rotation and reduction of permeability tensors. Quadros et al., 1995.

Even with such conservative assumptions for individual Q-parameter improvements, the predicted rock mass property improvements are impressive. Table 3 results are based on empirical methods described by Barton, 2002b[5], and at this stage they do not include specific grouting effects, which need testing.

The potential reduction in tunnel support needs with improved effective Q-values is illustrated in Figure 9. The reduced relative tunnel cost shown here, and similar advantages for time of construction, demonstrate that 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 (data given by Roald, see Barton et al., 2001/2002[3]).

Of course, pre-grouting apparently delays tunnel driving every fourth round or so, but the 20 to 24 hour 'delay' is an investment in trouble free advance for the next rounds, and water inflow restrictions at environ-

Right: Table 3 - An example of improvements achievable by pre-injection with fine, cementitious multi-grouts. (See Barton 2000b)

Before pre-grouting	After pre-grouting
Q = 0.8 (very poor)	Q = 16.7 (good)
Q_c = 0.4	Q_c = 8.3
V_p = 3.1 km/s	V_p = 4.4 km/s
E_{mass} = 7 GPa	E_{mass} = 20 GPa
$Sigma_{cm}$ = 9 MPa	$Sigma_{cm}$ = 25 MPa
P_r = 13.6 t/m^2	P_r = 4.9 t/m2
L = 2.5 Lugeon	L = 0.1 Lugeon
K = 2.5 x 10^{-7} m/s	K = 10^{-8} m/s
Δ = 25 mm	Δ = 1 mm
FC = 14°	FC = 63°
CC = 1.7 MPa	CC = 8.3 MPa
B 1.6m c/c	B 2.4m c/c
S(fr) 10 cm	None

Note: Q_c = Qx_c/100, and_c assumed equal to 50 MPa

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mentally sensitive locations are usually solved in the process – by one thorough pre-grouting cycle, as for example, described by Moen, 2004.

Conclusions

1. High pressure pre-injection of micro-cements at 5MPa to 10MPa 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.

2. The hydraulic apertures 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 JRC_0. These apertures will vary from domain to domain, and from rock type to rock type.

3. 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.

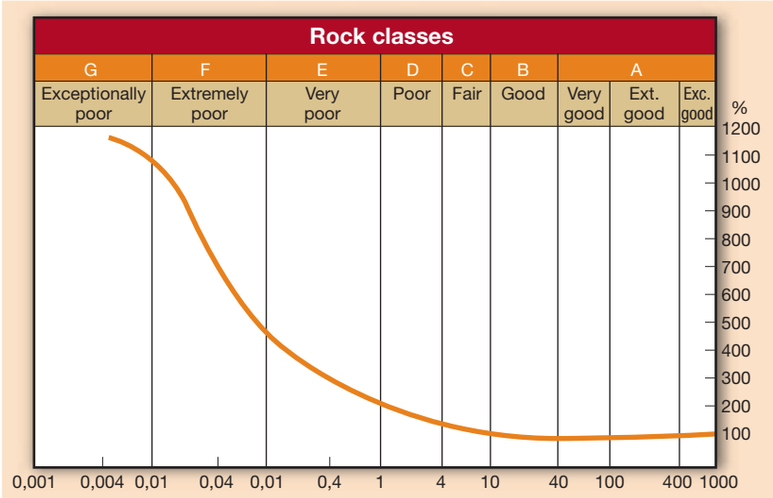
4. In situ stress estimation for modelling undisturbed joint aperture conditions may need to account for different stiffnesses in interbedded rocks like shale and limestone.

5. The Barton-Bandis model for predicting increased apertures from normal-opening or from shear-dilation, apparently provides realistic mean physical apertures, judging by application to recent tunnelling projects where different sized micro-cements and micro-silica were in use.

6. An important step in this judgement is the comparison of E+?E (the increased physical aperture) to an 'E' ≥4 d_{95} particle size joint entry limit, which has its origin in the rule-of-thumb 'E' ≥ $3d_{max}$. These give similar predictions.

7. 3D permeability tests performed simultaneously between several boreholes, gives evidence of principal value (tensor) rotation, reduction and homogenisation, as a result of grouting. The presumed successive sealing of different sets resembles the pressure plateaux recorded when pre-grouting, as observed by Klüver.

8. 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. T&T



Above: Fig 9 - Relative cost in relation to Q-value, for a major rail tunnel. Barton, Buen and Roald, 2001/2002