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STIFTELSEN SVENSK BERGTEKNISK FORSKNING
SWEDISH ROCK ENGINEERING RESEARCH
IN CO-OPERATION WITH
CHALMERS AND KTH, THE ROYAL INSTITUTE OF TECHNOLOGY

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VETENSKAP OCH KONST

4TH NORDIC ROCK GROUTING SYMPOSIUM

Stockholm 2001
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SveBeFo Rapport 55

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PREFACE

Sealing of tunnels and caverns is an important, time-consuming and costly task when going underground. The increasing concern for the environment is leading to tighter restrictions regarding acceptable leakage, and the need for better sealing techniques is obvious. Claims and technical failures manifest the complexity of the problem. To further benefit from the advantages of putting essential functions underground, the technology for grouting has to be advanced.

Research cannot on its own give the solutions on how to get the rock watertight. Research can present tools and understanding. The real advances, however, will be achieved when results are put into practise in the field.

By virtue of its long-term commitment to the issue, SveBeFo is now hosting the fourth Nordic Rock Grouting Symposium, which is held in Stockholm. Earlier meetings have been held in Gothenburg in 1992, Trondheim in 1995 and Helsinki in 1998. The aim of this fourth symposium is to provide an opportunity for researchers, those with experience from grouting contracts and others otherwise engaged in grouting, to meet and exchange knowledge and experience.

The planning and programme was made in collaboration with the Department of Geology at Chalmers University of Technology and the Division of Soil and Rock Mechanics at the Royal Institute of Technology, KTH. Papers were contributed from researchers, manufacturers and practitioners in Finland, Norway and Sweden. We wish to thank everybody for their time and effort.
It is our hope that this symposium will contribute to a greater and more widespread knowledge and better practise in the field of rock grouting. We also hope that those, who could not attend the meeting in Stockholm, will find this documentation useful.

Ann Emmelin
SveBeFo, Swedish Rock Engineering Research

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Håkan Stille,
Division of Soil and Rock Mechanics, Department of Building Science, KTH, Stockholm

Introduction

Grouting is the way for decreasing the ingress of water into an underground opening, that has been used for decades. The knowledge has mainly been based on practical know-how and a theoretical understanding has been lacking. This was not a problem until the cost for sealing the tunnels became a significant part of the total construction cost. The reason for this cost increase is that higher demands for allowable ingress of water have increasingly been set for environmental reasons and risk of damage to buildings when constructing in urban areas. Acceptable ingress of water to a tunnel in urban areas today is around 1-4 l/min and 100 m of tunnel. Corresponding figures 10 years ago were around 2-8 l/min and 100 m. The problems to reach acceptable longevity and environmental impact with chemical grouting has given that in Sweden today only cement based grouting is accepted.

The higher demands and the increasing cost for grouting have opened up for a discussion regarding alternatives to grouting, like concrete lining with or without a sealing membrane. The cost for concrete-lined tunnels is, however, so high that quite an extensive grouting can be carried out and still be cheaper than a concrete lined tunnel, as long as the sealing demands can be fulfilled. This implies that there is an incentive for developing the grouting technology by further research work.

Demands on a grouting operation

The sealing effect describes the amount of water that has to be reduced expressed as the percentage of ingress of water without grouting. The sealing effect is one way to express the demands that have to be put on a grouting operation. The sealing effect may be interpreted as the minimum portion of a tunnel length that has to be sealed by grouting. The sealing effect can be estimated by using the existing equations for calculating the ingress of water to tunnels.

For the normal variation of the rock mass permeability that we have in our Swedish bedrock, the sealing effect will be around 90 to 99 % for tunnels situated very shallow, down to a depth of around 150 m. The corresponding figures for tunnels that were constructed 10 years ago were 50 to 90 %.

The conclusion is obvious, today we have to seal the whole tunnel length by pregrouting. In order to keep the cost at an acceptable level, demands have to be raised on more efficient grouting operations.

Another very important factor is to which degree we have to seal our tunnels. This can be expressed as the demands we must put on the permeability of the grouted zone. We know from analysis that the extension of the grouted zone has less importance as long as it is thick enough not to be damaged by the blasting operation or the redistribution of stresses around the opening. The zone should have an extension corresponding to the radius of the tunnel or at least around 5 m.

The required permeability of the grouted zone around the opening can be estimated by using existing equations for ingress of water to a tunnel with a less permeable zone close to the tunnel surface. Normally it is found that the major part of the headlosses would occur in the grouted zone.

The rule of thumbs regarding the achievable permeability of the grouted zone is around 1 Lugeon or 0.3 to $3 \cdot 10^{-7}$ m/s for ordinary cement grout of today. To achieve the lowest value special types of cement has to be used like Cementas grouting cement. This is acceptable for very shallow tunnels with a low-pressure head and when the allowable ingress of water is higher then around 2 l/min and 100 m. For all other occasions like deeper tunnels or at very low demands for shallow tunnels, much higher demands must be put on the grouted zone. At least 10 times lower value on the permeability of the grouted zone is required corresponding to $0.5 \cdot 10^{-8}$ m/s. It is, therefore, obvious that especial attention has to be focused on the penetrability of the cement suspension into very fine joint apertures.

The experiences we have from our tunnels, like the Hallandsås tunnel, is that the first pregrouting operation may reduce the inflow corresponding to a permeability around 10^{-7} m/s. The first regrouting can further reduce the permeability to around $0.5 \cdot 10^{-7}$ m/s. Further regrouting operations will only marginally reduce the permeability further down to around $0.3 \cdot 10^{-7}$ m/s. This will be very time-consuming and costly since it will reduce the advance rate of tunnelling itself.

An alternative that is also very costly is to carry out postgrouting. In special cases this has been successful but in general the experiences is poor.

The objectives for the future grouting operations are the following:

- Pregrouting the total tunnel length with a mean value of 1.5 fans per grouting situation and at each situation being able to grout at least 14 m ahead of the face so that at least two rounds may be excavated before a new grouting operation has to be carried out

- A pregrouting fan should not take longer time than one shift.

- For the pregrouting operation a technique and a grout shall be used that give at least a permeability of $0.5 \cdot 10^{-8}$ m/s, preferably $0.1 \cdot 10^{-8}$ m/s.

If this would be achievable the cost for a grouting operation would still be competitive with a completely concrete lined tunnel.

It is quite obvious that to reach these goals we must be better on everything, like grouting technology, penetrability of the grout mix and interpretation of the waterways in the rock mass.

One important question is to develop better management systems for estimation, risk sharing, payment and control of the grouting operation.

Characterisation of the rock mass

The key to a successful grouting operation is good understanding of the rock mass that has to be sealed. The comprehensive research work that has been carried out the last decade regarding the waterways of the rock mass has increased our understanding of joint apertures and channel structure of the rock mass.

We can, today, estimate the mean aperture of different joint systems based on water loss measurements. We also know that this is not representative for the channel structures taking the grout. We must know much more of the channel structures like the variation of the openings of the channels and the interconnection between different joint planes. One other important factor is the geometry of the closed part of a joint plane where the stresses are transferred and will therefore be impervious.

These factors are important not only for the penetrability of the grout mix but also for understanding which drilling pattern would be most effective, with other words, we must have quite a high probability to hit the waterbearing joints in order to seal them. The high demands require that almost all joints would be hit and sealed.

Grout-mix and penetrability

The development of special grouting cement like "Cementas Injektering 30" (grain size 0.030 mm) has increased the penetrability of that channel openings from around 0.2 mm down to around 0.1 mm may be sealed. The experience of the penetrability of micro cement with grain size below 0.016 mm has, however, not shown up to have much better penetrability. Investigations indicate that the penetrability of micro cement decreases very rapidly with time after the mixing and, therefore, the expected better penetrability, which can be observed immediately after the mixing can be omitted. Better super plasticizers may be one solution to increase the "open keeping" time.

Mixing-times shorter than about 5 minutes can give an unacceptable grout-mix with uneven quality. It has also been found that the old colloid mixer will, especially if it is worn, give a lower penetrability. Other types of mixers may have better effect.

It looks as if many of the micro cements are very sensitive to the temperature of the grout mix. Higher temperature will give a faster setting and of course a poorer penetrability.

The methods existing today are all indirect types of penetrability measurement. There is a lack of knowledge regarding the real filtration mechanism in the joint during the grouting operation and therefore it is not obvious which testing equipment reflects the filtration process best like the NES equipment, sand column tests or the modified filtration pump.

It is believed that grout-mix penetrability is one of the most important questions to be solved in order to meet the aim of the future grouting operation.

Grouting technology

The main changes regarding grouting technology in the last 10 years are connected to better registration and controlling of the grouting process. The new computer technology has implied that the flow and pressure of the grout-mix can be continuously registrated. The basic equipment is the same. Above I have shown that the type and quality of the mixer has an impact on the penetrability of the grout-mix. There is room for improvement, both regarding the mixer but also the pumps.

There exist several different criteria of refusal, all of them based on experiences and very little on theoretical considerations. Our investigations at the KTH indicate that the refusal criteria must be studied and developed further. The improvement that has taken place regarding our theoretical understanding of the spreading of grout can be used to develop better rules. Modern super plasticizers will normally give a low yield value of the grout-mix even for quite a low w/c ratio. High grouting pressure can then give a very long penetration. In a rock mass of better quality and with low porosity it will then take a long time before maximum volume criteria or the refusal pressure will be obtained. The flow will decrease with time and the take will be small. If high grouting pressure should be used, it must be coupled to limitation in grout volume.

The experiences regarding the effect of leaking packers is that even for very small ingress of water to a borehole, fingering will take place before the grout starts to harden and a leaking borehole will be the result. Good packers and faster setting of the grout-mix will reduce the problem.

We believe that in the future a grouting procedure like for rock support based on theoretical consideration and in advance well-defined classes for the grouting work will be used.

Contractual aspect

The difficulty to achieve a fair risk sharing between the client and the contractor is evident. The claims are part of the daily situation. The system to being paid only on the quantities has large drawbacks. It is time that costs. The Norwegian system of being paid for the working time on the critical line can be one way to facilitate for a better risk sharing. This issue has to be studied further.

A databank based on a theoretical consideration has been established regarding the grout take and its relationship to the rock mass quality and the rock mass permeability as well as the grouting technology. The relationship is less uncertain than the other existing relationships like the grout take as a function of waterlosses. This databank can be used as a complement to existing experiences in order to estimate the quantities for grouting.

An active design approach, based on the gained experiences from the ongoing excavation, will give a possibility for improvement of the grouting operation.

CONCEPTUAL MODELS OF CONDUCTIVE STRUCTURES IN CRYSTALLINE ROCK - EXAMPLES FROM SOLUTE TRANSPORT EXPERIMENTS AT THE ÄSPÖ HARD ROCK LABORATORY, SWEDEN

Beskrivande modeller av vattenförande sprickor i kristallint berg – exempel från transportexperiment utförda vid Äspölaboratoriet

by

Anders Winberg, Conterra AB, Partille
Jan Hermanson, Golder Associates AB, Stockholm
Eva-Lena Tullborg, Terralogica AB, Gråbo

Abstract

One key component identified for understanding solute transport in fractured rock is the internal structure of investigated conductive fractures and structures. Conceptual model development for conductive fractures, infillings and immediate surroundings at Äspö was concentrated to Feature A, and the structures of the network involved in the TRUE Block Scale tracer experiments. Most conductive features at Äspö follow mylonitic brittle precursors. Fault breccia has been recovered from some structures in the TRUE Block Scale rock volume. An *in situ* epoxy resin injection pre-test, performed in a system of low-transmissive fractures revealed mean apertures in the order of 266 and 239 μm, associated with two smaller fractures with distinctive extension and shear characteristics, respectively. Remaining uncertainties relevant to understanding of solute transport included the geometry (pore space) of the actual *in situ* transport paths. Additional unknown entities were the *in situ* distribution of fault breccia, its fraction of fine-grained fault gouge infilling, and the latter's *in situ* porosity. It was identified that the detailed conceptual models produced from characterisation for solute transport experiments could help improve design, selection of grout recipes and grouting methodologies for similar geological environments.

Sammanfattning

En viktig komponent för att öka förståelsen av transport av lösta ämnen i sprickigt berg är beskrivningen av den detaljerade uppbygganden av konduktiva strukturer. Framtagandet av beskrivande (konceptuella) modeller av konduktiva sprickor, deras sprickfyllnadsmaterial och närmaste omgivning har koncentrerats till Feature A samt det nätverk av strukturer som undersökts i transportförsök inom ramen för TRUE Block Scale. Merparten av de konduktiva strukturerna på Äspö är associerade med myloniter. Sprickfyllnad i form av "fault breccia" har erhållits från vissa struktuer i TRUE Block

Scale. Ett *in situ* försök där epoxy injicerades i ett system av låg-konduktiva sprickor påvisade medelsprickvidder på 266 och 239 μm, relaterade till sprickor med extensions- respektive skjuvkaraktär. Kvarvarande osäkerheter vad gäller beskrivning av transport i sprickigt berg inkluderar geometrin (porvolymen) hos de faktiska flödesvägarna i spricksystemet. Till detta kommer osäkerheter kopplade till fördelningen av sprickfyllnad (fault breccia), dess andel av finfraktion, och den senares porositet *in situ*. De detaljerade beskrivande modeller som tas fram för beskrivning av transport i sprickigt berg kan användas för att förbättra planering, val av injekteringsämnen och injekteringsmetodik i likartade geologiska miljöer.

1. Introduction

A program for increasing the understanding of solute transport in fractured crystalline rock, the so-called Tracer Retention Understanding Experiments (TRUE) has been under way at the Äspö Hard Rock Laboratory (Äspö HRL) since 1994. The principle objective of the experiments is to investigate the relative role of radionuclide retention processes (sorption and diffusion) in crystalline rock, and the dominant immobile zone porosity (altered wall rock, gouge infillings and stagnant zones). Further, to investigate whether available model concepts for reactive solute transport are based on realistic descriptions of fractured rock, and whether adequate and relevant data can be collected during site characterisation for a geological repository for nuclear waste. Key components are the geometrical lattice of conductive features (hydrostructural model), the material properties of these features including a description of their internal structure. The conducted solute transport experiments have been carried out in an interpreted single fracture (Feature A) in the detailed scale (L< 10 m) (First TRUE Stage (TRUE-1)), [1] and in a block scale network of structures (L=10-100 m) (TRUE Block Scale) [2], [3] carried out at depths between 400-500 m. Supporting work include laboratory diffusion and sorption experiments, e.g. [4], and numerical modelling, eg. [5].

2. Geological setting

The dominant rocks at Äspö belong to the 1700-1800 M year old Småland granite suite, with mafic inclusions and dykes probably formed in a continuous magma-mingling and magma-mixing process [6], resulting in a very inhomogeneous rock mass, ranging in mineralogical composition from granites to dioritic or gabbroic rocks. Fine-grained granite and pegmatite also occur frequently at Äspö as more or less well defined dykes or veins intersecting the older rocks.

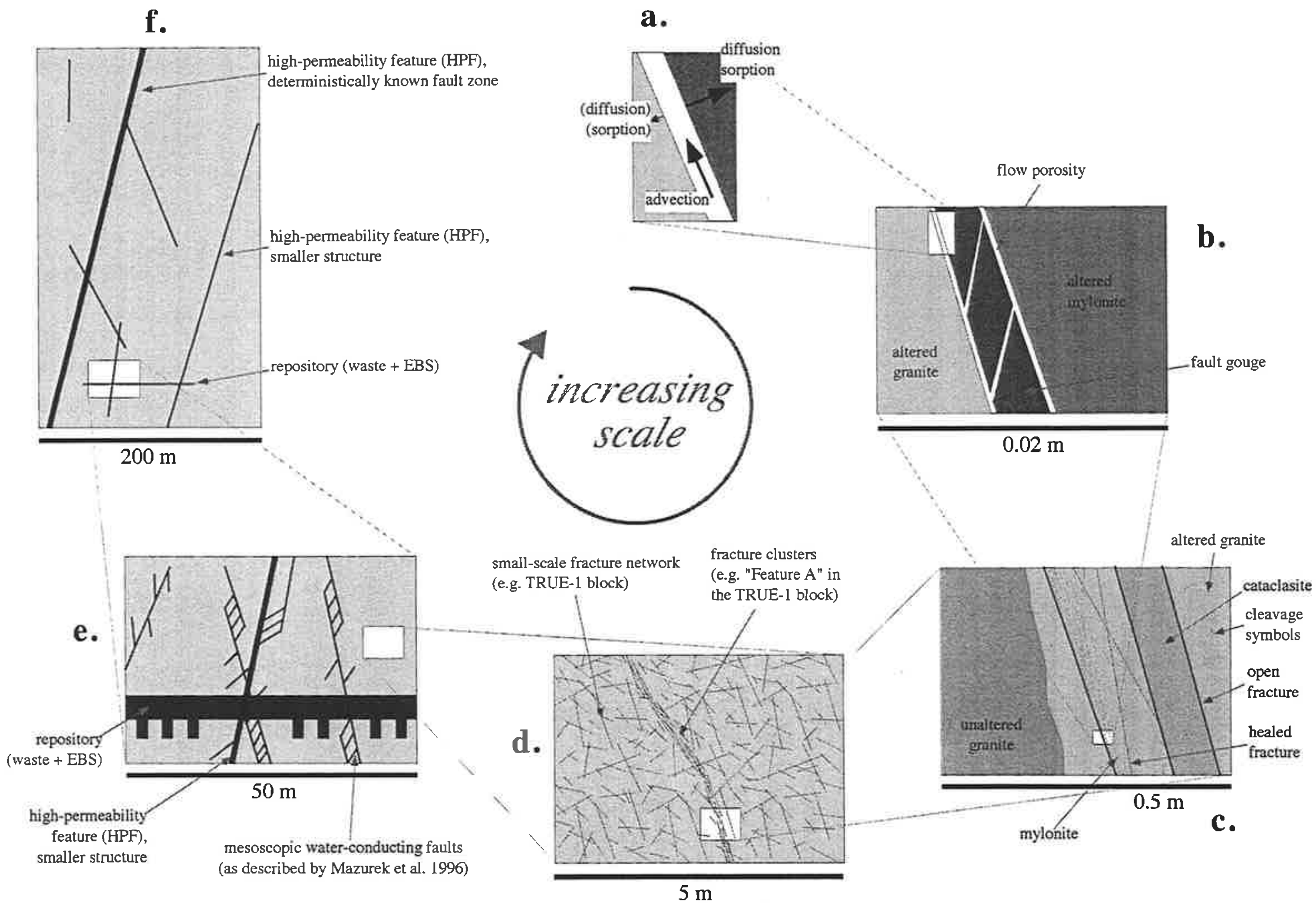

Figure 1. Sequence of conceptual models of conductive features at Äspö showing schematic fracture geometries and flow paths at a range of length scales [10]. EBS = Engineered Barrier System.

3. Overview of conductive features studied at different scales

The majority of water conducting features at Äspö strike NW-SE, ie. parallel to the large scale fault system seen which is oriented NW and NNW. The conductive fractures are also parallel to the present day maximum compressive stress. A minority of the conductive fractures strike NS to NE-SW. Ductile precursors (foliation, ductile shear-zones, mylonites) occur in both systems but are more frequent in the NE-SW system. Subhorizontal conductive features have not been identified [7], [8] and [9].

Conductive structures identified and studied at Äspö HRL represent different length scales and variable complexity, including *high-permeability features* (Section 4), *mesoscopic fractures/faults* (Section 5), *discrete fractures/clusters of fractures* (Section 5) and *members of small-scale fracture networks* (Section 7). The interrelation between different length scales and associated conductive features is indicated in Figure 1, [10]. The conductive features described at different length scales are to a variable degree mutually connected, making up a sparsely connected rock mass. Some of the larger structures have been subject to cement grouting in conjunction with the construction of the Äspö HRL and also in conjunction with site characterisation for the TRUE Block Scale experiments [9].

4. High-permeability features

A High Permeability Feature (HPF) is defined as a fracture, or system of fractures, or a fracture zone with an inflow rate (observed during drilling or flow logging) which exceeds 100 l/min, or alternatively exhibits a transmissivity $\geq 10^{-5}$ m^2/s [11]. Compilation and analysis of conductive structures which fulfilled the above criteria showed that [11]:

- Somewhat less than 50% of the HPF:s can be explained by what is classified during core logging as "crushed zones". The remainder of the HPFs are associated with one or a few natural fractures.
- About 50 % of the HPF:s are associated with the deterministically defined fracture zones of large extent.
- HPFs are found in all rock types but are most frequent in fine-grained granite.
- The arithmetic mean distance between HPFs evaluated from the existing database vary between 75-105 m.

5. Conductive mesoscopic features

This group of "mid range-sized" conductive features can either be faults, discrete fractures, groups of fractures or fracture zones. Their transmissivity ranges from 10^{-8} to 10^{-5} m^2/s and their average inter-distance vary between 10-50 m. Conductive features at Äspö HRL typically show variable amounts of tectonisation (indicated by the occurrence of cataclasites and/or mylonites) and variable degrees of chemical alteration

(alteration of biotite to chlorite, saussuritisation of plagioclase, oxidation of magnetite to hematite, etc.). Almost all studied fractures/structures show alteration and tectonisation of the wall rock, and most borehole intercepts are associated with mylonites.

Fault breccia is common in the mesoscopic features and has i.a. been recovered from some structures in the TRUE Block Scale rock volume using triple-tube coring. The amounts of material collected from borehole intercepts are relatively small making quantitative particle size analysis uncertain. Volumetrically larger breccia samples collected from other structures in the Äspö tunnel, which were sieved for multiple size fractions indicated fractal dimensions D_f for a three-dimensional network between 1.5 and 2.1. The particle size distribution for the so-called Redox Zone [12] is shown in Figure 2. Fault breccia includes variable amounts (up to 40% by weight) of fine-grained material (< 2 mm). The cm-sized breccia pieces and mm-sized breccia fragments are featured by a porosity between 1-3 vol%, whereas the unaltererd wall rock has a porosity of about 0.4-0.5 vol%. The fine-grained (clayey) fractions have been attributed an *in situ* porosity between 10-20% [8], but *in situ* determinations at Äspö HRL are presently not available.

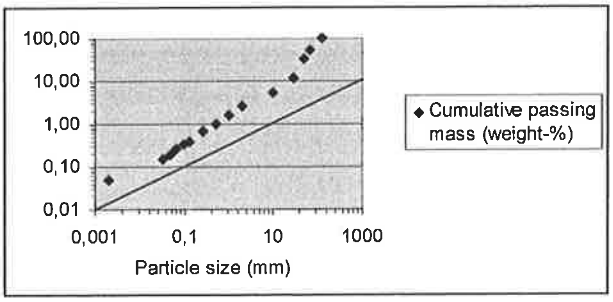

Figure 2. Particle size distribution of fault breccia/fault gouge obtained from a sample collected from the Redox zone (sample collected from the tunnel wall) (data from [12]).

6. Construction of hydrostructural models

Examples of conductive structures belonging to the category defined in Section 5 are Feature A, investigated as part of TRUE-1 [1], and members of the network of conductive structures investigated as part of the TRUE Block Scale project [9]. In the characterisation of these two sites, the principal objective of the site characterisation was to construct a hydrostructural model of the principal conductive structures. The main tools for constructing these models are cross-hole pressure responses (from drilling and pumping tests), high resolution borehole TV, cf. Figure 3, and flow logging.

This enabled identification of conductive features with a resolution of less that 0.1 m with a measurement limit in flow of about 0.002 l/min. In TRUE Block Scale the evolution of the borehole array was iterative with each new borehole being drilled with

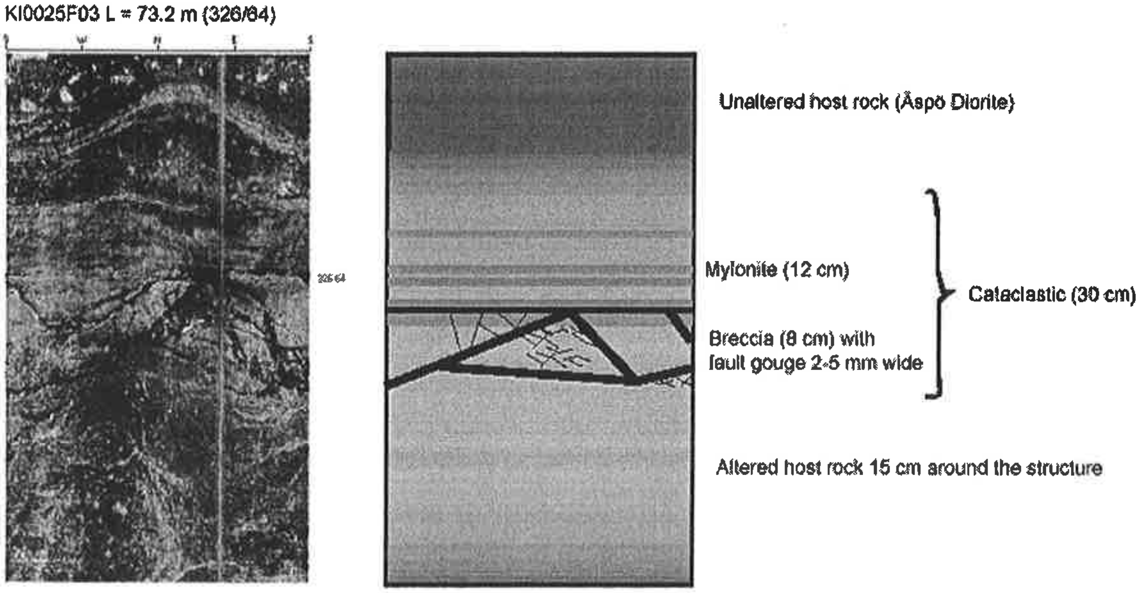

Figure 3. Example BIPS borehole TV image with associated simplified interpretation of lithology, structure and alteration [9]. L=72.96-73.29 m in KI0025F03 (Structure #20).

the purpose of improving the model to a stature where it could be used for planning, performance and interpretation of cross-hole solute transport experiments. An updated hydrostructural model was produced after each drilling and characterisation campaign. The construction of the hydrostructural model was made using the CAD-based Rock Visualisation System (RVS). A plan view of the final hydrostructural model of the TRUE Block Scale rock volume is shown in Figure 4.

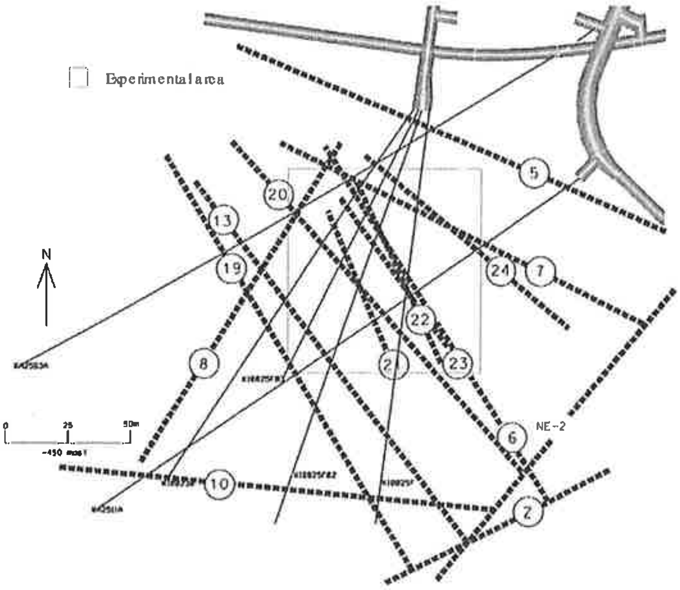

Figure 4. TRUE Block Scale – Hydrostructural model March'00, [9]. Plan view of horizontal section at Z=-450 masl . Access tunnel shown in shaded grey.

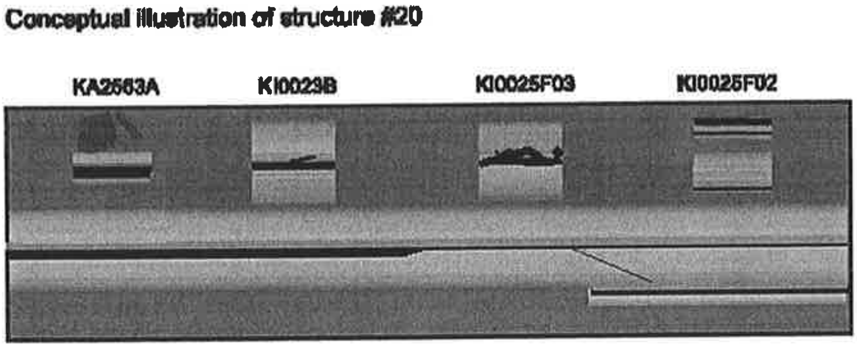

Figure 5. TRUE Block Scale - Integrated conceptual model of Structure #20 [9], cf. Figure 3 for legend and Figure 4 for geometrical context.

Figure 5 shows an integrated conceptual model of Structure #20 over the extent covered by borehole intercepts. Evident from the model is the highly variable character, including brecciated parts and an interpreted division into two fracture planes in the southern part. This variability in character is in accord with findings by Mazurek et al. [8] who found that simple segments (one master fault) could be connected by complex faults steps. The implication being that a structure classified as a simple fault on the basis of a tunnel observation could have a completely different appearance in a borehole intercept a few meters away from the tunnel. The consequence being that the predictive/prognostic capabilities are limited. Further, that a given classification assigned to conductive feature is non-unique, and various classification elements may blend along the extent of the conductive feature.

Those conductive fractures identified in the TRUE Block Scale site characterisation which could not be associated with the interpreted deterministic structures of the hydrostructural model are assigned to a background fracture population [9]. The conductive intensity P_{10} (L^{-1}) of these fractures varied between 0.22-0.27 m^{-1} in the boreholes. Fracture network simulations provided an estimate of the volumetric fracture intensity P_{32} (L^2/L^3) of 0.29 m^{-1}. A mean fracture transmissivity distribution for the background fractures was estimated on the basis of the available flow log results. The transmissivity distribution is log-normal ($^{10}\log T = -8.95$, $\sigma_{10\log T} = 0.93$).

7. Aperture distribution of small scale fractures

An *in situ* epoxy resin injection pre-test [13], [14], [1] was performed as part of TRUE-1 in a system of tunnel-proxy conductive fractures, some one-two orders of magnitude less transmissive than Feature A, cf. Section 8. Analysis of aperture using sections obtained from 200 mm cores revealed mean apertures of 266 μm (CV=37%) and 239 μm (CV=39%) associated with two fractures with distinctive extension (1% contact area) and shear (22% contact area) characteristics, respectively. Aperture was found be represented by a log-normal distribution, although the sheared fracture exhibited a more skewed distribution. Analysis of spatial continuity on a decimetre scale showed very little structure with spatial correlation (practical range) in the order of a few millimetres.

8. Conceptual models of conductive fractures

Conceptual model development for fractures and their immediate surroundings was concentrated to Feature A (TRUE-1), and the structures of the network involved in the TRUE Block Scale tracer experiments. Feature A, cf. Figure 6, is interpreted to comprise a conductive feature that follows a mylonitic brittle precursor. It may either be made up of a single undulating fracture plane over a distance of some 10-20 metres, or be made up of a number of connected fracture planes with slightly variable orientation. The transmissivity of the feature varies between $0.8\text{-}4{\cdot}10^{-7}$ m^2/s. The "cubic law aperture" is estimated to 0.06 mm whereas the transport (mass balance) aperture is estimated to 0.9 mm. No gouge material was collected from the conventional double core drilling performed. However, remnants of clay on fracture surfaces suggest presence of gouge material.

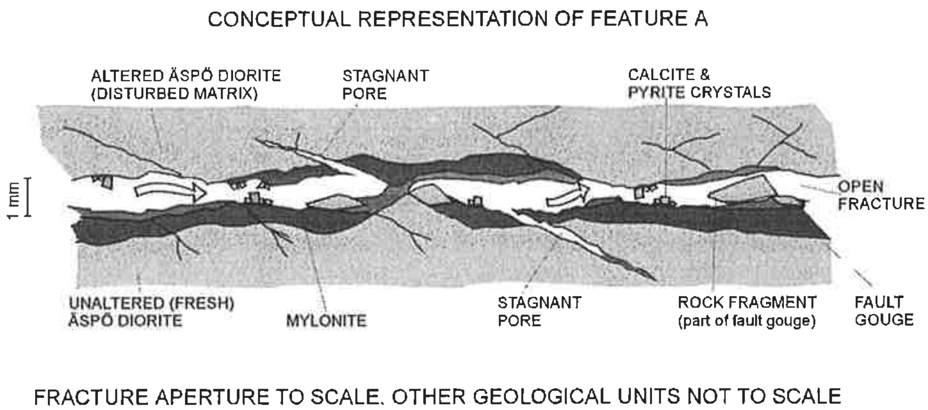

Figure 6. TRUE-1 - Schematic conceptual representation of Feature A in cross section [1]. Note that the fracture aperture is to approximate scale. The thickness of the remainder of the constituents are not to scale.

A generalised conceptual model for the TRUE Block Scale structures, of equitable or higher dignity than Feature A, is shown in Figure 7. The figure features, contrary to the model of Feature A, a more comprehensive filling of void space with fault breccia, cf. Section 5. The areal distribution of fault breccia and fault gouge material is however unknown and subject to uncertainty. It is assumed that physical flow channels of variable widths co-exist with gouge-filled parts along the extent of the studied structures, and along the studied flow paths.

9. Discussion, conclusions and outlook

One of the remaining uncertainties relevant to solute transport is the geometry (pore space) of the actual *in situ* transport paths. Furthermore, the *in situ* distribution of fault breccia, its fraction of fine-grained fault gouge infilling material, and the latter's *in situ*

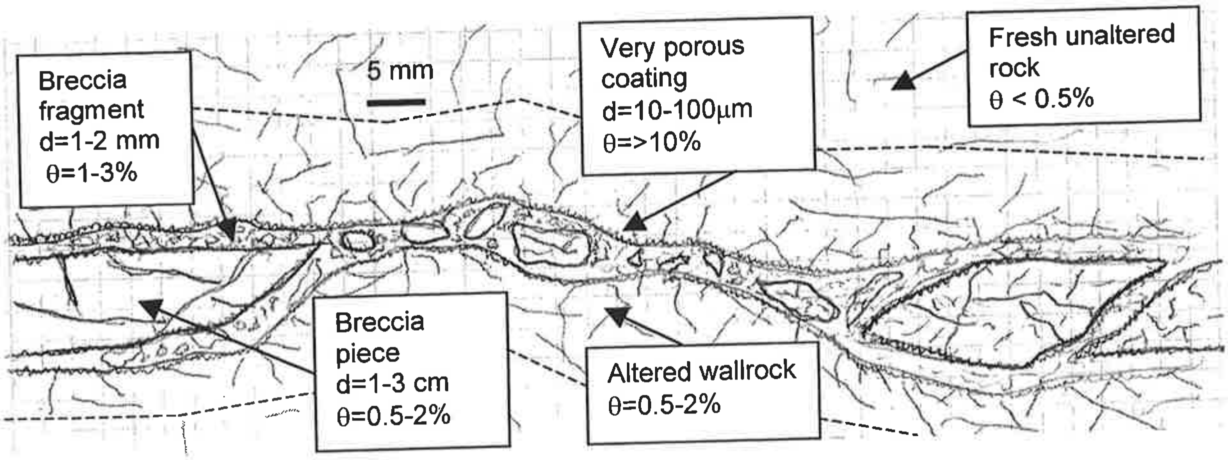

Figure 7. Preliminary conceptual illustration (including thickness d and porosity θ) of a typical conductive structure involved in the TRUE Block Scale tracer tests [9].

situ epoxy resin injection in the Feature A flow paths, with subsequent excavation and analysis, will add information and data which will improve conceptualisation of conductive fractures and help reduce remaining uncertainties.

The conceptualisation performed for solute transport is much more detailed than required for grouting of fractured crystalline rock. However, the detailed conceptual models produced from solute transport experiments and associated data can help improve design, selection of grout recipes and grouting methodology.

It is also noted that the Detailed Characterisation Phase of potential Swedish repository sites, which include investigations from underground openings, will increase demand on grouting performance. This includes use of low-invasive high-performance grouts with limited side effects on the hydraulic and chemical conditions.

10. Acknowledgements

The collective contributions from the field, analysis, and modelling crews of the SKB TRUE Project team, the FCC team, the Äspö Hard Rock Laboratory and colleagues in the TRUE Block Scale Technical Committee, field and analysis teams, are gratefully acknowledged. The TRUE-1 experiment was funded by SKB. The TRUE Block Scale work is funded jointly by ANDRA (France), Nirex (UK), JNC (Japan), ENRESA (Spain), Posiva (Finland) and SKB (Sweden).

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CHARACTERISATION OF FRACTURED ROCK FOR GROUTING USING HYDROGEOLOGICAL METHODS

Karakterisering av sprickigt berg för injektering med hjälp av hydrogeologiska metoder

Åsa Fransson, Department of Geology, Chalmers University of Technology

Summary

This paper aims at briefly presenting a methodology for characterisation of fractured rock for grouting using hydrogeological methods. The conceptual model is based on a grouting fan and is built up by fractures inferred from hydraulic tests and geological mapping. Instead of the commonly used Lugeon value, the specific capacity (Q/dh i.e., flow divided by difference in hydraulic head) is central since it has shown to be a robust parameter which can be related to transmissivity and fracture aperture. Fracture aperture is important for grouting design due to its influence on both penetration length and grout take. The methodology described for estimation of transmissivity and aperture distributions has potential for further development for computer use, which would enable a fast analysis of data from hydraulic tests and geological mapping at a working site. Based on aperture distribution and expressions describing the spreading of grout, the choice of input parameters such as grout properties, pressure and borehole distance could be improved. Furthermore, the transmissivity and aperture distributions for probe holes give a general description of rock, which is used for the interpretation of data from individual grouting boreholes. This description of fractured rock for grouting should be a good basis for further discussions and development as well as facilitating the choice of strategy.

Sammanfattning

Artikeln syftar till att kort beskriva en metodik för karakterisering av sprickigt berg för injektering som utgår ifrån hydrogeologiska metoder. Den konceptuella modell som används som utgångspunkt för analys av data baseras på en injekteringsskärm och består av sprickor man kunnat härleda med hjälp av hydrauliska tester och geologisk kartering. Specifik kapacitet (Q/dh, flöde dividerat med skillnad i vattentryck) har visat sig vara en robust parameter vilken kan kopplas till transmissivitet och sprickvidd och beskrivningen utgår därför ifrån denna istället för det vid injektering vanligt förekommande Lugeon värdet. Sprickvidden är av betydelse för injekteringsdesign eftersom den påverkar både inträngningslängd och bruksåtgång. Den metodik som använts för att uppskatta fördelningar av transmissivitet och sprickvidd har potential för

fortsatt utveckling för datoranvändning vilket skulle möjliggöra en snabb analys av data från hydrauliska tester och geologisk kartering direkt på plats. Baserat på sprickviddsfördelning och ekvationer som beskriver bruksspridning kan sedan valet av indata såsom bruksegenskaper, tryck och borrhålsavstånd förbättras. Transmissivitets- och sprickviddsfördelningar för sonderingshålen ger dessutom en mer generell beskrivning av bergmassan som i sin tur kan användas för att tolka data ifrån enskilda injekteringshål. Att beskriva den sprickiga bergmassan på detta sätt bör vara en bra utgångspunkt för diskussioner och utveckling samt underlätta valet av injekteringsstategi.

1. Introduction

The construction of a laboratory facility at Äspö, the Äspö Hard Rock Laboratory, Sweden, to investigate questions related to the deposition of nuclear waste has given rise to a need for further understanding and development of grouting technique. This is crucial, since the construction of access ramps, transport tunnels and deposition tunnels must meet demands for safe and controlled conditions that facilitate suitable sealing. Furthermore, information about the spreading and amount of grout injected is useful in analysing the behaviour of a deep repository when it is finally sealed. The work was initiated by the Swedish Nuclear Fuel and Waste Management Company, SKB, and this project, Characterisation of rock for grouting purposes (Fransson, 2001), is coupled to a parallel project named Spreading of grout in fractured rock, which is carried through at the Division of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm (e.g. Eriksson, 1999 and Eriksson *et al.* 2000). The main aims of the grouting project are: to increase the understanding of what properties are of importance, to quantify them, and to describe the result in a model of the grouting process. This knowledge should be utilised to form a strategy for sealing both larger discontinuities with high piezometric head and smaller fractures.

The objective, characterisation of rock for grouting purposes, was to establish a characterisation methodology that has four basic qualities. First, it should be consistent throughout all of the stages of a project. Second, it should be based upon a reasonable amount of tests and data. Third, it should facilitate the choice of strategy, such as e.g. the drilling pattern and what grout to use. Finally, it should enable predictions of grout take, penetration lengths and result of the application. The characterisation for grouting is looked upon as an engineering problem and the methods used are hydraulic tests and geological mapping. Simplifications are made to develop a method and a geometrical model, which should consider the prevailing conditions, such as limitations in time and availability of data.

2. Conceptualisation

Sealing of tunnels by grouting is a method commonly used to minimise inflow of water and to enhance the stability of the tunnel. To get an idea of what strategy to use before grouting a section of a tunnel, water loss measurements are performed in the boreholes of the grouting fan, resulting in a number of Lugeon values (Houlsby 1990 and Kutzner 1996). These values represent the volumes of water that are injected per unit time and per metre of borehole at a given pressure. However, they do not give any information about the discontinuities that actually transmit the water. The conceptual model developed for this characterisation is based on one individual grouting fan, where the boreholes cross fractures known to have varying ability to transmit water as well as different orientations and fracture lengths. The grouting parameters considered for this conceptual model are mainly the penetration of grout and the grout take, which are both influenced by the aperture of the fractures, see e.g. Gustafson and Stille (1996). The ability of a fracture to transmit water, or its transmissivity, is also dependent upon this fracture aperture, which makes it a key parameter in this work. Hydraulic tests and geological mapping were used as main investigations and the intersected fractures were assumed to be two-dimensional (2D) with radial flow which provide a useful and robust description to help solve this engineering problem, since it allows two-dimensional analyses for both hydraulic tests and grouting predictions. Fractures inferred from hydraulic tests (here full-length or section tests) and geological mapping should be connected to form a simplified model (referred to as "2.5D" since it is between 2D and 3D), Figure 1.

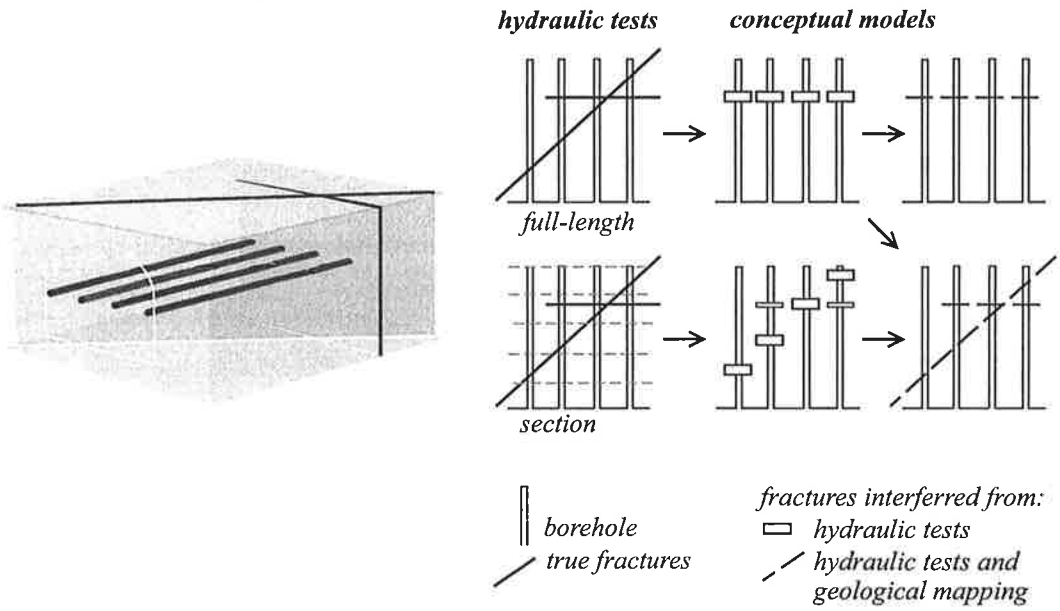

Figure 1 Principal sketches of a grouting fan where four boreholes cross two fractures. The upper part represents full-length hydraulic tests as well as the resulting conceptual models consisting of one fracture whereas the lower part from the section tests enables a more detailed description.

Consequently, the "2.5D" model consists of stacked 2D features along boreholes which, if possible based on transmissivity and orientation, are connected to other features of adjacent boreholes. The figure shows conceptual models inferred from hydraulic full-length and section tests.

Here, fracture orientation is considered important to increase the probability of intersecting the fractures, whereas the length gives an indication of the connectivity and where connection between boreholes may be expected. Figure 2 presents a conceptual overview of the main components investigated to look at the usefulness of hydraulic tests and geological mapping when characterising rock for grouting purposes. The overview includes: one discrete fracture; the fracture distribution along a borehole (probe hole) for which a non-parametric model was developed; fractures in a smaller volume of rock for a field experiment (the inner cube in the figure which could also represent a grouting fan) and; analyses of data from a larger area such as a tunnel situation (the large cube).

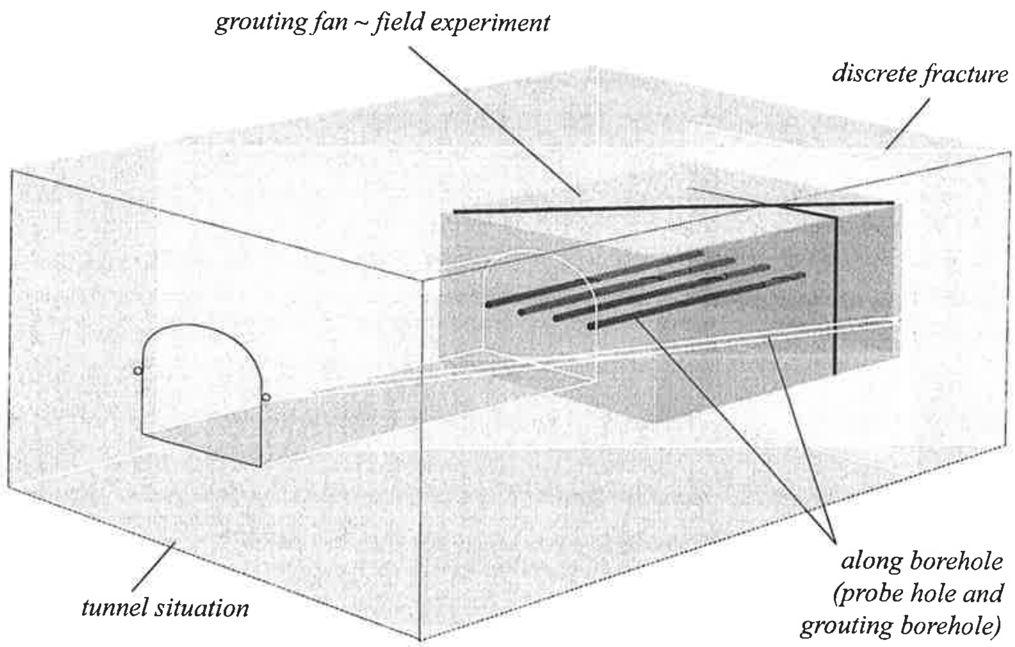

Figure 2 *Conceptual overview of the main components investigated to look at the usefulness of hydraulic tests and geological mapping when characterising rock for grouting purposes.*

3. Results and discussion

A field strategy that is based on this work is presented in Figure 3. The flow chart describes how the investigations and parameters are connected (figures in the text refer to the figures in the flow chart). The scale is here represented by a tunnel situation or a grouting fan, see Figure 2. The tunnel situation is used, since data originate from, and are aimed at describing, a larger part of rock where the parameters obtained are used to improve investigations, for predictions and for analyses of data from grouting boreholes within a fan. The volume of rock found immediately ahead of the tunnel front is referred to as a grouting fan. Investigations are geological mapping (with focus on fracture mapping) and investigations of probe holes and grouting boreholes within a grouting fan.

For the field test, the *orientation of fractures*, see (1) Figure 3, was used to increase the probability of the boreholes intersecting the fractures (2). *Fracture lengths* (3) are obtained directly from geological mapping; a measure of the size of conductive fractures is found by means of variogram analyses of the specific capacity from grouting boreholes (4). If adjusting the distance between the probe holes (5) to the intermediate fracture length, the representativity of probe holes and the predictions from data will be improved. Data from these probe holes (6) are subsequently used to calculate a *probability of conductive fractures* and a *distribution of transmissivities* (7) that accounts for the fixed interval length transmissivities and the number of fractures for each corresponding interval or section. From the transmissivity distribution, an *aperture distribution* is estimated and, since the grout take is proportional to the transmissivity and the penetration length is proportional to the aperture these distributions are valuable. The results from the probe holes (7) give an initial description of the rock ahead of the tunnel front, according to the transmissivity distribution, which is also used for analyses of data from grouting boreholes (8). These analyses give *probable number of conductive fractures* and *a distribution of the specific capacity* (9). The specific capacity, which was found to be close to the transmissivity and robust enough for grouting predictions (Fransson, 1999), reflects an aperture which influences penetration length and grout take. Furthermore, the fracture aperture and the hydraulic head are good as reference points when choosing type of grout and additives.

In brief, the characterisation methodology above gives a possibility to form a simplified model of a grouting fan based on hydraulic tests and geological mapping. Instead of using Lugeon values, the water pressure tests are performed to obtain a specific capacity (Q/dh, i.e., flow divided by difference in hydraulic head). Transmissivity and based on this work, specific capacity, are important for grouting design since they provide information about the discontinuities that actually transmit the water. Firstly, see Figure 4, transmissivity and specific capacity are important because the estimated apertures, b(Q/dh), give guidance about the groutability or what fractures we can expect to seal and what grout to use. Further, the result or the transmissivity and inflow ($Q \approx T*dh$) that would remain after sealing these fractures can be related to a tightness criterion.

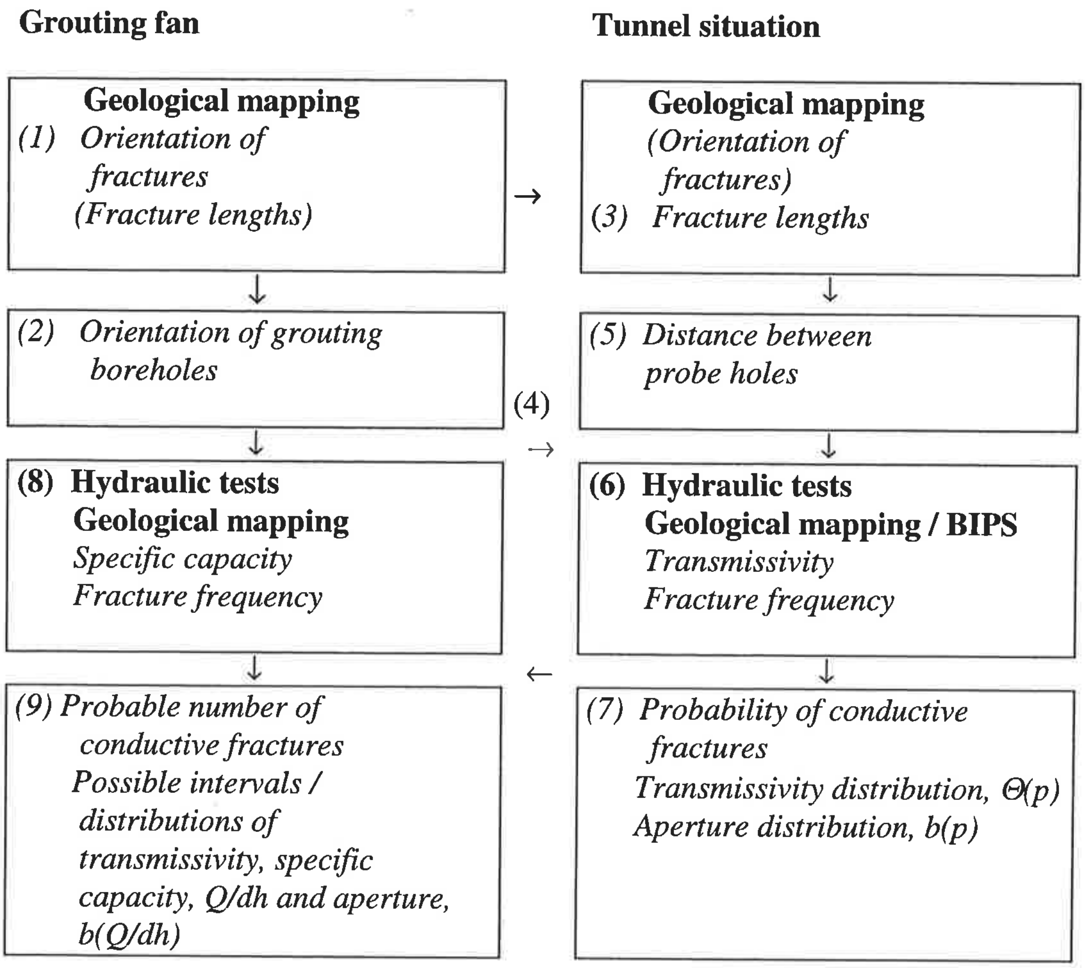

Figure 3 *Flow chart describing the field strategy.*

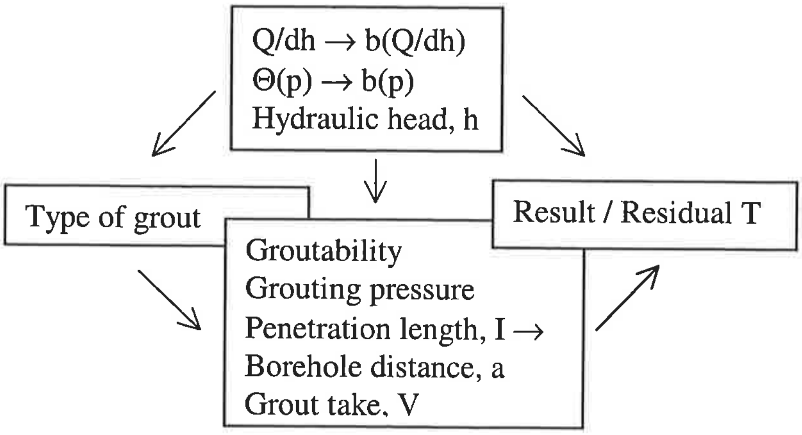

Figure 4 *Sketch showing how the different parameters are used in a grouting strategy.*

Based on a transmissivity distribution, Θ(p), (obtained from hydraulic tests and geological mapping of a probe hole, (6)-(7), Figure 3), a distribution of apertures, b(p), and subsequently penetration lengths, I(p), can be estimated, see Figures 4 and 5. In Figure 5 apertures smaller than 1×10^{-4} m are assumed not to be groutable. Using this, the penetration length and borehole distance, a, should be compared and adjusted to improve the grouting design. A residual transmissivity could be estimated based on transmissivities of those fractures that are expected not to be sealed or only partly sealed.

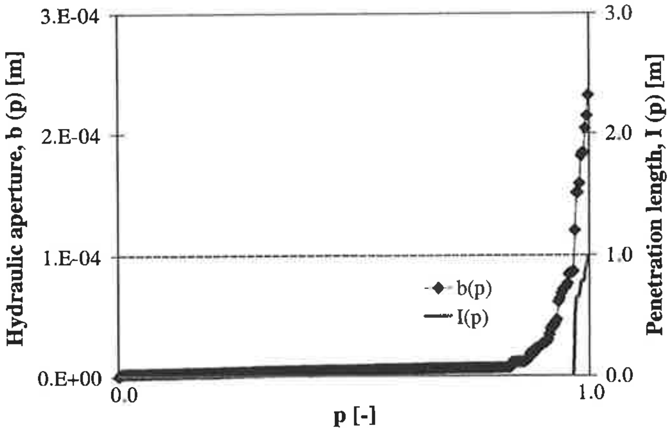

Figure 5 *An example of an aperture distribution and an estimated distribution of penetration length (Bingham model). Fractures with apertures below* 1×10^{-4} *m are assumed not to be groutable.*

Further, information from the boreholes within a grouting fan is used to construct a simplified "2.5D" model (Figures 1 and 6). A model where fractures inferred from hydraulic tests and geological mapping are connected. Investigations show that the specific capacity is a robust parameter and the median specific capacity, Q/dh_{50}, of several boreholes crossing the same fracture is close to the effective or cross-fracture transmissivity. This is a representative value of the fracture crossing the tunnel and the differences between the median and the other specific capacities or apertures indicate the variations in aperture within the fracture. Based on this conceptual model, penetration lengths are estimated from obtained apertures and subsequently compared and adjusted to the borehole distance, a. In this example (Figure 6), one fracture is fully sealed and the other partly sealed due to a smaller aperture. Here as well, a residual transmissivity could be estimated based on transmissivities of those fractures that are expected not to be sealed or only partly sealed.

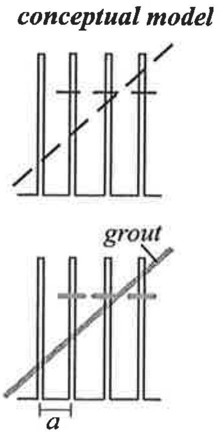

Figure 6 "2.5D" model based on a grouting fan (see Figure 1) where estimated apertures and assumed pressure and grout properties give one completely sealed and one partly sealed fracture.

4. Conclusions

Based on the investigations presented in Figure 2 (Fransson, 2001) it was concluded that specific capacity (Q/dh) from short duration hydraulic tests is robust enough to describe fracture aperture which influences both penetration of grout and the grout take. Using the specific capacity would be an improvement compared to the commonly used Lugeon value since it is more easily linked to the transmissivity and the aperture of a fracture. Analytical, numerical and experimental (both in the laboratory and in the field) approaches indicate that a small number of hydraulic tests are enough to give a general picture of the intersected fracture. They all show that the median specific capacity was found to be close to the effective cross fracture transmissivity. The importance of the cross fracture transmissivity is also reflected by the commonly good correlation found between transmissivity and grout take using analytical and numerical methods. Data from probe holes were used to calculate a probability of conductive fractures and a distribution of transmissivities using the non-parametric model developed. From the transmissivity distribution, an aperture distribution is estimated and, since the grout take is proportional to the transmissivity and the penetration length is proportional to the aperture, penetration length distributions could also be estimated. This could be used as guidance for choosing grout, grouting pressure, grouting borehole distance and give an idea of the ability of the remaining ungrouted fractures to transmit water. Furthermore, the probe holes can be used to improve the predictions for, and interpretation of data from, individual grouting boreholes. These data, obtained along individual boreholes using hydraulic tests and geological mapping, can be linked to form a simplified "2.5D" model. This was tested and verified during the field experiment as well as strengthened by other similar grouting fans.

The methodology described for estimation of transmissivity and aperture distributions has potential for further development for computer use, which would enable a fast analysis of data from hydraulic tests and geological mapping at a working site. Based on the aperture distribution and expressions describing the spreading of grout, input parameters such as grout properties, pressure and borehole distance could be chosen. Furthermore, the transmissivity and aperture distributions for probe holes give a general description of rock, which is used to improve the interpretation of data (specific capacity and fracture frequency) from individual grouting boreholes. This description of fractured rock for grouting should be a good basis for discussions and facilitate the choice of strategy.

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ROCK MECHANICS EFFECTS OF CEMENT GROUTING IN HARD ROCK MASSES

Ph.D. Staffan Swedenborg
NCC, Gothenburg, Sweden

Abstract
Large effort has been put into understanding how cement grout penetrates and hydraulically seals a fractured hard rock. Whether the grout sealing is capable to withstand stress re-distributions due to rock excavations is, however, sparsely discussed. This paper presents parts of a conducted research program with the aim to investigate the principal difference between grouted and ungrouted joints. From direct shear tests of grouted and ungrouted rock joint replicas, it was found that cement grout acts basically as a 'lubricant' in a joint subjected to shear stress. Initial numerical modelings, to study the consequences in a tunneling situation, are presented. The 'hydraulic failure[1]' was found to be a function of dilation and coincides with the joint peak strength. Under adverse geological conditions, such failure may propagate quite far into the surrounding rock mass, numerical modeling indicates more than two times the tunnel diameter.

Sammanfattning
Stora insatser har gjorts för att förstå hur injekteringsbruk penetrerar och tätar sprickor i hårt berg. Huruvida injekteringsbruket klarar av den spänningsomvandling på grund av bergsschakten diskuteras sällan. Denna artikel redovisar resultat från ett forskningsprojekt vilket syftade till att studera det principiella brottbeteendet hos cementinjekterade bergssprickor i relation till oinjekterade bergsprickor. Av de direkta skjuvförsöken med injekterade och oinjekterade sprickor kan man dra slutsatsen att injekteringsbruket fungerar som ett smörjande media i ett sprickplan utsatt för skjuvrörelser. Det "hydrauliska genombrottet" är en funktion av sprickplanets dilatans och uppträder samtidigt som det mekaniska brottet. Vid ogynnsamma geologiska förhållanden kan uppsprickningen av injekterade sprickor fortskrida långt ut i kringliggande bergmassa. Enligt de utförda numeriska modelleringarna kan uppskräckning uppträda upp till dubbla tunneldiametern från tunnelperiferin.

1. Introduction

One problem occurring while tunneling below the ground water table, is the inflow of water. If it is needed to "water-proof" the underground opening, pre-grouting can considerably reduce the transmissivity of the rock mass near the opening. Research conducted the last four decades has revealed most of the mechanisms that govern the permeation of grout into rock joints. Work procedures and quality control systems have been elaborated to ensure satisfactory grouting results. A limited number of control holes are drilled to verify dry conditions ahead of the tunnel face, prior to rock excavation. Should leakage[1] occur in the newly excavated area despite of dry control holes, the water bearing structure were maybe not intercepted by any of the grout or control holes, and therefore still transmissive. The research described here gives an alternative explanation to the phenomenon.

It is well known, that the excavation of a tunnel causes rock stress re-distribution; in some cases rock support measures are required to ensure the integrity of the underground opening. Rock mass deformations due to excavation may introduce shear stresses and subsequent dilation, capable of breaking the grouted joint and causing hydraulic failure of the previously sealed joints. This process is governed by joint properties, the orientation of joint sets and by the ambient rock stress field.

To the authors' knowledge, the properties of grouted rock joints have not previously undergone scientific study. Therefore, a laboratory investigation was conducted as a part of this research.

2. State of the art

Direct shear test on cement grouted joints were investigated by Coulson (1970) on artificial joints from cores of coarse and fine-grained granites by using a modified Brazilian splitting technique. These tests concluded that the shear strength of grouted joints was considerably lower than that of similar ungrouted joints. Another important observation was the transition of failure mode, depending on the normal stress level. At normal stresses below 0.8 MPa, a bond failure was observed in the interface between grout and rock. Hardly any deformation was required to reach the peak value in this failure mode. At higher stress levels, the failure progressed through the grout. To reach the peak strength in this mode, a certain amount of deformation was needed.

Like Coulson, Barroso(1970), found a reduction of shear strength of grouted joints, compared to the ungrouted ones. He also noted a change of failure mode, depending on the normal stress level.

For both studies above, the grout thickness in the joint was keep constant over the entire sample area, but varied in different tests. This excluded the possibility to examine the interfering failure, when both rock-to-rock contacts and grouted voids interact.

Joints filled with cohesive and frictional soils were tested for shear strength. Toledo&Freitas (1993) presented a comprehensive study, focusing on clay infillings. They concluded, for regular "jigsaw" samples, that the thickness of the infilling, compared to the amplitude of the surface, was most important for the failure mode. For saturated clay, the shear rate and drainage conditions were also significant. They concluded that two 'critical thicknesses' guide the failure mode; the thinner thickness represents the required thickness to disturb the rock-to-rock contact, whereas the thicker one is required to eliminate the effects of the sample surface geometry. Their findings are presented in Figure 1.

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[1] Hydraulic failure, coined in this paper, refers to the process when grouted joints re-gain their ability to conduct water

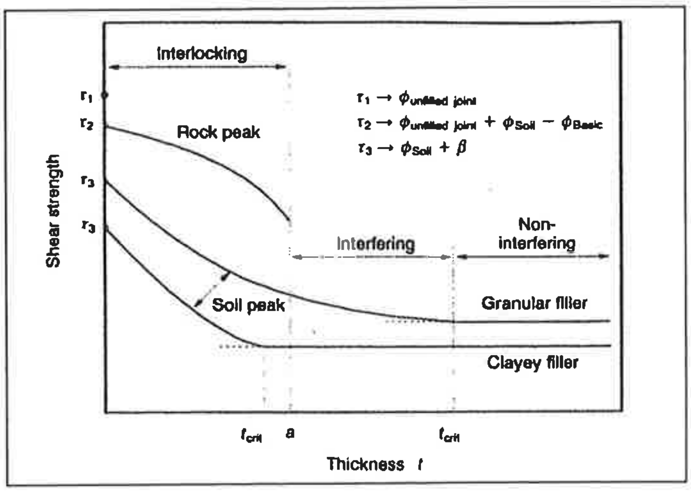

Figure 1: Strength model for infilled joint, after Toledo&Freitas (1993).

Their conclusions contradicts some earlier works, for example by Goodman (1970), who found that the combined friction angle of a filled joint could be less than the friction angles of the components making up the system. The line representing the interfering part in Figure 1 should probably be a dip, caused by rolling action of the infilling grains.

Carter&Ooi (1988) reported results of direct shear tests on concrete-rock interfaces. They used samples were the lower part was a diamond-cut sandstone surface upon which a concrete upper half was casted. The setting simulates the interface between rock and concrete in a cast-in-place pile foundation. The objective of the work was to characterize the strength and deformation behavior of the pile shaft when loaded. The shear resistance observed was divided into cohesion, C, friction, ϕ, and dilatancy, i. Dividing the shear deformation, u, into elastic, u^e, and a plastic part, u^p, and by fitting the curves, with exponential functions, the model becomes:

$$\Delta u = \Delta u^e + \Delta u^p \tag{1}$$

$$C = C_0 \qquad \text{if} \qquad u^p < \delta \tag{2}$$

$$C = C_0 \cdot exp\left(-k_1 \cdot \left(u^p - \delta\right)\right) \qquad \text{if} \qquad u^p \geq \delta \tag{3}$$

,where δ is the threshold value of plastic shear displacement, at which damage of the cohesive capacity commences. In the same manner, the mobilized frictional angle ϕ is a function of the plastic shear displacement:

$$\phi = \phi_0 \cdot \left(1 - exp\left(-k_2 \cdot u^p\right)\right) \tag{4}$$

For the dilatancy i, a similar function emerges from fitting the test data:

$$i = i_0 \cdot exp\left[-k_3 \cdot \sigma \cdot \left(\left(1 - u^p / \lambda\right) \cdot q_u\right)\right] \quad \text{if} \quad 0 \le u^p \le \lambda \tag{5}$$

$$i = 0 \quad \text{if} \quad u^p > \lambda \tag{6}$$

,where σ = normal stress; λ = plastic shear displacement when dilatancy ceases and q_u = uniaxial compressive strength of the weakest material. The model describes an interface between two materials, rather than a composite bond, such as the one between joint and grout. It is still believed to give a good indication of the mechanical behavior.

3. Performed Direct Shear tests

Flat surface samples

The test performed in this study on diamond-cut flat surfaces had two objectives; to verify that the selected replica material was suitable and to investigate the basic shear behavior of cement grouted joints. The tests were performed in the direct shear box at Chalmers University of Technology. The sample size was 100x100 mm, and the reference setting: *wcr* 0.8; σ_n=4Mpa; aperture=3mm; horizontal failure plane.
Following parameters were varied in the tests; normal stress σ_n, inclination of the diamond-cut surface, grout thickness *t*, and water-cement ratio *wcr*. Grouted samples were prepared and submerged for 7 days of curing.

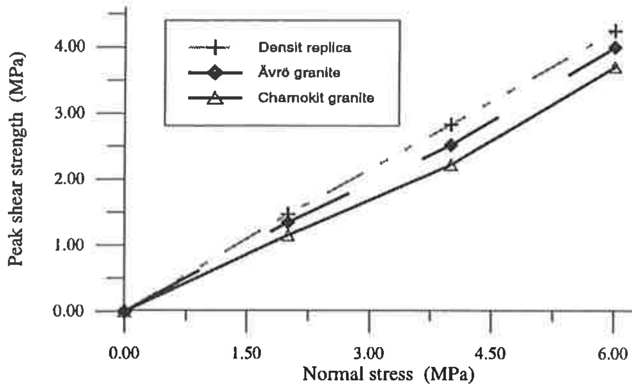

Figure 2 Established basic frictional angles.

The basic friction angle ϕ_{bas} of the replica material, Densit T2, was slightly higher than the reference crystalline rocks used, see Figure 2. But when grout was placed in-between the surfaces, this difference vanished, see Figure 3.

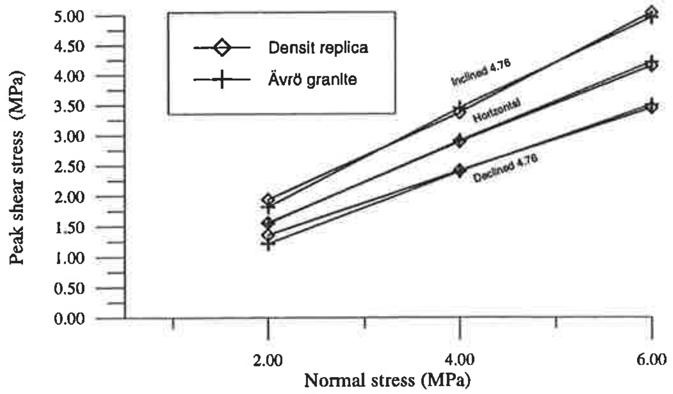

Figure 3 Results of shear test on grouted, 3mm joints.

A large number of tests were performed; these generally showed that the grouted joints had distinct cohesive capacity and that the shear strength was proportional to the applied normal stress. The main failure plane occurred near the interface between rock and grout, but always slightly into the grout. The phenomenon is known from the concrete industry and is usually explained by a slight increment of water cement ratio,*wcr,* close to coarse aggregates. A Mohr-Colomb based failure criterion appears reasonable. However the contribution of the cohesion vanishes after failure, a bi-linear criterion seems better. Theoretically, disregarding interactions of cohesion and friction, following relations are valid:

$$\tau_{peak} = C_0 = \tau_{adhesion} \qquad \text{if } u^p = 0 \qquad (7)$$

$$\tau_{peak} = \sigma_n \cdot tan(\phi_{grout}) \qquad \text{if } u^p > 0 \qquad (8)$$

In conclusion, full cement grouting of a void will increase the peak shear capacity at low normal stress, but will decrease it at stress levels common in rock engineering situations. Both $\tau_{adhesion}$ and ϕ_{grout} depend on the *wcr* of the grout used.

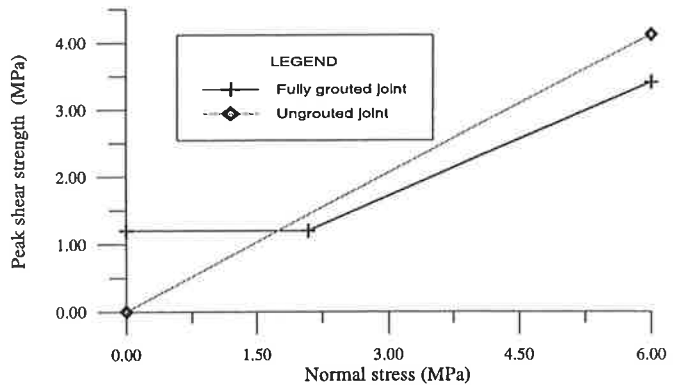

Figure 4 Failure envelope of fully grouted joints.

Furthermore, the failure mode at high normal stress levels divides into two categories. At the highest stress levels in this investigation, Riedel shears[2] propagated through the grout layer, allowing for local steep sliding within the grout layer. This caused considerable contraction of the grout layer. This phenomenon did not interact with the main failure plane, located close to the rock-grout interface.
What is stated above is valid for hard crystalline rocks; geological materials with low basic friction angle have not been investigated.

Jigsaw Samples

The results from samples100x100mm with symmetrical teeth, angled 27°, and with amplitude of 5 mm are shown in Figure 5.

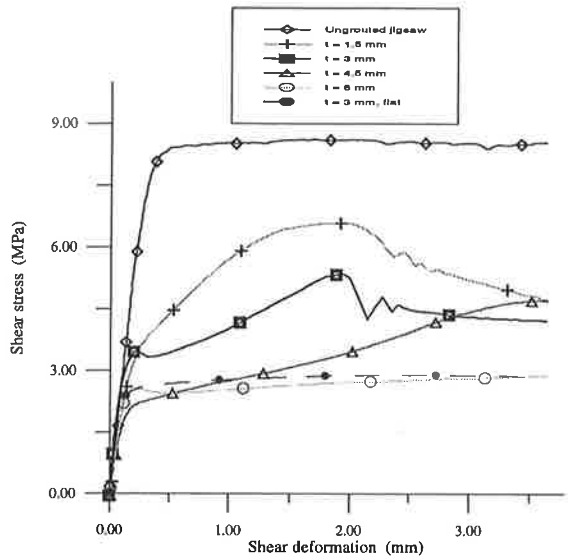

Figure 5 Shear-displacement curves for grouted "jigsaw" samples of varying thickness.

These test results bear a remarkable resemblance to the results in previous studies, using clay-filled joints of similar geometry, Indraratna *et al* (1999). The results focus on an important point; the thickness of the grout, separating the two irregular surfaces, will affect the mobilization of the peak frictional angle and rock-to-rock contact at different relative shear displacements In a natural joint, rock-to-rock contacts will always be present. The pronounced difference between apertures t=4,5 mm and t=6mm indicates that the critical thickness *t*, divided by the amplitude, *a*, of the surface, is unity.
The characteristics, visible in Figure 5 for both rock-to-rock contacts and grout filled channels, will interact to form the shear/deformation behavior of the grouted rock joint in the field.

[2] Multiple shear failure in thin layer due to the stress situation

Preparation of rock joint replicas

The shear box at University Joseph Fourier in Grenoble, used for this study on grouted rock joint replicas, required specially prepared samples. The cast replicas were made of Densit T2 with q_u =210 MPa. The *à-priori* estimated JRC was in the range 5 to 9 for the four selected samples. Two of the joint samples originated from fine-grained granite, the other two from coarse-grained granite. After fitting the samples into the frames, a center hole was drilled in the lower sample for joint access.

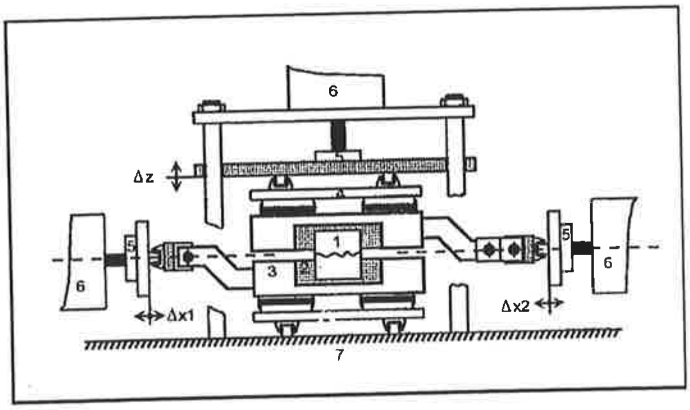

Figure 6 Principle of the shear box at UJF, Grenoble.

(1) Sample (2) Frames (3) Shear-boxes (4) Guideing plate (5) Load cells (6) Servo screw jacks

Grouting of rock joint replicas

The introductory tests indicated a large span of shear resistance, depending on failure mode. From the literature, it is known that permeation of cement suspensions depends much on the size of the void opening. Therefore, the sample halves were fitted to each other and a normal stress of 0,5-1 MPa was applied and locked mechanically to withstand the grouting pressure. In the gap between the two frames, affixing the joint sample, a silicone rubber sealing was fitted. The grout was injected through the center hole in the lower sample. The grout had cement particle size d_{95}<30 μm, a water-cement ratio *wcr* between 0.7 and 0.9 with 1 % plastizicer added.

Testing of grouted rock joint replicas

Prior to installing the samples in the shear box, the grout hole was re-drilled and a coupling was fitted, used to apply a water pressure in the joint plane. Since it was assumed that plastic normal deformations of the grout could occur at cycles with large normal loading, possibly destroying the hydraulic seal, the samples were loaded to the normal load of the test in one step. A water pressure of 2 MPa was applied to the center hole. A pressure conductor was connected to the hydraulic system; a loss of water pressure indicating a hydraulic failure.

The shear stress was applied at a rate of 1,0 mm per minute while automatically logging all events. The total shear displacement was 10 mm, thereafter shearing was reversed to its original position.

4 Results of direct shear tests

Grouted and ungrouted rock joint replicas

Many failure modes are present when a grouted rock joint is subject to direct shear tests. One objective of the test program was to study the difference between the grouted and ungrouted case and to establish mechanical parameters to be used for rock mechanical modeling. Ungrouted samples displayed JRC values in a small interval, 5.6 to 6.9. For grouted joints, a model based on Carter&Ooi(1988) was used, see chapter 2.

Comparison of grouted and ungrouted rock joint replicas

When comparing ungrouted samples with grouted ones, applied normal stress or water-cement ratios injected were varied. Results are exemplified below.

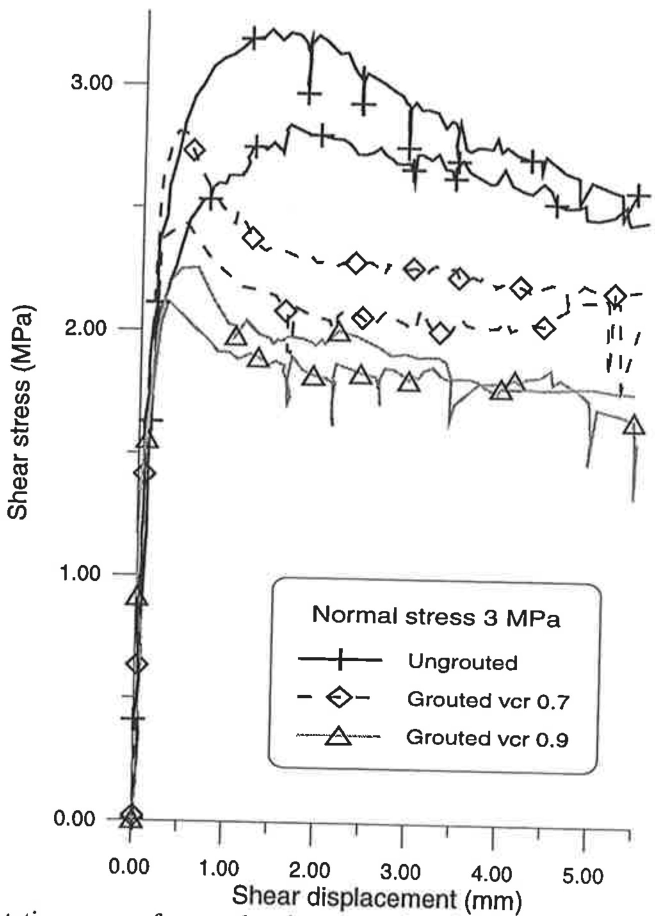

Figure 7. Representative curve of grouted and ungrouted rock joint replicas from one test set-up.

The curves reveal the basic difference of grouted rock joint behavior, compared to ungrouted. Grouted joints reach their peak shear strength when the cohesive part, $\tau_{adhesion}$, reaches its peak and then fails. Un-grouted joints reach their peak shear strength when the dilatancy is fully mobilized. Comparing the two, ungrouted joint require much larger relative shear displacement to reach peak strength

Since most of the normal stress is transferred via rock-to-rock contacts, the peak strength has to be the function of the cohesion <u>and</u> the mobilized friction. The peak is reached when plastic deformations commence, i.e. when the grout cohesion fails. Note that the dilatancy of the joint does not contribute. Following relation is obtained:

$$\tau_{peak,grouted} = \sigma_n \cdot tan(\phi_{rock}) + \tau_{adhesion,grout} \tag{9}$$

Further examination of the curves for the rate of dilantcy and hydraulic flow enables some other conclusions.

When the grout cohesion fails, plastic deformations are initiated, which cause dilation since the relative shear displacement has commenced. Therefore, at peak shear strength of grouted joints, the hydraulic failure occurs concurrently. Before plastic deformations occur, the friction angle between the rock walls equals the basic frictional angle of the host rock material, the walls remains in contact from the time of grouting. In the process of hydratization, the hardened grout fills the entire void space. Therefore, as soon as plastic deformations commence, hydrated cement will interact in the friction plane between the joint sides, reducing the friction. After failure, the shear resistance will increase due to dilatancy, but reduce due to rolling action of grout particles in the rock-to-rock shear zones.

Furthermore, assuming that the cohesive failure of the grouted bond is brittle, the initial part of the shear-displacement curves is mis-leading. The apparent joint shear stiffness $[\Delta\tau/\Delta u^e]$ is not a parameter of the joint, but the elastic shear modulus of the samples and the slack of the entire test set-up.

Assessment of input parameters for numerical modeling

The following behavior of grouted rock joints can be assumed based on the performed direct shear tests, Carter&Ooi (1988) and be used for numerical modeling;

$$\Delta u = \Delta u^e + \Delta u^p \tag{1}$$

$$C = C_0 \qquad \text{if} \qquad u^p = 0 \tag{10}$$

$$C = C_0 \cdot exp\left(-k_1 \cdot u^p\right) \qquad \text{if} \qquad u^p > 0 \tag{11}$$

,where u^e is the threshold value of shear displacement at the start of damage of the cohesive bond. The mobilized friction angle is also a function of the shear displacement, since the joint has rock-to-rock contacts;

$$\phi = \phi_{rock} \cdot \left(1 - exp\left(-k_2 \cdot u^e\right)\right) \tag{12}$$

However, since the grout decreases the frictional shear resistance, a correctional function is needed:

$$\phi_{red} = (\phi_{rock} - \phi_{res,grout}) \cdot \left(1 - exp\left(-k_4 \cdot u^p\right)\right) \tag{13}$$

In the grouted case, the efficient friction angle after cohesive failure becomes

$$\phi_{efficent} = \phi_{rock} - \phi_{red} \tag{14}$$

For the dilatancy, function received from fitting of test data becomes:

$$i = i_0 \cdot \left(1 - exp\left(-k_3 \cdot u^p\right)\right) \qquad \text{if } 0 \le u^p \le \lambda \tag{15}$$

$$i = 0 \qquad \text{if} \quad u > \lambda \tag{16}$$

The effect of aspiritis has been omitted. This is due to two reasons;

- The effect of aspiritis is limited at large scale. Barton (1981) suggested that it could be assumed to be in the range of 1° for numerical modeling of rock masses.
- When examining the joint surfaces after shearing, the damage of the grouted joints is significantly less than of the ungrouted ones. The grout fills the inverts of the joint surface and prevents interlocking which occurs in ungrouted joints. The effects of aspiritis can be assumed to be incorporated in the frictional and dilatancy factors.

5. Numerical modeling

A numerical model provides an opportunity to study the principal stress and deformation behavior of bodies given certain boundary conditions. For some engineering disciplines, the method may be used for detailed design. But for rock engineering, where input data for any rock mass and the *in-situ* rock stress state are highly uncertain, study of general behavior is more adequate.

Assessment of input parameters for numerical modeling

Typical values evaluated from all direct shear tests and used for numerical modeling, using equations 11-16, are presented in Table 1 below.

K_1=2000	$C_0=\tau_{adhesion}$= 0,6 MPa
K_2=15000	ϕ_{rock}= 33°
K_3=1500	ϕ_{grout}= 25°
K_4=1000	u^e=0,0001 m

Table 1 Typical parameters for numerical modeling

For the numerical modeling, an assumption is made which may not necessarily be true. The threshold value, u^e, is assumed common for the commencement for cohesive failure and development of dilatancy. However, a small relative shear displacement without dilation is possible, but from fundamental logic and the data available, 0.1 mm appears to be a reasonable, common value for both.

Results

Numerical modeling was performed with a dual joint sets at various locations in the rock mass around a tunnel at 35 m depth. The principal horizontal stresses was 3,5 MPa. Two important features can be exemplified with the UDEC plot below;

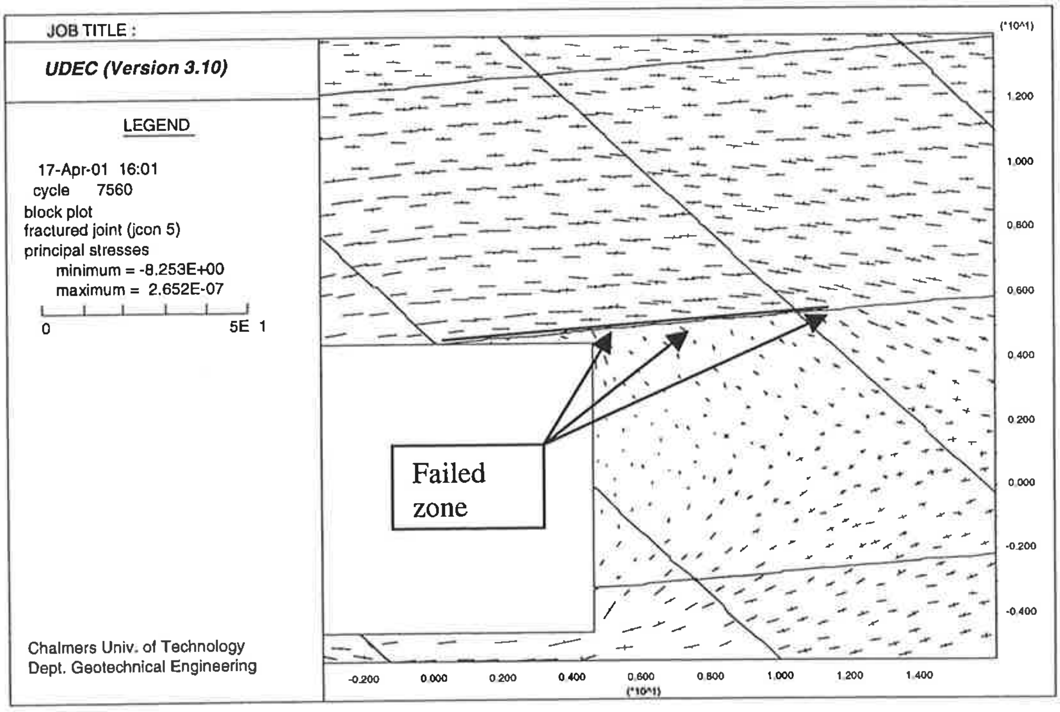

Figure 8: Possible failure in a hard rock mass, σ_h=3,5 MPa, σ_v=1.0 MPa. Failed zone = re-opened joint.

- The compressive stress field around the opening suppresses shear displacements and dilation, safeguarding the grouted water sealing
- Hydraulic failure may, under adverse conditions, propagate up to a tunnel diameter from the tunnel perimeter

This failure is caused by the principal shear stresses due to the stress re-distrubution caused by the rock excavation. In Figure 9 below, an other principal failure mechanism is shown, which may appear in sedimentary rock.

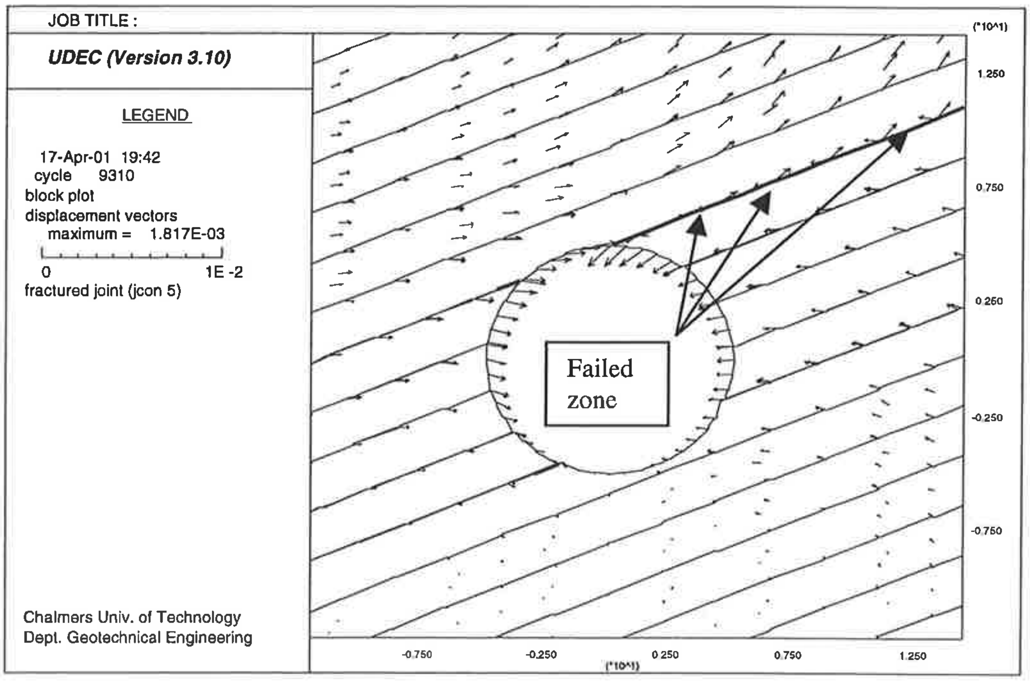

Figure 9: Possible failure in sedimentary rock, σ_h=3,5 MPa, σ_v=1.0 MPa.. Failed zone = re-opened joint.

Conclusive discussion

Performed direct shear tests on grouted rock joint replicas demonstrate that the grout seal is fairly brittle; even small deformations will initiate dilation of the rock joint which concurrently allows for water conductivity. Therefore, for successful/durable pre-grouting, the compressive stress field around the underground opening appears to be an essential prerequisite. The reinforcing effect of cement grout is very limited in tunneling situations In less successful cases, post-grouting situations emerge; it appears feasible to seek the origin of the seepage at distances at least one tunnel diameter away from the perimeter of the tunnel.

ACKNOWLEDGEMENT

The author wish to thank SweBeFo, Sweden for allocating funds for the presented research. Also, I would like to express my gratitude for the excellent co-operation with UJF at Grenoble and for me stay at this institute. The use of the direct shear device at Laboratory 3S was quite essential for this study.

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PENETRATION ABILITY FOR CEMENTITIOUS INJECTION GROUTS

Inträngningsförmåga för cementbaserade injekteringsmedel

Daniel Eklund, Vattenfall Utveckling AB/ KTH
Jan Alemo, Vattenfall Utveckling AB

Abstract

This study deals with the developing of the grouting technology with cementitious injection grouts. Cement based grouts are to prefer in several aspects, known long- term properties, harmless to the environment, easy and not harmful to handle and relatively cheep compared to chemically based injection agents. The limited penetration ability in thin cracks in rock or concrete structures is a problem for cement based grouts of today.

A common problem when the point of maximal penetration is reached is that the cement particles stick together and an impermeable plug is created. Further grouting is impossible. The ability for the grout to pass narrow sections without creating a plug is called filtration stability (FS). The filtration stability is a phenomenon, which consists of several properties in the grout. Properties which influents the FS is for example particle size distribution, W/C ratio and degree of dispersion and chemical composition.

Performed experiments in this study indicate that the particle size distribution should not contain too much fine particles less than 5 µm and not either too much coarser particles. A certain range in a particle size distribution curve seems to be optimal. The optimisation must be made from the fact that the grout wills not separate, be easy to pump and penetrate as long as possible in the structures. The interaction between these factors can be summarised in good rheology and filtration stability for the grout. The experiments indicate how that grout should be composed in terms of particle size distribution and W/C ratio. The influents of dispersion degree and additives are treated in a study later on in this project.

The project is organised as a PhD student program connected to the Royal Institute of Technology (KTH) in Stockholm and is planned to be finished in the year of 2004.

Sammanfattning

Denna studie behandlar utveckling av injekteringstekniken med cementbaserade injekteringsmedel. Cementbaserade injekteringsmedel är att föredra ur många aspekter, t.ex. kända långtidsegenskaper, miljövänligt, lätt och ofarligt att hantera och relativt billigt jämfört med kemiska injekteringsmedel. Den begränsade inträngningsförmågan i betong och bergkonstruktioner är dock i dagsläget ett problem.

Ett vanligt problem när man uppnått maximal inträngningsförmåga är att cementpartiklarna klumpar ihop sig och en ogenomtränglig plugg bildas. Vidare injektering är omöjlig. Förmågan för bruket att passera trånga sektioner utan att bilda plugg benämns filtreringsstabilitet (FS). Filtreringsstabiliteten är ett fenomen som beror av många olika bruksegenskaper. Egenskaper som påverkar FS är t.ex. kornstorleksfördelning, vct, graden av dispergering hos bruket samt dess kemiska sammansättning.

Utförda experiment i denna studie indikerar att kornstorleksfördelningen inte skall innehålla för mycket finkorniga partiklar under 5 µm och ej heller en stor andel grövre partiklar. Ett speciellt intervall för kornstoleksfördelningen verkar vara optimalt. Optimeringen måste göras med utgångspunkt från att förhindra separation, uppvisa god pumpbarhet samt inträngningsförmåga. Samverkan mellan dessa faktorer kan sammanfattas i god reologi och filtreringsstabilitet för bruket. Experimenten påvisar hur kornfraktionen samt vct kan väljas för att erhålla ett bruk med god inträngningsförmåga. Inverkan av graden av dispergering samt tillsatsmedel, behandlas i ett senare skede av projektet.

Projektet bedrivs som ett doktorandarbete kopplat till Kungliga Tekniska Högskolan (KTH) i Stockholm. Projektet beräknas vara avslutat under 2004.

1 Preface

The aims of grouting structures are often to fill voids, cracks and pores and sometimes strengthen the structure. The sealing of the structure prevents further attack on the structure. The structure consists of rock, concrete or soil. This paper deals with rock and concrete grouting, when the technique is almost the same for both. Cracks can be dead or alive. The movement in living cracks can depend on different causes, changing in temperature over the day or movement due to seasonal variation. Cracks can be either dry or wet. Depending on the demands and purpose of the grouting, construction material, type of cracks, aperture, movements and degree of moisture in the cracks, the choice of grouting agent will be made. This paper only deals with cementitious grouts.

2 Introduction

Possibilities of sealing structures are of great importance in both economical and environmental point of view. The rising demands of closeness on rock structures make that all parameters that influence the grouting results have to be investigated. The demands of closeness on the rock structure affect both the functions as achieving a dry tunnel and the influence on the surrounding environment. The costs of grouting have in certain projects been as high as the cost for the blasting and excavation of the tunnel.

To improve the technique of grouting with cement based material, it is necessary to focus on the properties of the used grout mixture. You can attack the problem in many different ways, in our research we have chosen to deal with the problem according the following way. Limiting penetration ability in thin cracks of rock or concrete can be divided into to two parts. One part is related to the transportation of particles in the crack (rheology) and the other deal with the formation of plugs in the entrance or further in the crack (filtration stability). Both these phenomena have to co-operate for a successful grouting result.

The work with developing new measuring system and material models for the grouting material, are probably some of the key issues to solve. According to today's knowledge and research in grouting material are parameters as particle size and concentration, viscosity and yield value of very great importance for the penetration ability. The use of modern measuring equipment opens new possibilities for characterising the grouting material in a more precise and accurate way. The developing of new measuring equipment would involve both equipment suited for laboratory and field use.

A common problem when the point of maximal penetration is reached for the grout is that the cement particles stick together and an impermeable plug is created. Further grouting is impossible. The ability for the grout to pass narrow sections without creating a plug is called filtration stability (FS). The filtration stability is a phenomenon, which depends of several properties of the grout. Properties which influents the FS are for example particle size distribution, W/C ratio, shape of particles and degree of dispersion and chemical composition.

The formation of the plug can either occur at the entrance or at a constriction of the crack, see figure No. 1.

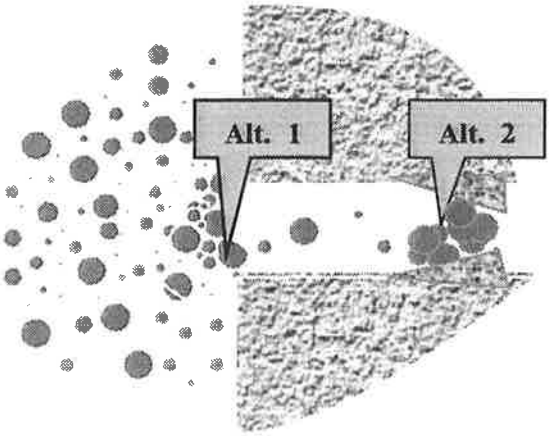

Figure 1. Formation of a plug in a crack.

The mechanism of FS is probably different when comparing grouts with low or high W/C ratio. This study is mainly focused on more or less stable grouts. With stable grout means grout which has no water separation. Stable grouts are desirable in many point of views, both practical in terms of facilitate mixing and pumping and also for receiving a resistant and a good sealing efficiency of the grouting.

What this project will receive can be summarised in:

- Map and explain the mechanisms which governs the filtration stability.
- Measuring methods for filtration stability.
- Recommendations of how a grout should be composed for good filtration stability.
- A model for the filtration stability which can be used for prediction of the spreading of the grout.
- Investigation of how the mixing equipment influents the filtration stability.

3 Laboratory experiments, Performance

The geometrical properties (particle size distribution, particle shape and W/C ratio) which influents the penetration ability are investigated by the use of inert particles. The reason of using inert particles is to avoid influences from chemical reactions and the time dependent properties of cement. The inert material that is used is crushed dolomite stone, in the fraction of 0- 30 µm. The trading name of this crushed dolomite stone powder is Myanit.

The particle size distribution is investigated by mixing three different fraction intervals of Myanit into a number of new mixes with different particle size distributions, see figure 2. The original fraction intervals are Mix 1, 2 and 3. The fraction mixes are then mixed with different amount of water before the analyse.

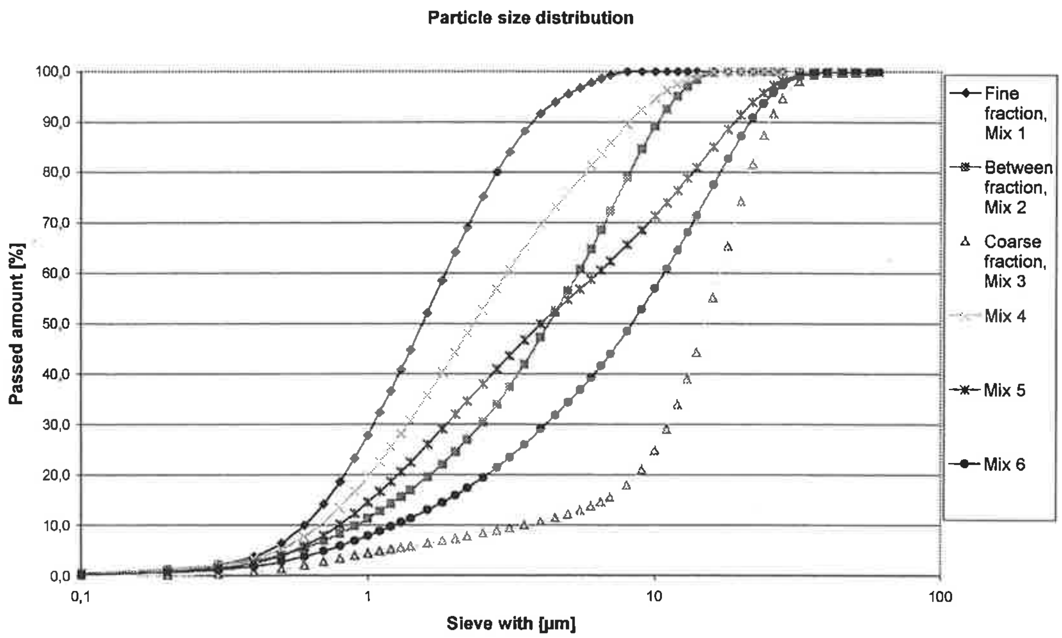

Figure 2. Particle size distribution for six different mixtures.

Filtration stability and rheology have then been measured for the different particle size distributions.

Filtration stability is quantified by a measuring device that press the mixture through a net with the quadratic mesh width of 45 μm. The pressure gradient over the filter is 30 kPa. The amount of mixtures that passes the filter is a measure of filtration stability. When a large amount is passing the filter, then the mix is said to have good filtration stability. A small amount indicates that a filter cake is being built up at the filter which after a while makes it impossible for the mixture to pass through the filter. The amount of Myanit per mixture is always 100 g and the amount of water is variable between 60 g to 140 g, depending on the W/C ratio that is investigated.

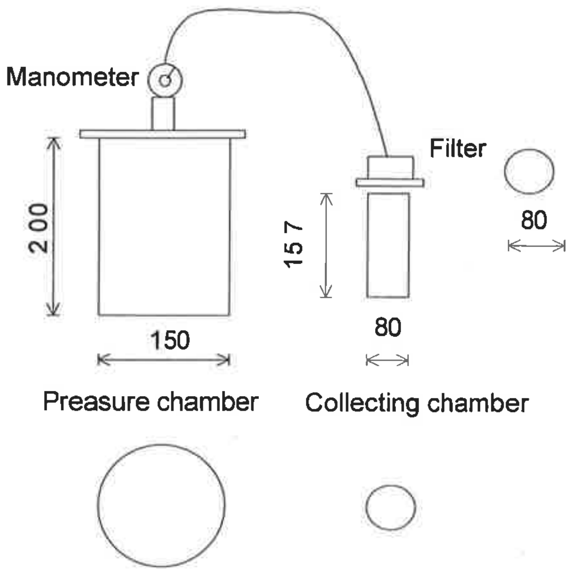

Figure 3. Measuring device for filtration stability.

Rheological parameters as viscosity and yield value according to the Bingham model have also been measured. Viscosity and yield value are measured both before and after that the mixture has passed the filter. Lower viscosity and yield value after that the mixture has passed the filter, means that some particle fractions in the mixture have stopped at the filter and some filtration has occurred. The measuring device that has been used is plate-plate geometry according to DIN 53018.

1 Introduction

This paper is an extract from research work performed for Swedish Nuclear Fuel and Waste Management Co (SKB).

A well performing injection grout should have a good flowability when injected and a distinct hardening on place. Cement based grout, which is a particle slurry, do not have these properties and they must thus on the given premises be optimised for the injection situation. This paper shortly describes the hydration system and the possibilities of cement based grout.

Cement based injection materials (grouts) are based on ordinary cement clinker optimised for building purposes. The cements are broadly divided into rapid hardening types of cement for normal buildings and slowly reacting types for heavy constructions. The major difference between ordinary cement and injection cement is that the later often is more fine-grained to give better penetration ability. In some types the cement hydration is manipulated to get early stiffening.

A well performing grout should have good penetration ability and when it is grouted it should stiffen/set rapidly. A disadvantage with Portland cements is the relatively long setting time, which is further prolonged at lower temperatures. In the investigation we have focused on two aspects, one to get an optimised grout for ordinary grouting regarding penetration, the other to find a fast setting grout for strongly water carrying (transmissive) zones. To get an optimised ordinary grout a Swedish low heat ultrafine Portland cement (Degerhamn P12/P16) was chosen. For the rapid hardening grouts Rheocem 800 and Rock-U were chosen. These cements were tested together with different types of superplasticizers, at different water to cement ratios and at different temperatures especially referring to flowability and setting time.

The work shows that to be able to utilise ordinary Portland cement for optimal grouting one must understand the cement hydration reactions and the different admixtures properly.

2 Cement reactions

Ordinary Portland cement consists of four main clinker minerals, C_3S, C_2S, C_3A and C_4AF. The most reactive of these is C_3A, which is responsible for the early setting. To retard the C_3A reactions gypsum is added to the cement clinker at grinding. The reaction rate can be quantified in a calorimeter where the released energy and heat flow can be measured. This can be seen in figure 1 and 2. During normal hydration there will be a first reaction when the cement surfaces are wetted. Following this there will be a dormant period during which the cement hydration is slow. After a certain period of time the calcium silicates start to hydrate and the concrete becomes hard. The beginning of this is the time of setting (arrows in figure 1). A higher energy release will give faster hardening. The “open time” during which a concrete can be casted or a grout injected is the dormant period plus the first period of hardening.

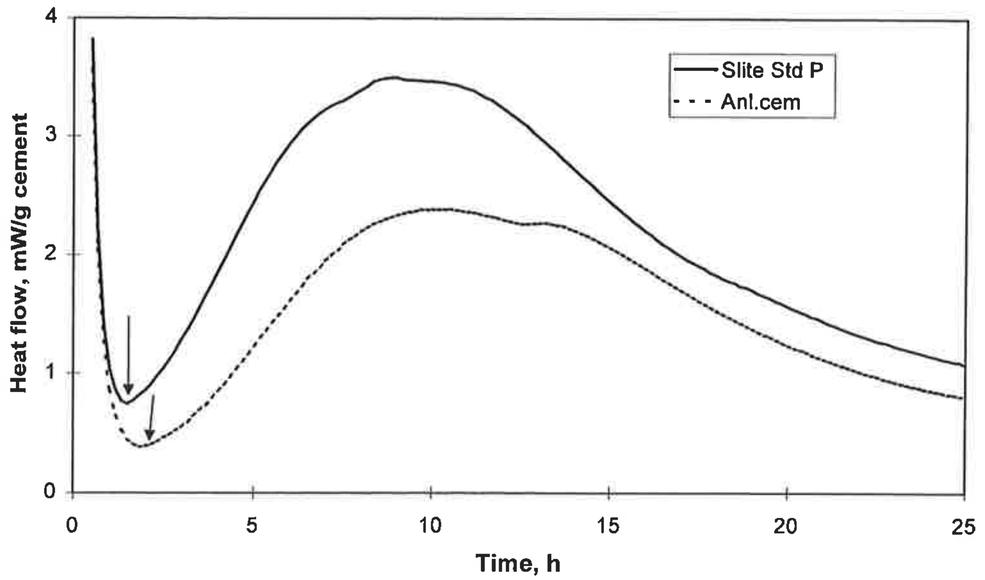

Figure 1. The heat flow for Slite Standard and Degerhamn Standard (Anl.) cements.

In figure 1 the heat evolution is shown for two ordinary Swedish cements, Slite Standard P and Degerhamn Standard P (Anl.), which is a low heat cement. The reactions start somewhat later for the Degerhamn cement (arrows). Moreover, the total energy of released heat is lower for the low heat cement, which is beneficial for heavy constructions.

In figure 2 the heat flow for the ordinary Degerhamn cement and the ultrafine Degerhamn cement is plotted. The time at which the cement reactions start is almost the same (arrow), but the rate of the reactions is much faster for the ultrafine cement. The consequence will be that although the calcium silicate reaction starts at the same time the hardening process will be faster and the "open" period shorter. It is the same for a lower water/cement ratio.

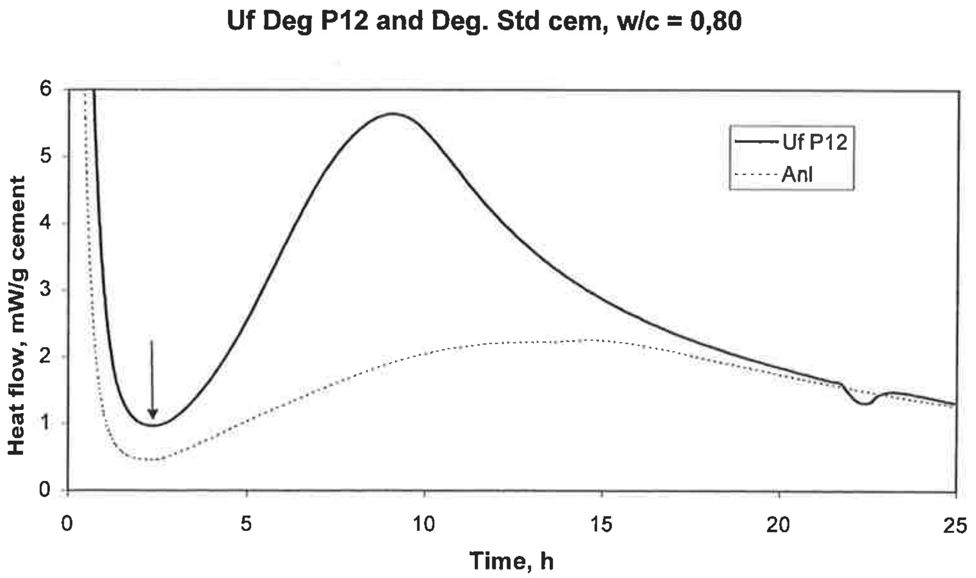

Figure 2. The heat flow for Degerhamn Standard (Anl.) and the ultrafine Degerhamn cement (P12).

The figures above show the normal hydration system. If the sequence of the hydration processes is changed the pattern will be different, which is shown in Fig. 5 and 6.

CEMENT BASED INJECTION GROUTS – SETTING AND CEMENT REACTIONS

Cementbaserade injekteringsmedel – bindning och cementreaktioner

Leif Fjällberg, Cement och Betong Institutet
Björn Lagerblad, Cement och Betong Institutet

Abstract

Injection grout is based on cement optimised for building purposes. The main difference is that some of them are fine-grained for better penetration ability. This paper, which is an abstract from a SKB-project, shows how cements can and are manipulated to give certain properties to the injection grout.

The reaction rate and setting depends on the type of cement clinker. Normal cement is composed in such a way that it after mixing has a dormant period during which concrete can be casted (or grout injected). The length of this "open" period depends on the type of cement clinker. Based on calorimetric measurements we found that the fine-grinding and water cement ratio did not affect the dormant period (initial set) but the speed of reaction (strength development/final set). Thus like with accelerators one must divide between initial and final set. The effect of these principally different mechanisms will however be a shorter "open time". Different accelerators can be used to either change the setting time or hardening process. They must be carefully optimised. Mineral additives do not alter the setting but will affect the strength development.

The superplasticizers (SP) extend both set and strength development. Thus the type of cement and SP must be optimised carefully to get a good balance between flowability and setting time.

The finely ground rapid cements, with high contents of aluminates (C_3A), gave very varying results where the cement in different batches had stiffening times from 30 minutes to more than one day. Moreover, the superplasticizer did not work as expected. Investigations showed that this was due to irregularities in the hydration system, which sometimes gave early false setting and a proper hydration much later. This is probably a result of the high fineness of the rapid hardening cements, which affects the gypsum/aluminate system.

If the false setting can be controlled this can and is used in some injection cements like Rock-U. In this product a controlled false setting give a first stiffening to stop the water flow and later proper cement hydration gives a hardened product.

The experiments show that the results can be very varying depending on the type of cement, the superplasticizer, the water to cement ratio, temperature etc. This demands a careful analysis of the rock, the situation and a careful optimisation of the mixture composition. If a late setting time (about 10 h) is not a disadvantage the low heat cements are to prefer. If shorter setting times are required an accelerator or the rapid hardening cements can be used, but the control must be very careful.

Sammanfattning

Denna artikel baseras på undersökningsmaterial producerat och sammanställt för Svensk Kärnbränslehantering AB (SKB)

Cementbaserade injekteringsmaterial är baserade på vanlig cementklinker som är optimerad för byggnadsändamål. Cementen kan grovt delas in i snabbhärdande cement för husbyggnad och långsamt härdande cement för anläggningsbyggande. Den största skillnaden mellan vanligt cement och injekteringscement är att det senare cementet är mera finmalt, för att ge en bättre inträngningsförmåga. I vissa typer av cement utnyttjas cementreaktionerna för att ge en snabb tillstyvnad.

Ett bra injekteringsbruk bör ha god flytförmåga då det injekteras och en distinkt tillstyvnad efter injekteringen. Cementbaserade bruk, som bestått av en partikelslurry, har inte dessa egenskaper utan de måste optimeras under givna premisser för injekteringssituationen. Denna artikel beskriver kort hydratationssystemet och möjligheterna för cementbaserade bruk.

De finmalda cementen är normalt baserade på långsamt härdande klinker. Den större finheten kräver relativt sett mera vatten för att ge bra flytförmåga, vilket i sin tur ger en större porositet och lägre hållfasthet. Detta kan kompenseras med nya typer av superplasticerare, vilka möjliggör lägre vct utan att försämra flytförmågan. Dessa superplasticerare har en tendens att retardera cementreaktionerna mera än de traditionella typerna, vilket gör det nödvändigt att omsorgsfullt optimera bruket för att få en god balans mellan flytförmåga och tillstyvnande. Den större finheten förkortar inte bindetiden utan ökar reaktionshastigheten vid hårdnandet. Inblandning av andra finmalda produkter ger samma resultat; de påverkar inte reologin eller cementreaktionen nämnvärt. Olika acceleratorer kan användas för att förkorta bindetiden, men de måste noggrant optimeras.

Försöken med det snabbhärdande finmalda cementet Rheocem 800 gav mycket varierande resultat, där cement från olika leveranser hade bindetider från 30 minuter till över ett dygn. Inte heller superplasticerarna fungerade normalt med detta cement. Undersökningen visade att detta berodde på variationer i hydratationssystemet, vilket ibland gav en tidig falsk bindning åtföljd av de egentliga cementreaktionerna mycket senare. Detta är troligen ett resultat av de snabbhärdande cementens stora finhet, vilken påverkar gips-aluminatsystemet.

Om den falska bindningen kan kontrolleras går den att utnyttjas, vilket man gör med Rock-U. Denna produkt ger en kontrollerad falsk bindning och tillstyvnad som stoppar vattenflödet och de senare egentliga cementreaktionerna ger den slutliga hårdnade produkten.

Försöken visar att resultatet kan variera starkt beroende på typ av cement, superplasticerare, vct, temperatur osv. Detta kräver en omsorgsfull analys av berget, situationen och en noggrann optimering av blandningens sammansättning. Om en sen bindetid (ca 10 timmar) inte är någon nackdel är det långsamt härdande cementet att föredra. Om kortare bindetider krävs kan en accelerator eller ett snabbhärdande cement användas, men kontrollen måste vara mycket noggrann.

5 Conclusions and discussion

This study indicates that the particle size distribution and W/C ratio both strongly influence the rheology and filtration stability. The correlation between the amount of fine or coarse particles and rheology seems to be quit clear. More fine particles generate a raising yield value and viscousity, compered to a mixture with coarser particles. A low W/C ratio means that the distances between particles in the mixture are smaller and a greater interaction is obtained and therefore showing an increasing yield value and viscousity.

The work of finding key correlations between particle size distribution, W/C ratio and the passing amount of mixture through the filter seems to be quit difficult and is not showed yet. The result show that a large amount of fine particles or coarse particles gives low filtration stability while a particle distribution in between gives a much higher filtration stability. The work of finding correlations is much a work of finding representative parameters to compare. The describing of particle size distribution in terms of for example d_{10}, d_{50} and d_{90} is one way of doing it. Using the W/C ratio as a parameter for describing particle concentration and in some sense even particle packing, can even be a question mark. There is probably the volumetric concentration and not the weight concentration of particles, that is of the biggest interest when the grout formation a plug.

It is reasonable to believe that particle packing is of great importance for the formation of the plug. Particle packing is then an important factor for both understanding the geometrical- and chemical phenomena when plug formation occurs. It is likely to stipulate according to performed experiments, that the plug is more easily formed if the mixture contain a certain particle size distribution (geometrically phenomena). Particle packing in terms of porosity or surface distance between particles, could be a better correlation parameter than d_{10} d_{50} or d_{90} against the passing amount of mixture through the filter. In a chemical point of view, particle packing is important for the mixture behaviour when adding for example superplasticisers. Superplasticisers adhere to the surface of the particles and act via repulsing forces between the particles. The effect of the additive is dependent of the distance between the particles. Some good model that describes particle packing with n number of particle sizes and in a specific volume is needed.

The formation of the plug in relation to the properties of the crack is an important parameter to investigate. The net mesh is one way of simulate a constriction or narrow crack. The formation of a filter cake is probably related to both the design of the entrance and the type of crack (single crack or a pattern of neighbouring cracks). The net mesh can be said to represent the pattern of neighbouring cracks with sharp entrance. 'The plug formation has also to be investigated when using a model for a single crack with a sharp and rounded entrance.

When you try to explain or describe the reason why the grout sometimes creates a plug, is it important to be aware of the stochastically behaviour of the phenomena. The stochastically variation can either depend on variation in the material or some parameter, which varies stochastically, or a combination of both. In this study we are going to found your statements on both empirical correlations from experimental tests and theoretical assumptions of how different parameters interact when a plug is initialised. It should be noticed that performed experiments just represents a few measuring values. The finally evaluation of the correlation between different parameters will be founded on more measuring values.

Density measurements are performed to analyse the change in W/C ratio between the original mixture composition and the filtrated mixture. The relation between measured density and W/C ratio is:

$$W/C\ ratio = \frac{\rho_w \cdot (\rho_m - \rho_c)}{\rho_c \cdot (\rho_w - \rho_m)}$$

ρ_w = *Density, water*

ρ_m = *Density, mixture*

ρ_c = *Compact* density, Myanit

W/C ratio = *Water Cement ratio*

Figure 4. Formula for calculation of W/C ratio from density measurements.

4 Laboratory experiments, Results

Parameters that have been varied in the experiments are particle size distribution and the W/C ratio. A large variation in the amount of passing the filter was observed, when different particle size distributions were examined. Some combinations were excluded because that just more or less stable mixtures, generates reliable and reproducible measuring data. For example with a mixture with a large part of course particles and also a high W/C ratio, the mixture will strongly separate during measurement. The biggest passed amount for all the tested W/C ratios (0,6, 0,7, 0,8 and 1,4) were noticed for mixture "Between fraction, Mix 2", see table 1.

Table 1. Passed amount of mixture through the filter (45 µm).

Passed amount [%]				
W/C ratio	**0,6**	**0,7**	**0,8**	**1,4**
Mix nr.				
1		0	63	
2	**49**	**84**	**86**	**99**
3	10	10	8	
4		9	16	
5	25	50	81	97
6		24	14	

The rheological measurements also show a great difference for different combinations of particle size distributions and W/C ratios. The yield value and viscosity is calculated by applying the Bingham model to the measuring data.

Table 2. Yield value and viscousity for the original mixture.

Yield value					Viscousity			
	[Pa]				[Pas]			
W/C ratio	**0,6**	**0,7**	**0,8**	**1,4**	**0,6**	**0,7**	**0,8**	**1,4**
Mix nr.								
1		46,0	17,9			0,531	0,413	
2	9,6	1,8	1,6	0,2	0,222	0,096	0,062	0,010
3	2,9	1,1	0,4		0,004	0,002	0,002	
4		0,6	0,2			0,025	0,023	
5	35,0	7,8	6,8	1,3	0,571	0,149	0,220	0,049
6		1,7	1,1			0,070	0,091	

The purpose of the density measurements is to illustrate how the filtration influents the mixture. Mixtures with a disadvantageous particle size distribution are strongly affected by the filtration, for example mixture No. 3, 4 and 6. A higher W/C ratio after filtration compared with the original W/C ratio, shows that these mixtures contain a lower concentration of particles.

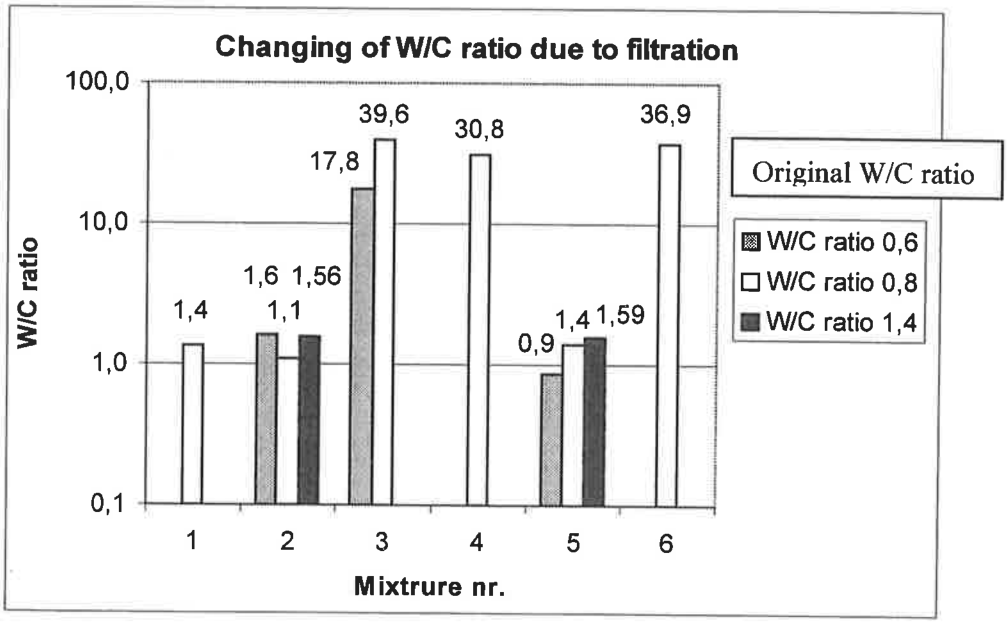

Figure 5. Difference in W/C ratio before and after filtration at 45 μm mesh width.

3 Effect of superplasticizer

With superplasticizer the reactions are retarded with several hours. This can be seen in figure 3, where the heat flow of mixes with superplasticizer added to the pure system is shown. The effect is mainly that the dormant period is much longer. With 0,4 % of the superplasticiser the proper cement reactions start to accelerate after about 10 hours and with 0,7 % after about 15 hours. Thus it is very important not to overdose the superplasticizer.

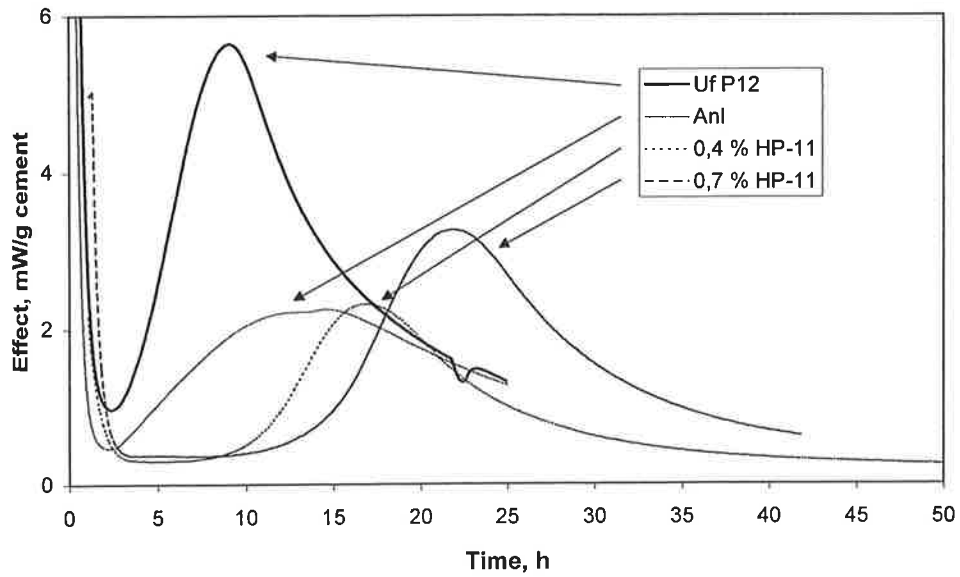

Figure 3. The heat flow for Degerhamn Standard (Anl.) and the ultrafine Degerhamn cement (P12) without superplasticizer and with 0,4 % and 0,7 % of superplasticiser (HP-11).

4 Effect of temperature

When the temperature decreases the cement reactions are retarded and the flowability is decreased. The temperature in the mixes has been measured continuously during the hydration and the time at which the cement reactions start and the temperature begin to increase is called T_{start} (dormant period). The flowability was measured with a little funnel and is given as the time for 100 ml of the paste to pass through the funnel. In table 1 the effect of temperature can be seen. Two different superplasticizers were used, SSP-150 and Glenium-51. When the temperature is decreased from 25°C to 8°C the dosage of superplasticizer is increased. The flowability is however somewhat decreased and the cement reactions delayed with 1-3 hours. This shows that the superplasticizer must be optimised for the proper temperature regarding both temperatures at mixing and in the rock.

Vct = 0,80	25°C			8°C		
	Dos.,%	T_{start} , h	Time,s	Dos.,%	T_{start} , h	Time, s
SSP-150	0,20	8,3	24	0,40	11,2	25
Glenium-51	0,20	14,8	19	0,40	15,9	25

Table 1. The dosage, time to T_{start} and flow time for the ultrafine cement P16 at 25°C and for the ultrafine cement P12 at 8°C.

5 Effect of additive

In some cases it may be of interest to decrease the cement content in the mixture e.g. when a lower pH is necessary. Therefore two finely ground minerals were intermixed with the cement to see how the flowability and the rate of the cement reactions are affected. The minerals chosen were Mikrodur and M-6000.

Mikrodur is a finely ground blast furnace slag and M-6000 is a silicate mineral, cristobalite. The ratio between cement and additive in the mixtures was 50/50. The time T_{start} when the cement reactions start is almost unaffected of the additive, compared to the mixture with pure Portland cement (Fig. 4) . The reactions are however delayed with increasing amounts of superplasticizer, SSP-150.

The results show that when you add an inert material or a slowly reacting cementitious material like Microdur it is the grain size and distribution of the material that rules the rheology, while the cement reactions rule the setting. This shows that it is possible to mix the Portland cement with other fine-grained products without loosing properties.

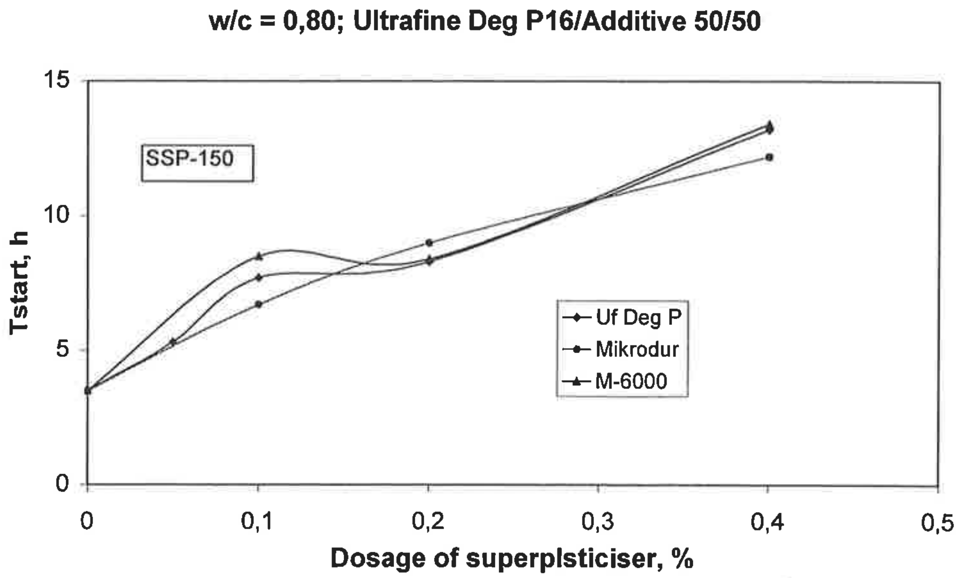

Figure 4. The time to T_{start} with two types of additives.

6 Accelerators

One of the most effective accelerators of the cement reactions is $CaCl_2$. It affects both the flowability and the cement reactions. With increasing amounts of accelerator the flowability of the mixture is decreased much before the onset of cement reactions. In table 2 the effect of 2 % $CaCl_2$ of the cement weight in the mixture, on the flowability at different times from mixing start and on T_{start} is shown.

Accelerator $CaCl_2$	Dos., %	Time, s at 15 min	Time, s at 30 min	Time, s at 45 min	Time, s at 60 min	T_{start}, h
	0	38	33	31	31	12,7
	2,0	35	300	174	141	9,3

Table 2. The flow time (Time, s) in seconds at different times from mixing start and T_{start} (Dormant period) in hours, for ultrafine Deg P16 without or with accelerator. The mixes contains a superplasticizer (0.30 % SSP-150).

With 2 % $CaCl_2$ the flowability at 15 minutes is almost the same as without accelerator, but at later ages the flowability decreases considerably. The time when the cement reactions start is however shortened with more than 3 hours. In practice one must consider if the short stiffening time of the paste is enough to stop the water flow or if we must wait to the proper cement reactions.

7 Rapid hardening cements

7.1 Rheocem 800

Rheocem 800 is a finely ground Portland cement for grouting purposes with almost the same composition as the Swedish Slite Standard cement. The problem by grinding this kind of cement so finely is the fast C_3A reactions. The aluminates reacts with the sulphates and form ettringite. If the aluminates are released too fast the fast formation of large amounts of ettringite may give a false setting. This was also the case in the investigation, e.g. in one batch of Rheocem 800 the cement paste stiffened after about 5 hours. This false setting can be seen as a sharp peak in the heat flow diagram, figure 5. After about 40 hours the normal cement reactions started.

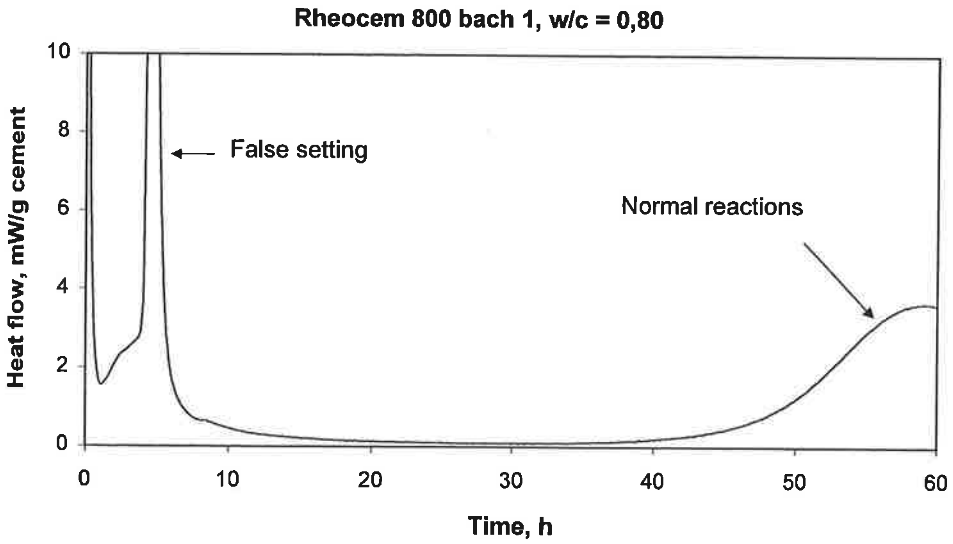

Figure 5. The heat flow for Rheocem 800.

By X-ray diffraction (XRD) we can identify individual minerals by their diffraction peaks. By measuring the peak intensity we can quantify the amount of an individual mineral. As both ettringite and the cement clinker are minerals we can by XRD at a certain stage of hydration both quantify the amount of ettringite and portlandite formed and the amount of cement clinker minerals consumed (degree of hydration α).

Cement paste prepared of Rheocem 800 with w/c = 0,80, which stiffened in about 30 minutes, is compared with an ordinary coarse grained Portland cement, Slite Standard in table 3.

Std P Slite					
Age, h	Ettringite/Int.	C_3A/α	C_3S/α	$(C_3S+ C_3S)/\alpha$	$Ca(OH)_2$/Int.
1	81	4	9,5	10	31
6	135	18	54	29	384
24	177	61	79	79	702
Rheocem 800					
Age, h	Ettringite/Int.	C_3A/α	C_3S/α	$(C_3S+ C_3S)/\alpha$	$Ca(OH)_2$/Int.
1	404	37	0	8	53

Table 3. XRD-analysis at different times of hydration. α = degree of hydration in %, Int is intensity of the peak.

In the cement paste made of Rheocem 800 much more ettringite had developed in one hour, than what had developed during one day in the ordinary Portland cement, with almost the same chemical composition. The C_3A has also reacted much faster during the first hour in the paste made of Rheocem 800 than in the other paste, α = 37 % and 4 % respectively. The clinker components C_3S and C_2S have hardly at all reacted in neither

cement at that time (α = 0-10 %). Why the C_3A reacts so fast in Rheocem 800 is probably due to the fact that it is so finely grained.
This means that one must be very careful about fine-grained fast cements. Today Rheocem 800 contains a retarder to prevent false setting. The false setting may, however, be used to stop flow and later the proper hardening will give strength. This requires a strict control and knowledge of the process.

7.2 Rock-U

Rock-U is an ordinary Portland cement with very low sulphate contents. This will not give a false setting but a flash set. The false setting gives ettringite while the flash setting gives calcium aluminate hydrates. Normal cement contains gypsum (calcium sulphate) to prevent flash set.

This reaction is controlled by a component B which contains a plasticizer (lignosulfonate which contains sulphate) and a retarder (citric acid).

XRD-analysis shows that no ettringite has formed during the first day of hydration, instead calcium aluminate hydrate has formed. This early setting can be seen as a sharp peak in the heat flow diagram, figure 6.

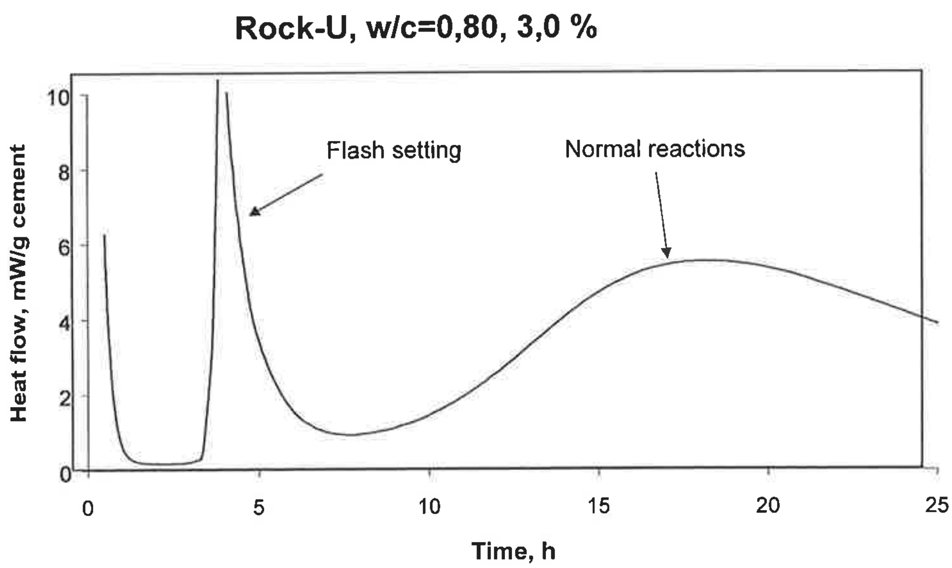

Figure 6. The heat flow for Rock-U with 3 % of Compound B.

With increasing amounts of Compound B the setting time is delayed, which means that the sharp peak in the heat flow diagram appears later. Even with compound B the flash setting occurs. After the flash setting the normal cement reactions take place, which can be seen in the heat flow diagram.

8 Conclusions

Cement based injection grouts are finely ground Portland cements, which is necessary to get a good penetration ability. The greater fineness demands relatively more water to achieve good flowability, which in turn gives a higher porosity and lower strength. With new types of superplasticizers the water/cement ratio can be decreased without impairing the flowability. With increasing amount of superplasticizer the cement reactions however are retarded.

Finely ground Portland cement with a low C_3A content have a good flowability but the setting time is very long and is even longer at lower temperatures. By using accelerators the setting time can be shortened with several hours, but the flowability of the cement slurry is decreased with increasing amounts of accelerator.

The problem with finely ground Portland cements with higher C_3A content is the rapid C_3A reactions. These reactions give a false setting, which means that the slurry may have stiffened within half an hour. It is a great problem for the injection work if the slurry stiffens in the mixing bowl, pipes etc.

One possibility to work with this kind of cement is to use a retarder, which controls the setting time.

The experiments show that the results can be very varying depending on the type of cement, the superplasticizer, the water to cement ratio, temperature etc. This demands a careful analysis of the rock, the situation and a careful optimisation of the mixture composition. If a late setting time (about 10 h) is not a disadvantage the low heat cements are to prefer. If shorter setting times are required the rapid hardening cements can be used, but the control must be very careful.

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MICROCEMENT - PENETRATION VERSUS PARTICLE SIZE AND TIME CONTROL

Mikrocement – Inträngning kontra kornstorlek och tidsstyrning

Staffan Hjertström, Cementa AB

Abstract

Penetration versus particle size.
Results from NES testing shows that the capacity of injection mortars to penetrate finer rock fissures has no direct link with the maximum particle size of the microcement. The cement brand Injektering 30, with a maximum particle size of 30 um has a better penetration capacity in 75 um fissures than various microcements with particle size, less than 16 um.
Penetration versus time control
Results from filtration stability testing shows that time control of injection grouts stands normally in sharp contradiction to penetration capacity. What you gain with accelerated injection grout you easily lose in poorer penetration properties. There is nevertheless concept that makes it possible to combine time control with god penetration-properties. The newly developed SetControl concept is designed for this purpose.

Sammanfattning

Inträngning kontra partikelstorlek
Det finns inget tydligt samband mellan injekteringsbruks förmåga att tränga in i finare bergsprickor och maximal kornstorlek hos olika mikrocement. Injektering 30 med en maximal kornstorlek på 30 um, har en bättre inträngningsförmåga i spalter på 75 um än många mikrocement med kornstorlek på 6 – 16 um.
Inträngning kontra tidsstyrning
Tidsstyrning av injekteringsbruk står normalt i skarpt motsatsförhållande till brukets inträngningsförmåga. Det man vinner med accelererade bruk förloras lätt i sämre inträngning och täthet. Med det ny utvecklade SetControl konceptet kan man förena god inträngning med tidsstyrning.

Microcement – Penetration versus particle size and time control

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Background

Impermeability requirements in tunnel construction have increased considerably during recent years. Normal requirements, in Scandinavia today, are for completely dry tunnels. Such demands create a need to be able to dimension and to predetermine injection method as well as functional characteristics, which are able to systematically seal the rock structure to provide a completely dry tunnel.

Traditionally, rock injection is a skilled trade, in many ways depending much on the personal experience of the individual. The actual task of injecting has, to a considerable extent, had low status and has had to be accommodated in the time gaps between blasting and rock removal. This has contributed to injection results of considerable variation with regard to achievement of permeability and leakage of water. To predetermine a certain level of leakage under these conditions has been difficult.

Rapid development of injection techniques

The partially, strongly fissured and difficult to seal rock structures of Hallandsås in Sweden and Gardemoen in Norway, in combination with very stringent impermeability requirements have subjected contractors and their injection methods to tough trials. Injection experts and material manufacturers from around the world have been involved in these projects. These, difficult to master sealing works have had the added benefit of advancing the development of injection techniques. At the same time, we have during past years seen a considerable tightening of impermeability requirements in normal tunnelling works throughout the world, especially in Scandinavia. These difficult to seal the rock, combined with increased demands on impermeability have clearly shown the need for testing methods which describe the capacity of the injection mortar to penetrate into and to seal fine fissures.

Dry tunnel

According to knowledge available today, it is estimated that fissures down to 75-100 um need to be sealed to fulfil the requirements of a completely dry tunnel. (References; Martin Brantberger 2000. Metodik vid förinjektering I uppsprucket berg. 2.5.2 Tätning av sprickor. Licentiate thesis 2056, KTH) In some extreme cases, depending on rock structure, ground water pressures and interconnection of fine fissures, even finer fissures may need to be penetrated to limit water inflows to acceptable levels and/or to strengthen the ground to required levels.

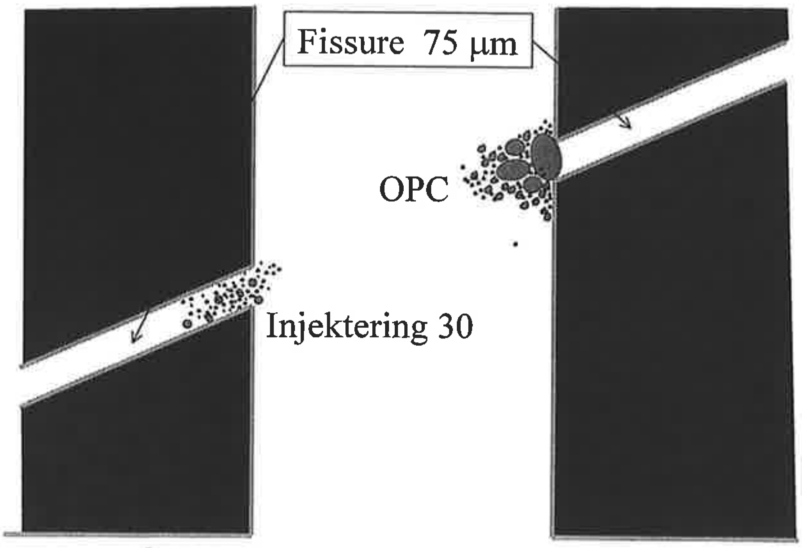

Fig.1. OPC has a normal maximum grain size (Dmax) of approx.120 um and can for reasons of geometry not seal fissures of 75-100 um. Injektering 30 has a maximum grain size of 30 um. As a thumb rule the maximum particle size of the cement should not be greater than one third to one fifth the fissure size that you want to penetrate.

Evaluation of injection mortars

Technical development is accelerating. It is becoming an increasingly complex task to develop systems and products which stand up to the impermeability demands of today as well as the future, at the same time satisfying the contractor's demands for productivity. There has been a lack of standardised and generally accepted testing methods for evaluation of injection mortars together with their penetration and sealing properties. This has made it difficult to work with product development. It was in this scenario that the testing method NES was evolved about 3 years ago.

The NES method.

The ideas behind the testing method originally come from Magnus Nilsson at Skanska. These have been further developed to the NES method by Tor Eriksson and Paul Sandberg at Cementa. This has been used to evaluate mortar in Hallandsås and several other projects. The NES method is similar to the actual injection procedure in that injection mortar at excess pressure is squeezed through gaps of the required width. The flow of material through the gap is measured continuously by continually weighing the out flowing mortar. Flow over time is a measure of the penetration capacity of the mortar. In the account of NES tests that follows, the following preconditions have been applied. Cement mortar of water/cement ratio 1.0 under a pressure of 20 bars is forced through a steel flange with gap widths of 75 and 100 um, respectively. Mixing has been carried out using Ultraturrax spinning at 20,000 r.p.m. For 2 minutes. Microcement from 5 different manufacturers with different Dmax. Types of additive and dosage according to the manufacturer's recommendations.

Inappropriate to only specify maximum size of particles and the Blaine value
Traditionally, penetration and sealing properties of cement based injection mortars have generally been considered to depend on the maximum particle size of the cement and its specific area, as measured by the Blaine method. As a result of extensive development work and tests, we have been able to establish that this is not the case. There are several examples of commercial microcements which have very poor injection properties and which are not able to penetrate into the largest fissures, in spite of a maximum grain size of as little as 6 - 16 um. These microcements have substantially inferior penetration capacity than considerably coarser cements such as Cementas Injektering 30 with a maximum particle size of 30 um. How come? Clear-cut answers are probably not available today. However, we must not forget that cement is not an inert but a reactive material, which together with water, starts up a number of chemical processes. Cementa AB's extensive development work during recent years shows that factors such as the chemical and physical composition of the cement as well as type of manufacture (burning, grinding, etc.) are of importance. Also the make-up and preparation of the injection mortar are of decisive importance to the ability of injection mortars to penetrate fine fissures and to seal rock structure.

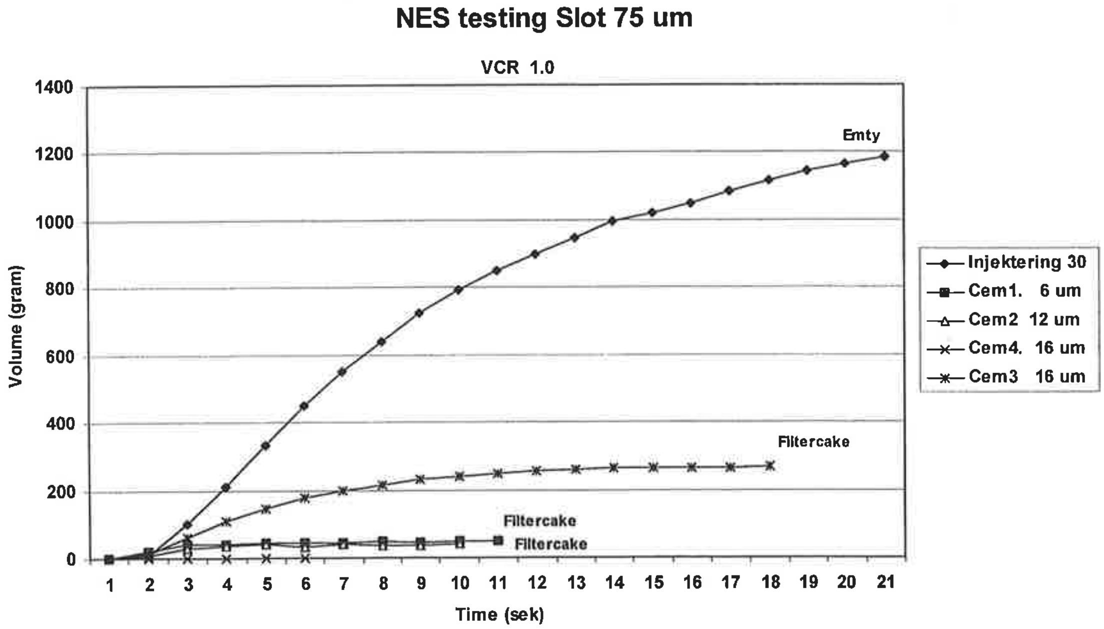

Fig. 2. NES test shows that Dmax is an unreliable measure of penetration capacity. The best result was obtained using Injektering 30 and the worst with Cement Cem 4, Cem 2 and Cem 1 with a Dmax of 16, 12 and 6 um. These finely ground cements did not actually penetrate the slot, but immediately formed a filter cake.

The water/cement ratio is an important parameter for penetration. A water/cement value of 1.0 can be too low for extremely fine ground cement with a maximum particle size of 6 and 12 um. At a higher water/cement value, these mortars should have better possibilities to penetrate a 75 um slot.

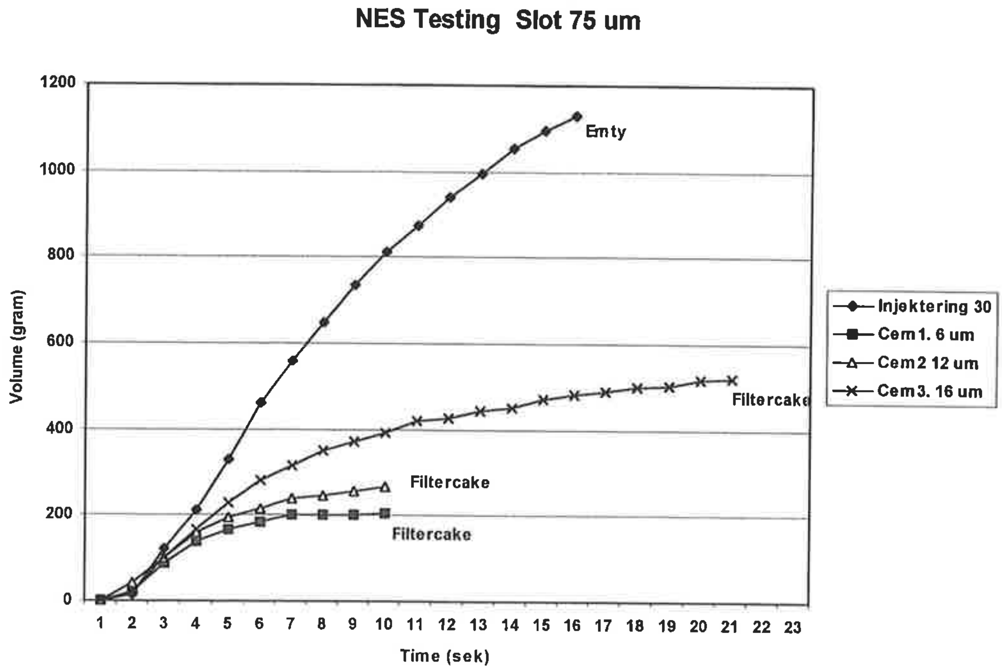

Fig. 3. NES tests show that better results were achieved for the extremely fine ground cements but Injektering 30 still gave the best result. These finely ground cements had poor penetration capacity in the 75 um slot and, fairly soon, formed a filter cake.

Similar results in field investigations

Similar results have been achieved in field investigations in rock tunnels at the South Link project in Stockholm. The conclusions was; "Injektering 30 should be in use for tight rock mass situations and if ordinary grouting equipment are used. The tested Microcement do not perform better." (Freely translated)

Reference; "Injekteringsförsök vid södra länkens bergtunnlar" Thomas Dalmalm m.fl. 2000. Rapport 3075. Royal institute of Technology. Stockholm.

Cement modification

Cementa AB:s product development indicates that different Cement parameter have a significant influence on the penetration-properties of the injection grout.

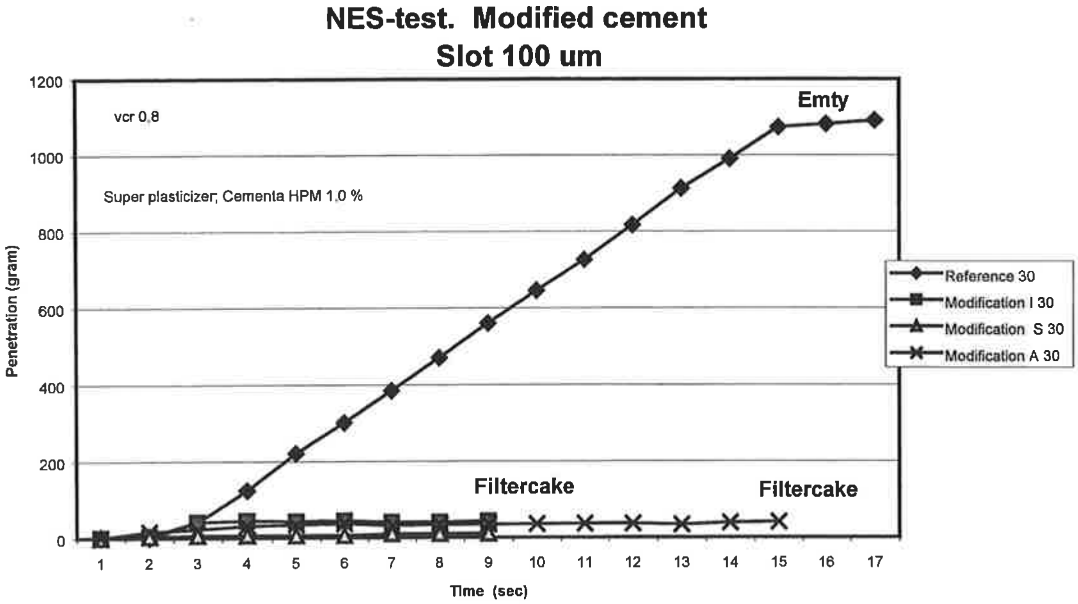

Fig. 3. Modification of material parameter has significant impact on the penetration capacity measured with the NES apparatus. Four microcement modifications with maximum particle size of 32 um were tested.

Pressure

One parameter, which has been discussed, is to what extent pressure is of importance for the penetration of the injection mortar into the rock fissures. From the NES tests, it is found that both the quantity of injection mortar as well as the speed of penetration increases with raised pressure. These findings are valid for mortar based on Injektering 30.

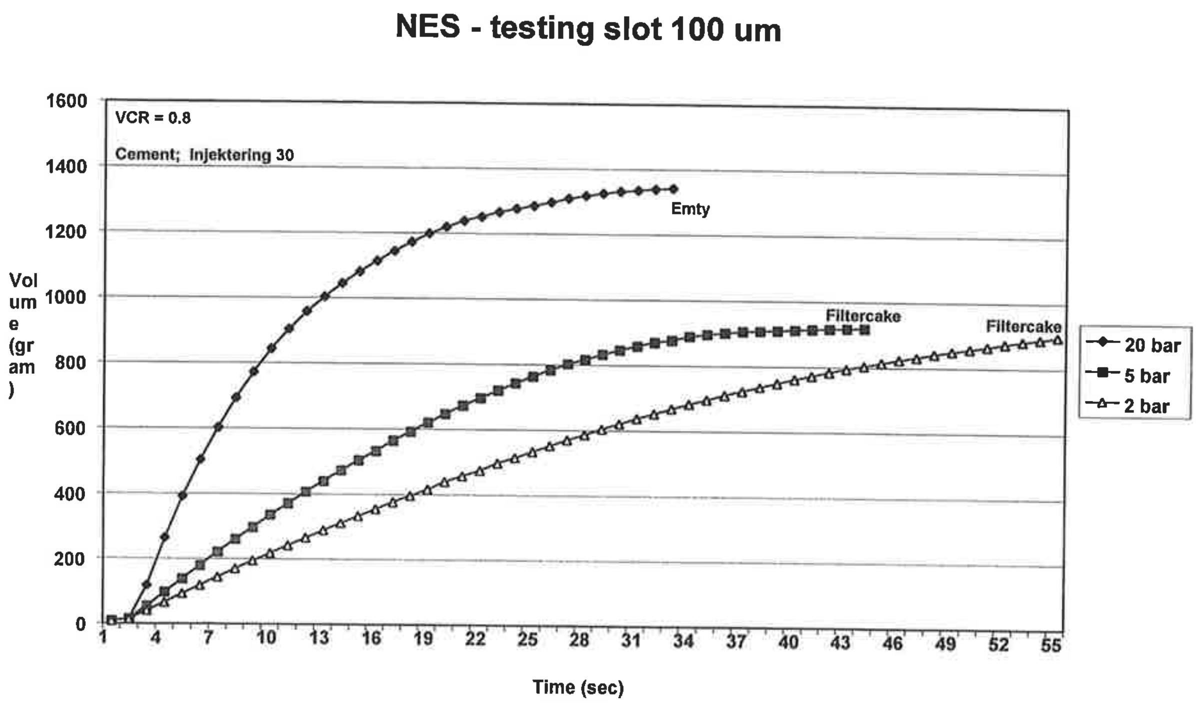

Fig. 4. Relationship between injection pressure and penetration sequence for injection mortar based on Injektering 30.

Time control versus penetration

Time control of injection grouts stands normally in sharp contradiction to penetration capacity of the grout. What you gain when you accelerate the injection grout you easily lose in poorer penetration. It is not possible to renounce the best possible penetration-properties with today's very stringent impermeability requirements of totally dry tunnel.

Injection grout with time control and retained best possible penetration.

Accelerating of the injection grout will normally influence negative to the penetration properties. Cementa AB has under a longer time period worked with this problem. The work has leaded to a time control concept called SetControl. With SetControl it is possible to combine time control with god penetration-properties.

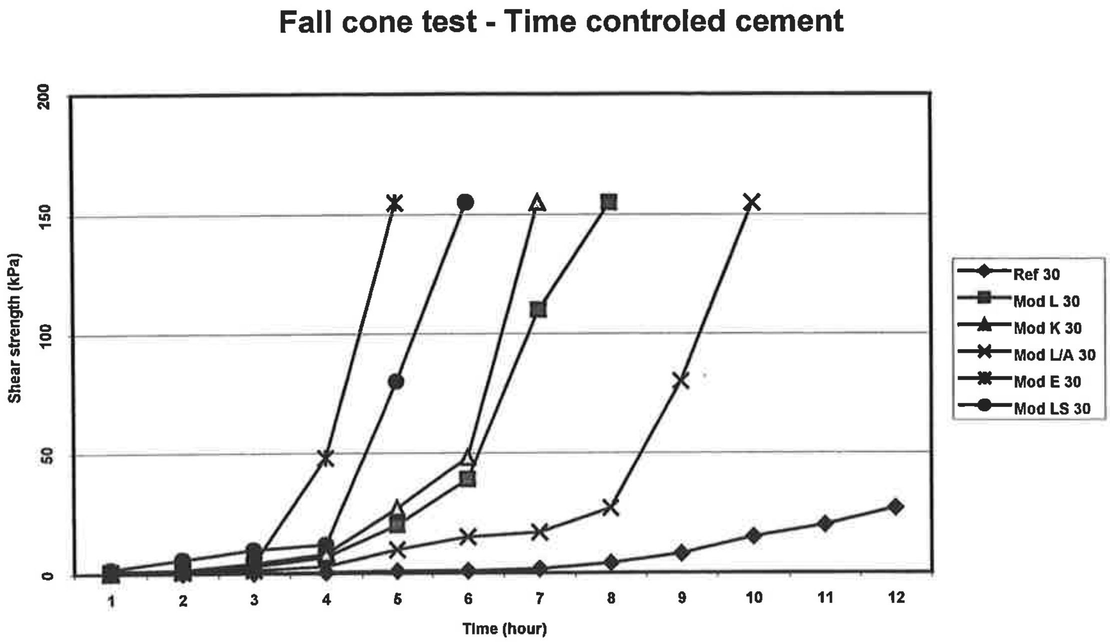

Fig 5a. To control the setting time of the microcements is quiet easy to obtain. In Fig 5a and b is six different modifications of a microcement showed.

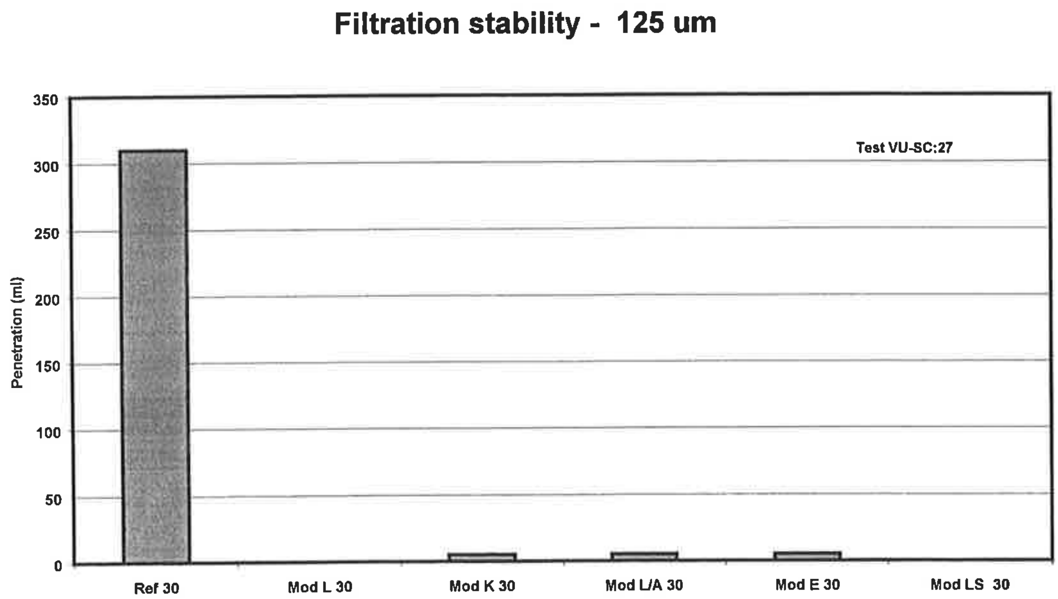

Fig 5b. It is only the cement Ref 30 with a normal setting time that is showing up an acceptable filtration value with the 125 um metal wire cloths. All quick setting – cement modifications show up poor filtration stability. The result confirms the negative aspect with a quick setting-mikrocement. Accelerating of the injection grout

give normally a poorer penetration capacity. The filtration stability test is a standard test (VU-SC:27) in Sweden. The grout should pass a woven metal wire cloth of 125,75 or 45 um.

SetControl

With the SetControl concept is it possible to combine time control with god penetration-properties.

SetControl consist of an additive in combination with the microcement Injektering 30 or Ultrafin 16. The concept gives you an injection grout with very good penetration capacity and simultaneously possibility to control the setting time.

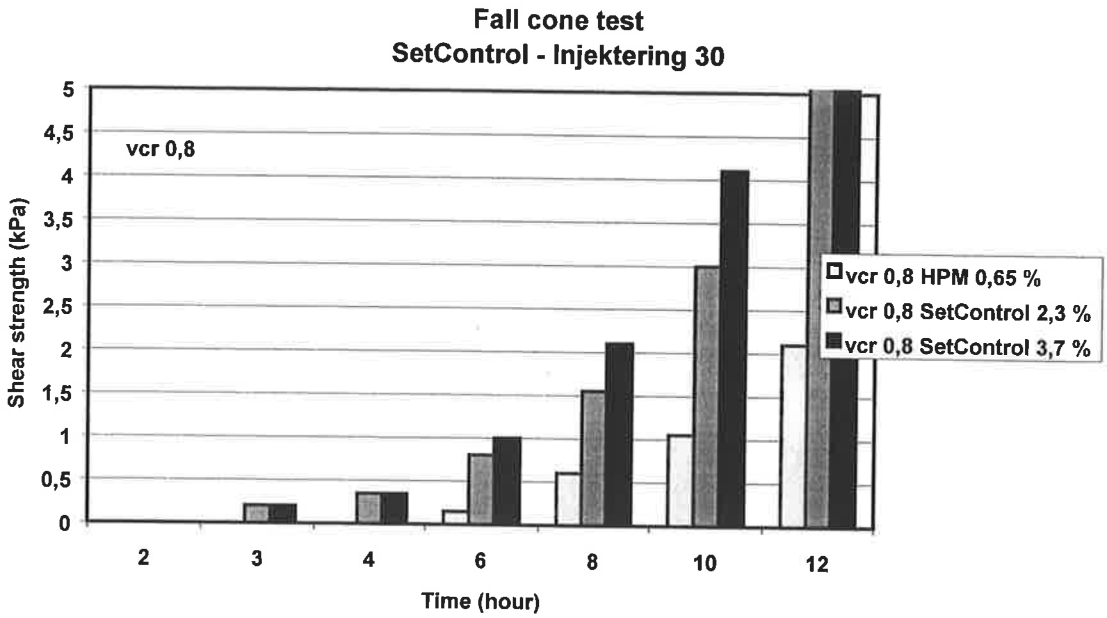

Fig 6a. The shear-strength test indicates that the SetControl reduces the setting time. In this case with 3 hour compared with the standard mix.

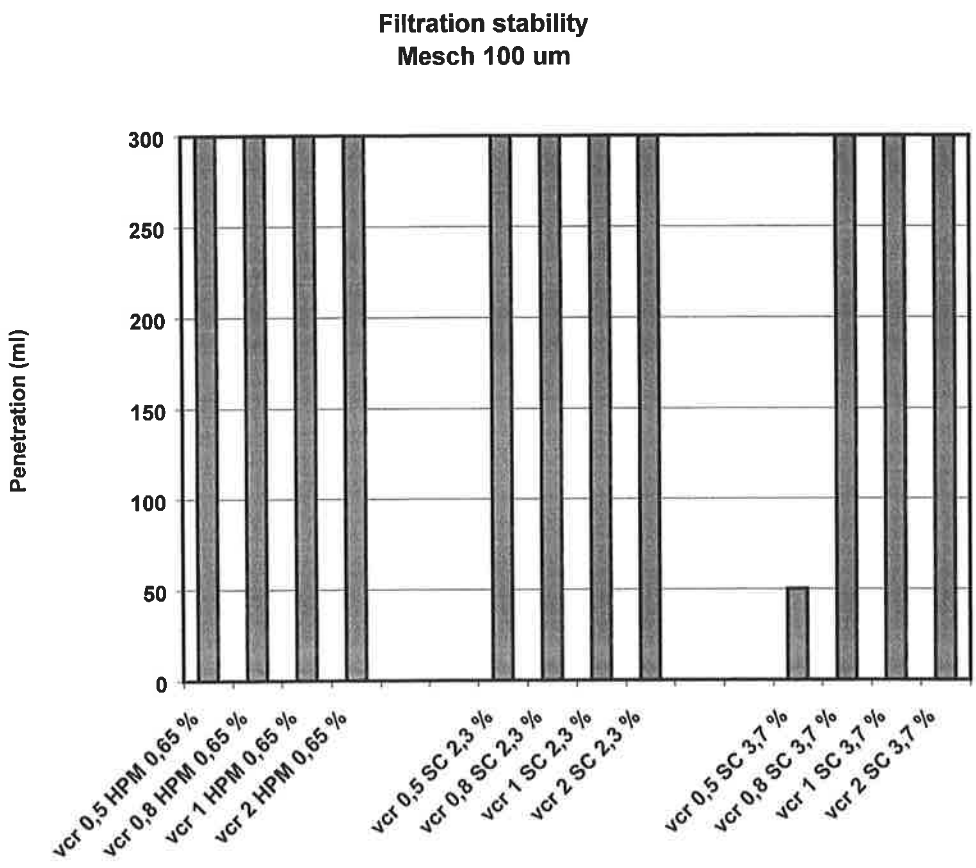

Fig 6b. SetControl (SC in diagram) in combination with Injektering 30 gives equal filtration stability compared with the standard mix. The standard mix consist of the microcement Injektering 30 and the super plasticizer Cementa HPM. The filtration stability test is a standard test (VU-SC:27) in Sweden. The grout should pass a woven metal wire cloth of 125,75 or 45 um.

ENVIRONMENTAL EVALUATION OF NON-CEMENT BASED GROUT

Miljövärdering av icke-cementbaserade injekteringsbruk

Professor Jan Hartlén, PhD, Lund University, Dept. of Geotechnology

Abstract

The use of chemical grout has been looked at as an option in many cases where cement-based grout has not given the required tightness. In Sweden the use of Rhoca Gil at Hallandsåsen and its negative impact on groundwater and surface water, started a large debate if chemical grout should be allowed. Since then, procedures have been developed to give a basis for evaluating any possible negative impact by such use. This paper presents in summary procedures for risk assessment worked out for projects as Hallandsås and Citytunneln in Malmö. The exact procedure for each of the projects is thus not presented here.

Sammanfattning

Användningen av kemiska injekteringsmedel har funnit en marknad, där cementbruk visat sig vara mindre effektiva. Användningen av Rhoca Gil i Hallandsås-projektet, med dess stora negativa effekt på yt- och grundvatten, har dock på ett dramatiskt sätt ändrat förutsättningarna att använda kemiska injekteringsmedel. Som en följd härav har metodik utvecklats för hur man skall utvärdera om det finns risk för negativ miljöpåverkan. Artikeln presenterar i sammandrag de procedurer som utvecklats för Hallandsås och Citytunneln i Malmö. Exakta krav för respektive projekt anges således inte.

Background

The choice of a grouting method to prevent groundwater lowering in the neighbourhood is always considered during the design phase of a project. It should in this context though be mentioned, that grouting is not always the most effective solution. Other measures may instead be more relevant, as re-infiltration of groundwater, lining of the tunnel, etc. Such alternatives shall thus be considered, especially if there is a risk of contaminating the groundwater by chemicals in the grout.

Strategy

The Swedish National Road Authority (Vägverket) has presented a report in which recommendations are given how to evaluate conditions, possibilities and strategies regarding planning and execution of grouting (Vägverket, 2000). These procedures are in overall followed by Citytunneln.

Cement-based grout shall be used as much as possible. The additives then used shall however also be evaluated following the rules of the "Environmental Law" (Miljöbalken) and the Chemical Inspectorate. This means that the best available technique shall be used as well as the principle of precaution followed. It is thus important to notice that all additives also in cement grout shall be evaluated. Only at specific conditions chemical grout shall be considered.

Principles

The steps to be followed, when evaluating the impact of chemical grout, are summarised as follows:

- Relevant chemical grout is defined based on the actual case. Geological and geohydrological conditions shall be well defined.
- A detailed site specific risk assessment is performed. (A more detailed presentation is given below about the facts to be considered.)
- The most proper product is chosen considering the "precaution principle" (försiktighetsprincipen).
- The local or regional authority shall be informed and if they ask for it, be presented a risk evaluation to approve the use of the product.
- Working procedures and control measures shall be defined by the contractor.
- If needed, cleaning processes of the waste water shall be evaluated and chosen.
- A control programme shall be defined and normally be approved by the local authority.

Risk assessment

The risk assessment shall be based upon

- the environmental profile of the product, as ecotoxicological and biodegradable properties as well as occupational health and safety aspects;
- volumes to be used;
- risk that substances are solved in water;
- risk of dispersion and transport in groundwater;

- risk of contaminating valuable ground and surface water bodies;
- risk of getting harmful contaminants in the waste water;
- special caution is required if the product contains substances included in the "OBS-" or "Begränsningslistan".

Concluding remarks

Chemical grout can in many cases be used. It is however very important that a proper and detailed environmental risk assessment is done. By now, it is however often very difficult to get the proper information for such an analysis. It is then of the greatest importance that the client is persistent. Else it may result in a long-lasting "experience" before an approval is achieved if any.

It is thus important that the type of here presented risk analysis always is performed as this will promote development of environmentally acceptable products.

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GOVERNING FACTORS FOR THE GROUTING RESULT AND SUGGESTIONS FOR CHOICE OF STRATEGIES

Styrande faktorer för injekteringsresultatet och förslag på val av strategi

Magnus Eriksson, div. of Soil and Rock Mechanics, KTH.

Abstract

The spreading of grout in rock depends on several issues related to the rock fractures, the grout properties and the technique employed. Some of these issues are studied using numerical analysis in respect to their influence on the grouting result. Through the numerical analysis it was found that a large span in the result can be obtained for fractures with a certain geometrical description. This span in result coincides with practical experiences where a large variation in obtained grouting result have been observed under apparently similar geohydrological circumstances. It is therefor found possible that the field observations that the grouting result is stochastic in its nature origins from variations in fracture geometry. It is as well illustrated that technical issues, such as the minimum flow criteria, can have a significant influence on the result.

It is found that different factors are important in different ranges of fracture apertures. A matrix is given where the importance of different issues is weighted for three intervals of fracture aperture. Based on the matrix, variations in technique are discussed to improve the grout spreading.

Sammanfattning

Spridningen av bruk i berg beror av flera faktorer hos bergsprickorna, bruksegenskaperna och i tekniken. Baserat på beräkningar utvärderas några faktorer med avseende på deras inverkan på bruksspridning. Utgående från den numeriska analysen visar resultaten att en relativt stor spridning i resultat kan erhållas för sprickor med en viss geometrisk beskrivning. Detta överensstämmer med fältobservationer där det visat sig att stor variation i resultat kan erhållas i liknande geohydrologiska situationer. Det kan därför vara möjligt att variationer i fältobservationer kan ha sin naturliga förklaring i sprickors varierande egenskaper. Det visas också hur teknikval avseende ett minsta flödeskriteria kan påverka resultatet i en betydande grad.

Slutsatserna är att olika faktorers betydelse varierar för olika sprickvidder. Mot bakgrund av detta sammanställs en matris där faktorernas betydelse värderas i tre intervall på sprickvidd. Baserat på den matrisen diskuteras även variationer i teknikvalet för att möjliggöra ett bättre resultat.

1. Introduction

Grouting in rock is a practical method to use for mainly sealing the rock from flow of water. Common applications are to seal the foundation of a dam or to seal tunnels from inflow of water. It is found in several situations though, that the result of the grouting is difficult to predict. In tunnels where a stipulated maximum allowed inflow of water must be reached, these difficulties in predicting the result is often followed by a pronounced cost increase. Another situation where a reliable prediction of the grouting result is favourable is for the construction of storage facilities for nuclear waste material. Most programs for such storage around the world have a pronounced interest in usable predictions to avoid unforeseen events and due to publics concern.

To facilitate the design of grouting works, fundamental understanding of the governing mechanisms is essential. To increase the understanding both research and experience are important. Grouting research has the objective to improve both the understanding of the rock and the grouting material, as well as the flow and spreading mechanisms. The work presented in this paper concerns the spreading of a cement grout in rock fractures and how the result can be predicted. The fundamental relation for describing grout spreading is found in Equation 1 (Gustafson & Stille, 1996) to be

$$I = \frac{\Delta P \cdot b}{2 \cdot \tau_0}$$

where I denotes the maximum penetration length of the grout, ΔP the excess pressure, b the aperture of the fracture and τ_0 the yield value of the grouting material. From this we find that the governing factors concerning the grout spreading is the technique (represented by ΔP), the rock conductivity or transmissivity (represented by b) and the fluid properties (represented by τ_0). However, it is also understood that the reality is much more complicated than the simple expression suggests. For instance, the flow properties of a grout depends not only on the yield value (τ_0) but as well on the viscosity and is time dependent.

This paper expands Equation 1 and studies variations in grouting technique and in rock and material properties. Some general conclusions are made concerning the factors in these three issues that appears to mostly affect the grouting result. Also, some general comments and suggestions of design values are presented based upon the findings. This paper is an extract from a Ph.D. thesis under completion and does not give the full background to the calculations. Background for the calculations are presented in previous papers, see Eriksson et al (2000) and Eriksson (2001).

For clarification, in this paper grouting result refers to several different aspects. One is the expected inflow, for instance to a tunnel after a certain grouting operation has been performed. Other issues are the amount of grout and time used for the works.

2. Short presentation of calculation approach and measurement of grout properties

To study governing factors for the grouting result calculations of grout spread and sealing effect is made. The calculations are made in a finite difference model where the flow geometry is modelled with a orthogonal net of conductive elements. The flow model for the cement based grout is the Bingham model. The calculation tool have as well been complemented to include the limited penetrability of cement based grouts. The flow equations used are given in Hässler (1991) and in Eriksson et al (2000). The model for limited penetration ability is presented in Eriksson et al (2000) and in Eriksson (2001).

The model for calculation have two distinctive parts. One is calculation of the water flow and grout propagation in the network which can be seen as a deterministic description of one possible result. The other part is a Monte-Carlo simulations to obtain a distribution of possible results based on the variations of input values.

The calculations of water and grout propagation is performed in a network of conductive elements, illustrated in Fig. 1. The net is described in geometrical parameters, mean aperture and standard deviation of the aperture, and with location is space, dip, strike and depth. It is also possible to input a certain amount of contact area within the net.

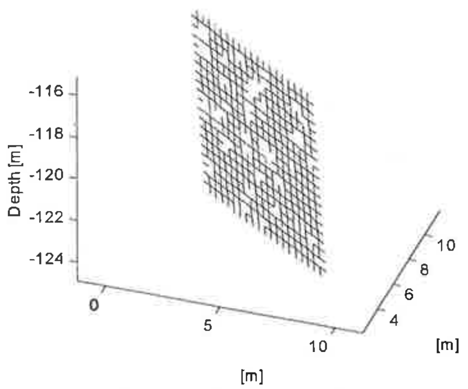


Figure 1. A visualisation of a network in 3D space representing a fracture.

The distribution of aperture field is obtained by randomly distributing widths to the nodes in the generated geometry. The distribution of apertures is made according to a log-normal distribution. The aperture of a channel is given the mean value between the two nodes it is connecting. If a node has zero aperture the channel is given zero aperture as well. This procedure results in different aperture distributions between the nodes and the channels, see Fig. 2. A limited spatial correlation is obtained since the aperture of the node is reflected in all four directions surrounding it.

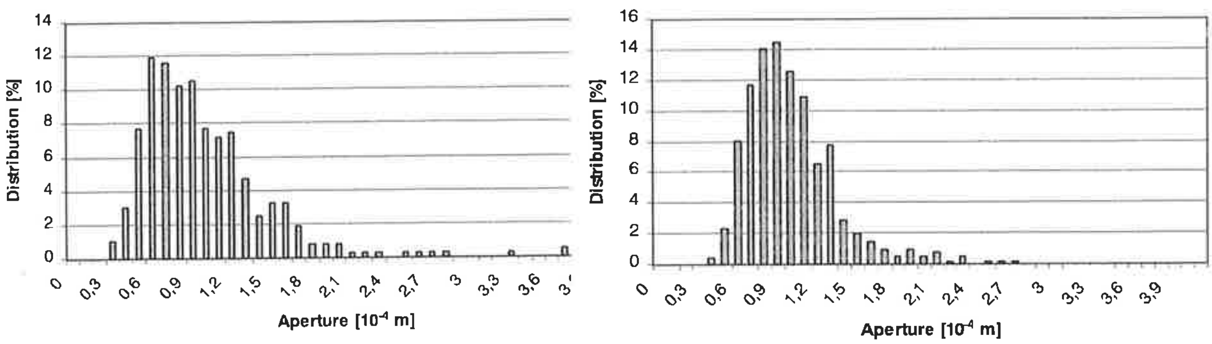


Fig. 2. Example on realised fracture aperture field. To the left is the aperture distribution among the nodes and to the right among the channels. Input values are a mean aperture of 0.1 mm with a standard deviation of 0.05mm and an amount of contact of 0%.

The grout properties are characterised in respect of rheology, penetrability and bleed. The rheology, described with the Bingham model, is measured with a rotational viscometer which evaluates the yield value and viscosity against time. The penetrability of the grout is measured with a specially developed device called penetrability meter which characterise the grout in two parameters, a minimum (b_{min}) and a critical aperture ($b_{critical}$). In the calculations, if the aperture in a channel is less then the minimum aperture no grout enters the channel. If the aperture is larger than the critical the grout can pass unaffected. If the aperture of the channel is larger than the minimum but less than the critical the grout will be filtered and only a limited amount of grout can enter. The limited penetration ability a grout possesses changes with time. Immediately after mixture the grout has a certain penetrability but this decreases rapidly with time.

The bleed is expressed in percentage and refers to the separation of solid particles and the water in the suspension. One way of measuring the bleed is with the standardised test in a cylinder of 30 cm height. This method of measuring is developed for concrete works. In Eriksson et al (1999) it was found that the size effect of a large cylinder is likely not representative for the bleed that occurs in a fracture of small aperture why it was recommended to use a small (~1 cm in height) cylinder when measuring the bleed. This method of evaluation excludes some of the size effect such as consolidation.

3. Governing factors concerning grout spread

This chapter has the objective of highlighting properties and factors concerning the features of fractures, grout and technique that influences the grouting result. The method to do this will be by calculating the grout take for some different cases and compare the results.

The calculations are performed in a geometry as shown in Fig. 1. During the grouting three boundaries are given no flow conditions and one boundary a prescribed pressure, this to simulate grouting in a single fracture which is connected to other fractures in the boundary with a prescribed pressure.

In order to judge the importance and effect of different governing factors the calculations needs to be compared to each other. A set of calculations will be used as reference objects to which other calculated results can be compared and these are called basic cases. The basic cases are calculated based on simplified conditions where a time constant grout with a yield value equal to 1 Pa and a viscosity equal to 0.005 Pas and a total penetrability is used. The calculations are as well performed with a reduced pressure compared to a real situation. In the calculations of the basic cases the grouting is allowed to continue until the flow is below $1 \cdot 10^{-8}$ m^3/s and then stopped. In order to make the calculations fast a large hole radius, of 0.4 m, is used. In Table 1 the geometrical parameters used in the basic cases are presented. Each case is calculated based on 50 realisations to obtain a distribution of possible results.

Table 1. Presentation of the basic cases. The parameter that is distinctive for that case is marked with grey. b is the mean aperture, σ the standard deviation of the aperture and % the amount of contact.

Case no.	Rock related parameters			Case no.	Rock related parameters		
	b	σ	%		b	σ	%
1	0.10	0	0	6	0.15	0.075	0
2	0.10	0.05	0	7	0.15	0.10	0
3	0.15	0.075	0	8	0.10	0.05	30
4	0.20	0.10	0	9	0.10	0.05	50
5	0.15	0.05	0	10	0.10	0.05	70

3.1 Basic cases

The calculated grout take for the basic cases 2-4 and 5-7 are shown in Fig. 3. In the left figure a considerable difference is noticed due to the difference in mean value. The medium grout take in the three different cases are 2.3, 8.5 and 14.5 litres. The individual relation of the grout take values is supporting the dependency of the aperture in cubic, i.e 8.5/2.3=3.7 is close to $(0.15/0.10)^3=3.4$. The difference in grout take due to the variation in standard deviation in the aperture are in these simplified cases not well pronounced as seen in the right figure even though some difference is noticed.

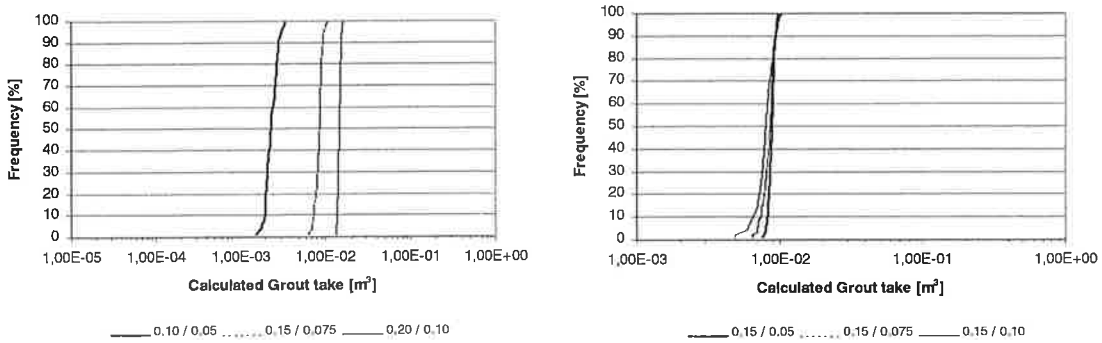


Figure 3. Grout take in cases 2-4 and 5-7.

In Fig. 4 the calculation of the grout take in the fracture with 0.10 mm mean aperture and a standard deviation of 0.05 is compared to the value calculated for a fracture with no variability in aperture, i.e. a plane-parallel fracture (case 1).

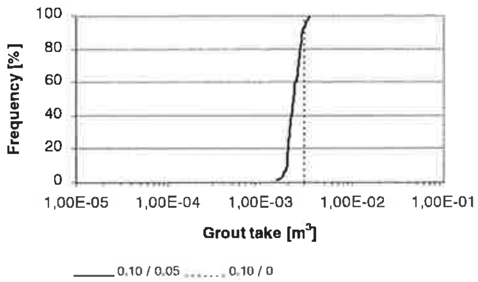
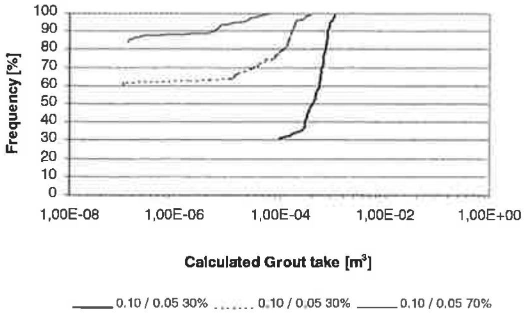


Figure 4. Calculated grout take calculated in a plan-parallel fracture with an aperture of 0.10 mm (Case 1) and in a fracture of mean aperture 0.10 mm with a standard deviation of 0.05 mm (Case 2).

It is seen that the calculated grout take in the case with a variability varies between 1.5 and 4 litres. This means that the grout take and grout spread to large extent depends on where in the fracture the bore hole is placed, independently of used technique and grout. It is seen that the median grout take compared to the plan-parallel case is somewhat smaller, 2.3 litres compared to 3.0 litres.

The contact area that can be present in a fracture causes the grout to travel in a tortuosic path which reduces the effective grout spread length. Fig. 5 shows the calculated grout take in the three cases of different amount of contact (cases 8-10).



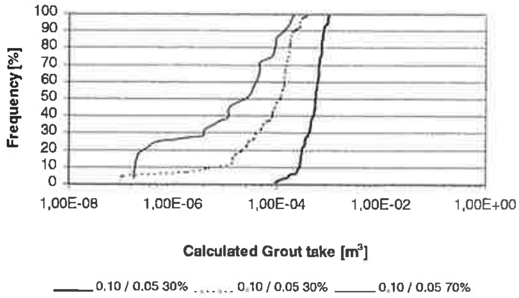


Figure 5. Calculated grout take for three different cases of contact area. To the left all values are presented and to right have all zero values been removed.

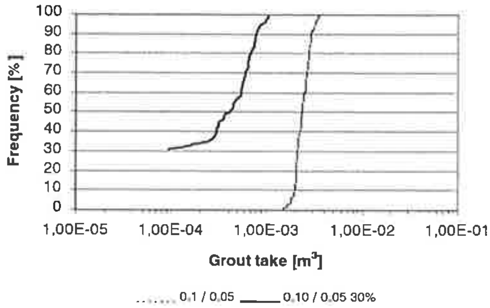


Figure 6. Calculated grout take calculated in a fracture of mean aperture 0.10 mm, a standard deviation of 0.05 mm and with presence of 30% of contact.

The presence of contact area in the fracture is found to have two major influences on grout take. One major influence is that some of the potential grouting holes strikes the fracture in a contact area and hence not contribute to the sealing of that fracture. The effect of this is highly bounded to how the amount of contact is distributed. If the amount of contact is positioned in areas larger that the bore hole diameter some grouting holes will strike areas of contact fully and then not be conductive. However, if there is a reasonably low amount of contact divided on several places within the fracture a grouting hole would partly be in a contact area and partly in a conductive area of the fracture. In such a case the influence of the contact is limited even though the effective bore hole radii decreases. In the network used of the calculations the a point of contact covers one node, i.e. if the case of 30% of contact 30 % of the nodes are non-conductive.

The other effect is that even if a conductive part of the fracture is hit with the bore hole the grout take reduces as the amount of contact increases. The result can be compared to the same situation but with flow of water. Chen et al (1989) shows for instance a calculated reduction in normalised permeability of around 50% due to an amount of contact of 30%.

If the calculated grout take in the cases with presence of contact areas are compared to the cases without contact present it is seen how the grout take decreases. The median grout take without presence of contact was 2.3 litres and with presence of 30% of contact 0.4 litres, see Fig. 6.

3.2 Aspects related to the material

The properties of the grout are of significance to the grouting result. Concerning rheology this has for instance been shown by Hässler (1991). The aspects concerning the grouting material are for spreading of grout rheology and penetration ability and for sealing effect bleed.

As with the rheology the penetration ability is time dependent, see Eriksson et al (1999). The strongest time dependency is seen early after mixture to reach a sill after approximately one hour in the examined grout used here. Fig. 7 shows a evaluated measurement on grouting cement (d_{95}=30μm) with a w/c ratio of 0.7 and 0.2 % of superplasticizer.

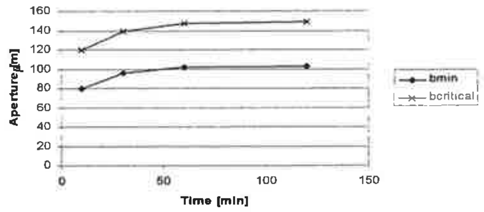


Figure 7. A measurement on the two parameter b_{min} and $b_{critical}$ versus time. The grout is based on grouting cement (d_{95}=30μm) with a w/c ratio of 0.7 and 0.2 % of superplasticizer.

In Fig. 8 are shown the grout take for cases with different mean value and different standard deviations and with the time dependent penetrability acknowledged. It is seen how both the mean value and the degree of variability have a strong effect on the grout take. The medium value on grout take for cases 2-4 is 0.03, 2.2 and 11.1 litres compared to the previously calculated values of 2.3, 8.5 and 14.5 litres. For cases 5-7 the medium grout take is 1.7, 2.2 and 2.9 litres to be compared to 8.0, 8.5 and 8.8 litres when calculated without considering the limited penetrability. This reduction in grout take exemplifies the importance of incorporating the limited penetrability.

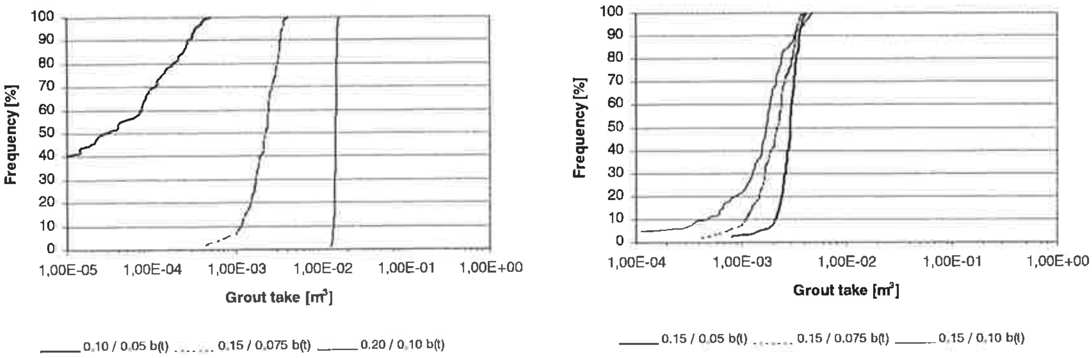


Figure 8. Calculated result of grout take as the limited penetration ability is incorporated.

3.3 Aspects related to the technique

The technique by which the grouting is performed is often less discussed than for instance the choice of grout mix. Technique is here considered to cover the hole density, the grouting pressure and special refusal criterion, flow and volume criterion.

The grouting pressure refers to the maximum pressure to be used and is often set to 2-4 MPa over the existing ground water pressure. During the grouting it is more often the case that the pump is restricted in flow so that the grouting is performed during the main part of the time at a lower pressure than the maximum. In the later part of the grouting when the flow decreases the pump can use a higher pressure. When the pressure reaches the maximum and the flow is less than the minimum flow criteria the grouting is stopped. Sometimes this does not occur and the grouting seems to could be continued a very long time. In such situations it can occur that a volume criteria is reached, stating that when a certain grouted volume is reached the grouting is to be stopped. It is evident that these kind of alterations changes the grouting result compared to the theoretical and therefore needs to be evaluated. In the following an illustration of the use of a minimum flow criteria is made.

The time to reach full (theoretical) penetration is in the normal case very long. This is the reason why in practical grouting a limit on the flow is set under which the grouting is stopped. This flow criterion is often set in the range of 0.5 litre / min.

Fig. 9 shows a calculation of flow and penetration against time in a set of plan-parallel fractures with the aperture of 0.2 mm. Shown in the figure is the accumulated flow for 1, 2 and 4 fractures.

Fig. 9 shows that the level of the minimum flow criteria can be of different significance depending on whether one or several fractures are met. In this particular case where 0.5 litre/min was set it is seen that if only one fracture is met (with the aperture of 0.2 mm) the grouting is immediately stopped. If instead 2 or more fractures are met a certain penetration is reached before the grouting is stopped. In the case of 2 fractures approximately 1.5 meter penetration is reached and in the case of 4 fracture approximately 2.5 meter are reached.

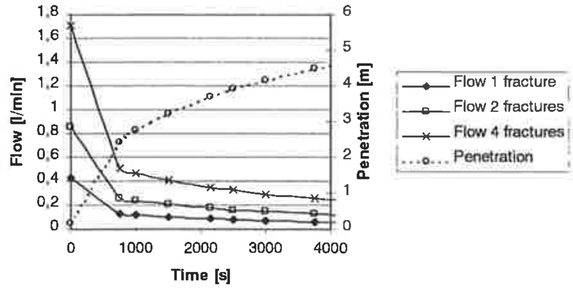


Figure 9. The total flow in bore hole for 1, 2 and 4 fractures, each 0.2 mm in aperture and plane-parallel. The penetration in each fracture is also given.

An example of obtained sealing effect, evaluated as the reduction in cross fracture flow, as function of different refusal criterion concerning the pump flow is shown in Fig. 10. The values are for a single fracture.

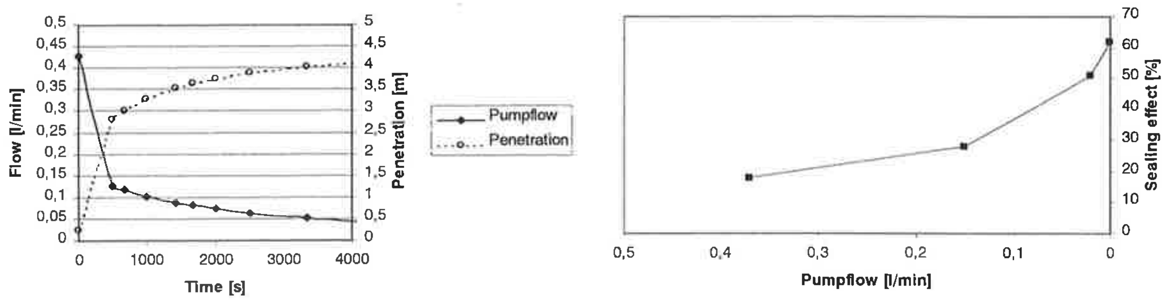


Figure 10. Pumpflow during grouting of a single fracture. To the left is shown the pumpflow and penetration length in the fracture against time. To the right is shown the obtained sealing effect at different levels of pumpflow.

4 Conclusions

In this paper some governing factors for the spreading of grout in fractures have been studied. Through the numerical analysis it was found that a large span in the result can be obtained for fractures with a certain geometrical description. This span in result coincides with practical experiences where a large variation in obtained grouting result have been observed under apparently similar geohydrological circumstances. It is therefor found possible that the field observations that the grouting result is stochastic in its nature origins from variations in fracture geometry.

For the spreading of grout, it is found that the mean value, the standard deviation in aperture and the amount of contact are important properties.

The dependency on the mean value is well known as governing the flow of fluids, but less discussed is the dependency on the variability in aperture. It is found that the variability increases in importance as the mean value decreases and when the limited penetration ability in the grout is considered.

The outcome of possible grouting results was severely affected by the introduction of a certain amount of contact within the fracture surface and therefor this is important parameter in the fracture description. However, in this work the significance of the size and distribution have not been evaluated and the result must be viewed in respect to this.

A separate analyse of the flow against time revealed that technique issues, such as a minimum flow criteria, can have significant impact on the grouting result. In situations where a low inflow of water is requested and a sparsely fractured rock mass is encountered great improvements in sealing effect can be obtained by lowering the minimum flow criteria.

5 Discussion and suggestions of grouting design value

Table 2 is presented as a summary of the results concerning the importance for the sealing effect of different geometrical factors and factors connected to the grout properties. In this table the importance of each factor has been judged based upon the presented calculations. Each factor is judged for fractures of an aperture less the 0.1 mm, between 0.1 and 0.2 mm and of an aperture larger than 0.2 mm. All calculations for this analyse are not presented in this paper due to page limitations and are therefor referred to the forthcoming Ph.D. thesis of the same author.

Table 2. The weighed importance for the sealing effect of the factors concerning geometrical features of the fracture and the functionality of the grout. ++ represents high importance, + important, - not important

	Factor	← 0.1 mm	0.1 mm – 0.2 mm	0.2 mm→
Geometrical factors	Standard deviation of aperture	++	+	-
	Amount of contact	++	+	-
Functionality of grout	Low Viscosity	++	++	+
	High Penetrability	++	+	-
	Low Bleed	-	+	++
	High Yield value	-	-	+

The different features of the fracture geometry are then concluded to have an impact on the grout spreading and the obtained sealing effect. Since the geometrical features of the fracture generally cannot be changed (neglecting techniques proposing hydraulic jacking and likewise), this needs to be compensated in the choice of technique. Table 3 weights various issues in selecting technique for various aperture sizes.

Table 3. Proposed attentions when choosing technique for fractures of different apertures. ++ represents high importance, + important, - not important

	Factor	← 0.1 mm	0.1 mm – 0.2 mm	0.2 mm→
Technical issues	High pressure	++	+	-
	Low minimum flow	++	+	-
	High max volume	-	+	++
	Small distance between grouting holes	++	+	-

Of course, all this four issues concerning the choice of technique are favourable for a good grouting result. However, all four issues as well increase the necessary amount of grouting works so that the cost increases. The objective is however to pin point the minimum amount of works to achieve an acceptable level on inflow and Table 3 is given in respect to this.

Acknowledgement

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GROUTING FIELD INVESTIGATIONS AT SOUTH LINK

Injekteringsförsök vid Södra länken

T. Dalmalm
Royal Institute of Technology/NCC AB
T. Janson
Golder Associates AB
B. Karlsson
National Road Administration

ABSTRACT
During the construction of the rock tunnels at the South Link, Stockholm, grout investigations were carried out. The aim was to develop alternative methods/concepts for sealing of tunnels compared to the
methods/concepts described in the contract. The investigation started with a laboratory study, which outlined the most appropriate grouts for field trials. The field trials consisted of seven grout fans. The front of each fan was divided for equal pumping test values, and grouted with different concepts. The sealing result and the spreading of the grout were studied in 10 holes in the tunnel front and in 10 holes perpendicular to the tunnel. The holes were test pumped and studied with a borehole camera. From the investigation it was concluded that regardless the maximum grain size of cement (9.5, 16 or 30 mm) fine fractures (< 100 mm) were still unsealed after grouting. In open fractures or rock masses (>1 Lugeon) the micro cement was more effective than the more coarse cement type. Higher stop pressure did not during these limited trials give a better sealing result and some parameters achieved at laboratory especially for micro cements were not possible to reproduce with ordinary field mixers.

SAMMANFATTNING
Under byggandet av bergtunnlarna för Södra Länken utfördes ett antal injekteringsförsök. Syftet med försöken var att utveckla alternativa metoder och/eller koncept som kunde ge bättre tätning och på så sätt minska användandet av eventuell efterinjektering med kemiska injekteringsmedel. Försöken inleddes i laboratoriet där lämpliga injekteringskonsept för de följande 7 fältförsöksskärmarna togs fram. Varje fältinjekteringsstuff delades i två halvor med lika vattenförlust. Skärmhalvorna injekterades därefter med olika injekteringskon-cept. Täthetsresultatet och spridningen av bruket studerades dels genom 10 observationshål i stuffen och dels genom 10 hål vinkelrätt tunneln. Alla hål studerades med videokamera och vattenförlustmättes. Från försöken noteras att, oavsett maximal kornstorlek hos cementen (9.5, 16 or 30 mm) förblev fina sprickstrukturer (< 100 mm) otätade även efter injektering. I mer uppspruckna bergmassor eller sprickor med Lugeon >1, var mikrocementen mer effektiv än de grövre graderade cementen. Ett högre injekteringstryck gav under dessa begränsade försök inte något förbättrat täthetsresultat. Bruksparametrar erhållna från laboratoriet var svåra och i vissa fall omöjliga att reproducera i fält med fältutrustning och då särskilt för mikrocement.

1 Introduction

The project was a co-operation between Swedish National Road Administration and Royal Institute of Technology and NCC AB. The project was carried out during the construction of rock tunnels for the south part of the Stockholm Circle. The investigation was carried out between July –99 and February –00. A working team from The Royal Institute has planned, performed and analyzed the trials, (Dalmalm et al. 2000). A reference group of representatives from the co-operation companies have, during the project, participated in the work.

The tunnel excavation at the South Link was, due to the location in the Central Stockholm, connected to high demands regarding sealing and performance. The Swedish National Road Administration wishes to develop methods for sealing of tunnels, based on latest experiences and science results, to reach a high sealing efficiency and a low impact on the environment.

The aim was to develop alternative methods/concepts for tunnel sealing, in relation to the methods described in the contracts, so that the need for post-grouting and chemical grouting could be reduced. The alternative methods/concepts should be cement based and production efficient. The alternative methods/concepts should be compared to the methods/concepts described in the contract.

In order to fulfill the sealing demands, a sealing efficiency between 90 to 95 % is needed. The grout then needs to both penetrate and fill the fractures, and in addition, the grout needs to be long term resistant. To reach the sealing demands it was necessary to obtain a hydraulic conductivity of the grouted zone of 0.5-0.4 $*10^{-8}$ m/s, equal to seal fractures down to an aperture of 0.1 mm partially. It was predicted that it would be difficult to reach a hydraulic conductivity below $0.5*10^{-7}$. When sealing apertures less than 0.2-0.3 mm filtration of the grout will affect the result. The grout trials confirmed the predictions. At this point, a priority order between the different methods/concepts to seal the fracture system was done. The trials started with finding a proper grout mix, to be followed up by trials with high grout pressures. A number of demands on grout properties were set up, related to environment, function, and production demands. At the area of the trials, the sealing demand was expressed in such way that the inflow to the tunnel should be less than 1-3 litres per meter of tunnel, depending of location. The specification of the grout was as follows: Shear strength after 2 hours has been specified for production purpose. The measurement should be done with a fall cone and the strength should be 3 kPa after 2 hours. The grout should be separation stable, specified in such way that separation and shrinkage should be less than 2 % after 2 hours. The filtration stability should be measured with a filter pump. One hour after mixing, at least 300 ml should be able to pass through a 125 μm filter.

2. Grout trials

2.1 Investigation of grouts

The trials started with a request to nine different cement producers. The geological situation was described and they were asked to recommend a cement based concept, based on the information given. After our laboratory investigation, three concepts were chosen for an initial field investigation. The concepts were modified for the additional information on the geology of the trial area. The initial laboratory study of grout included the cement: Rheocem 800, Injektering 30, RockU, P-U, P-X, Thermax (not cement). The different cements were tested for different W/C-ratios and different additives, to achieve optimal results. In the laboratory, properties such as rheology, stability, filtration and shear strength development were studied. The different properties were then evaluated by means of a Marsch cone, viscometer, mud balance scales, filter pump, sand column, graduated glass and fall cone equipment. During the field trials, exactly the same set up of grout examinations was performed, except for the sand column, in order to make them comparable. The strategy, during the trials was, firstly to mix a recipe in the lab in order to obtain reliable grout parameters, secondly to mix at the grout rig and use the same equipment for grout property measurement as in the lab. During the trials, some recipes showed a large divergence from the laboratory study. An extra control of the dispersion ability of the field grout mixing equipment was, therefore, performed.

2.2 Geology

The geology varies strongly within the area of the grout trials. A General judgment, prior to the trials, was that the rock mass from a grouting point of view could be described as:

- A low hydraulic conductivity, approx. $1*10^{-7}$ (ca.1 Lugeon) or lower.
- Single fractures, between 0.1-0.5 fractures per meter, with varying conductivity.
- Narrow apertures, < 0.5 mm, with varying fracture filling.

The prior judgment was in accordance with the rock mass present at the site of the trials. The measured average Lugeon values were very low, equivalent to a hydraulic conductivity between approx. $3*10^{-7}$, and $2*10^{-8}$ m/s.

2.3 Field trials

The grout trials consisted of seven fans, each 20 meters of length. They were performed in three stages, 2+ 2 + (2 +1). The fan was drilled in agreement with the ordinary drill plan for the area, which made the trials comparable to surrounding ordinary grouting. All holes were test pumped, and then the fan was divided into two halves with equal Lugeon sums, see Figure 1.

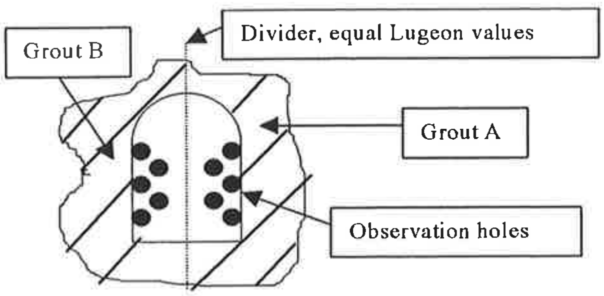

Figure 1 Principal layout. Divided fan and corresponding observation holes.

In the fan, 10 observation holes were drilled forwards, five in each half (Figure 2). The holes were test pumped and studied with a digital camera. The holes were sealed with a sleeve and water-filled.

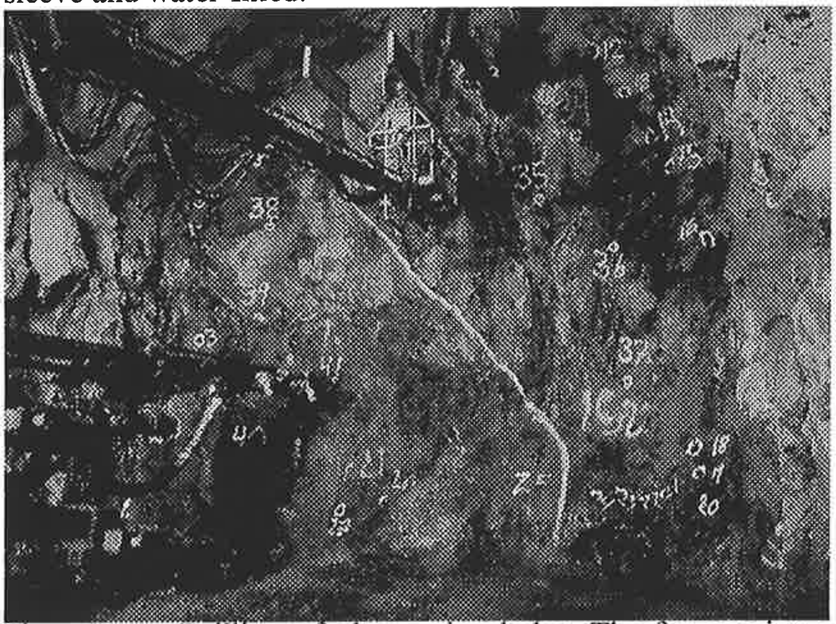

Figure 2 Drilling of observation holes. The fan consists of 32 grout holes in the periphery and 10 observation holes in the middle. The separator between equal Lugeon sums is shown.

The fan was grouted hole by hole. Connections with other holes were noticed. After grouting and hardening, the observation holes were again test pumped. After blasting and 10 metres ahead of the grouting point, 5 spreading holes were drilled in each half, perpendicular to the tunnel, in order to study the perpendicular grout distribution (Figure 3). The spreading holes were also test pumped and studied with digital camera. A number of times, during the tests, the rock mass was so tight, that no grouting were necessary, or that the pumping test values before grouting were so low that a significant difference after grouting would not be possible to detect. Therefore, the tests were interrupted at an early stage. Survey of the trial areas was carried out for the tunnel, both as ordinary survey and by means of digital photography.

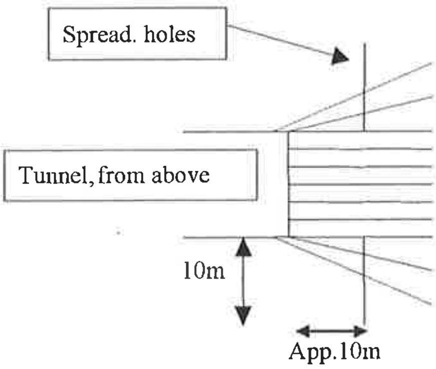

Figure 3 Principal layout of spreading holes in a fan.

Table 1 Performed concept for the different trial fans.

Fan	Concept A	Concept B
1*	RockU wc 1.0	Rheocem 800 wc 1.2
2*	Rheocem 800, wc 1.2	RockU wc 1.0
3	RockU wc 1.0	Inj 30 wc 1.0
4	Inj 30 wc 1.0	RockU wc 1.0
5A	Inj 30 wc 0.6, high pressure	Inj 30 wc 1.0 – 0.5
5B	Inj 30 wc 0.6, high pressure	Inj 30 wc 1.0 – 0.5
6	Inj 30 wc 1.0 – 0.5	Inj 30 wc 0.6

* In the two first fans, colored grouts were used. The measurements with the filter pump showed decreased penetration for the colored grouts. Therefore, it was decided not to use colored grouts in subsequent fans.

3 Results

3.1 Laboratory testing

The grout, which best fulfilled the demands were Cement Rock U, closely followed by Cement Rheocem 800 and Cement Injektering 30. From laboratory three mixes were then presented as ready for field investigation, see table 2.

Table 2. The three grout mixes which where selected for field investigation.

Type	Rheocem 800	RockU	Cementa IC 30
VC ratio	1.2	1.0	0.6
Additive	Rheobuild 1.4 %	Compound B 4 %	HPM 0.7 %
Accelerator	-	-	2 %

It may be noticed that both RockU and Rheocem are micro cements, while Injektering 30 is an ordinary cement grout, with coarser grains. It may also be noticed that RockU is gypsum-free and that the retarding gypsum content is added in the superplasticiser. In that way grout hardening could be directed, as desired. Furthermore, it was noticed that hardening of Rock U were strongly correlated to the amount of superplasticiser and the present temperature, which might be both an advantage and a disadvantage. During grouting grout consumption normally varies strongly. With low consumption, risk for grout hardening in the agitator is high. As a solution, adding of gypsum superplasticiser of Rock U into the agitator was investigated. By doing so, the short hardening time needed in some situations could be combined with cement that does not cure in the agitator. The penetration ability of the different grout mixes was compared by means of three different methods: filter pump, sand column and NES equipment. Generally it was noticed that a finer grain size distribution gives a higher penetration, but a number of exceptions was also noticed. The three different methods also showed some difference in the results. For the filter pump, penetration and grain size correspond well. Even for the sand column, penetration and grain size correspond, but change of filter (sand) is a troublesome operation, making the method a bit slow. A certain threshold value was also observed where some particles passed and others were trapped. For the NES equipment, grain size and penetration correspond initially. Thereafter, a threshold value is noticed. The threshold value is here a smallest column width, which no particles pass, without respect to grain size. This result is interesting in combination with the field trials. According to the field trials, the sealing efficiency was not always improved by using micro cement. For rock mass purpose, a combination of the NES equipment and the filter pump should be appropriate for measuring the penetration ability.

3.2 Field trials

An overview of the results from the field trials is presented in Figure 4.

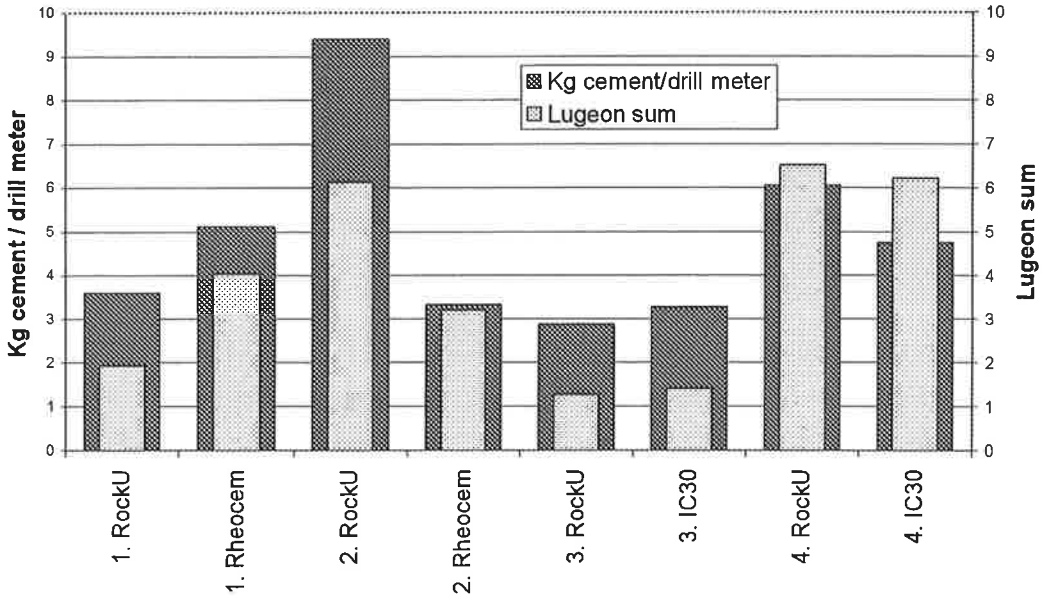

Figure 4 Lugeon sum and grouted cement amount for 8 fan halves.

In general, for all fans, the grout amount in kg is correlated to the Lugeon value, although some exceptions and explanations are noticed. In Fan 4, two of the grout holes had contact with soil, above the rock mass, resulting in a grout stop prior to flow criteria. A tendency during all trials was that the start grout of the two halves had higher grout consumption than the following grout. In fan 5A, there was both face leakage resulting in far to high Lugeon values, and data loss from the grouting. In Fan 5B, high face leakage occurred, resulting in abnormal Lugeon values, not corresponding to grout volume. The Lugeon values from Fan 6 were very low. According to the criteria for trial grouting, they were too low, which means that no significant difference in prior and post grouting would be possible to measure in the observation holes.

3.3 Observation holes

The result from the grouting has been measured in observation holes.

A comparison between observation holes, affected by the different concepts is shown in Table 3. Holes with a value below 0.15 Lugeon are considered water tight, although it is not possible to measure differences of that scale. A hole is interpreted as influenced, if the Lugeon value is lower after grouting. According to table 3, it may be noticed that Cement Injektering 30 have influenced all corresponding holes in all fans. Compared to the result in Table 4, it may be noticed that this influence is equal to a 49 % reduction of the Lugeon value. According to Table 4, Cement Rheocem achieved a 66 % reduction of the Lugeon value, equal to that 67 % of the holes were uninfluenced.

Table 3 Observation holes influenced by grouting.

Fan	Rheocem	Cementa Inj. 30	Cementa Inj. 30 high pressure	Rock U
	Influenced (total)	Influenced (total)	Influenced (total)	Influenced (total)
1	3 (3)			0 (1)
2	1 (3)			0 (0)
3		0 (0)		1 (2)
4		2 (2)		1 (2)
5A		2 (2)	1 (2)	
5B		5 (5)	3 (4)	
6		- (0)	- (1)	
Influenced %	67	100	57	40

The conclusion from above is, therefore, that Cement Injektering 30 more often hits the water bearing fracture, but not seals it so good and Cement Rheocem less often hits the water bearing structure, but when doing so, the sealing effect is very good.

Table 4 Lugeon sum before and after grouting, as measured in the observation holes. T_{eff} = sealing efficiency.

Fan	Rheocem	%	Cementa Inj. 30	%	Cementa Inj. 30 high pressure	%	Rock U	%
	Before (after)	T_{eff}	Before (after)	T_{eff}	Before (after)	T_{eff}	Before (after)	T_{eff}
1	7.35 (0)	100					1,07 (0.59)	45
2	1,46 (1.01)	31					0.5 (0.12)	76
3			0.44 (0.23)	48			2.73 (1.96)	28
4			2.78 (0.23)	92			0.78 (0.7)	10
5A			1.33 (1.16)	13	0.96 (0.73)	24		
5B			10.1 (6.64)	44	2.70 (1.79)	34		
6			0.42 (-)		0.63 (-)			
Mean		66		49		29		40

- = Missing value.

From table 4 it could be understood that both micro cements (Rheocem and RockU) had a very good sealing in fan 1 and 2, located in a fairly fractured rock mass. Cement Injektering 30 had a better sealing effect than Cement Rock U at fan 3 and 4, located in a dense fractured rock mass. At fan 5A and 5B a higher grout pressure was used together with Cement Injektering 30, but the sealing effect was not good in the fairly fractured rock mass.

3.4 Mixing trials with field mixer

During the trials at South Link, two micro cements were used. For both of them, but in particular, for Cement Rock U, the pre-laboratory properties of the grout, were not achieved during the field trials. Therefore, so called "field mixing trials" were performed, without using the grout for grouting. To produce a correct mix takes between 7 and 8 minutes - much longer than is common for ordinary production grout a mix. It is in addition very important that the different components are added in due order time. The purpose of field mixing trials was to examine the difference in properties achieved using a field mixer and a laboratory mixer, respectively, and, in addition, to strictly mix according to the producer's recommendations. One parameter, which clearly did not correspond between laboratory and the field mixing, was the filter pump values. In table 5, a comparison is shown between field and laboratory for the filter pump. Even though the mixing was carried out strictly according to the producer's recommendations, it was not possible to reproduce the parameters from the laboratory. It might, therefore, be suggested that the divergence between field and laboratory results depend on a poorer dispersion, when using the field mixer.

Table 5 Filter pump values for the cement Rock U. Measured at the field trials and refrigerated laboratory (field/lab) [ml].

Time / Filter:	125 μm	75 μm	63 μm	45 μm
10 min	300 / 300	120 / 140	50 / 130	- / 25
30 min	300 / 300	90 / 140	50 / 120	- / 25
60 min	280 / 300	75 / 140	30 / 90	- / 20
120 min	240 / 300	75 / 130	40 / 15	- / 20

3.5 Spreading holes

In order to be able to verify the impenetrability of the grouted sections and to estimate perpendicular grout spreading, holes where drilled in the middle of each trial fan, see Figure 3. In each fan there were 10 spreading holes, which were test pumped and studied with a borehole camera. Results from pumping tests are shown in Table 6. The table has been divided into grout 1 and grout 2, corresponding to the first and second performed grout mix of each fan. In the table, the average value for all spreading holes of each fan is also shown. As shown in the table, the sealing result measured from the spreading holes is very good for all except one of the fans, indicating that a good sealing of the rock mass has been achieved as far out as 10 meters from the tunnel face in those fans. In Fan 5A, high Lugeon values were obtained which can be explained by high face leakage. In Fan 6 the rock mass was very tight even before the grouting, and very small amounts of grout were consumed.

Table 6. Measured Lugeon values in the spreading holes for the different fans.

Fan	Lugeon (mean)		Mean
	Grout 1	**Grout 2**	**1 and 2**
1	0,33 (Rheocem)	0,13 (RockU)	0,23
2	0,07 (Rock U)	0 (Rheocem)	0,04
3	0,04 (Inj 30)	0,04 (RockU)	0,04
4	0,26 (RockU)	0,22 (Inj 30)	0,24
5a	3,24 (Inj30 high)	3,10 (Inj30 contr.)	3,17
5b	0,01 (Inj30 contr.)	0,60 (Inj30 high)	0,31
6	0 (Inj30 contr.)	0,71 (Inj30 high)	0,35

This indicates that the sealing achieved was local in the near field of the grout hole, and that sealing 5 to 10 meters from the drill hole not was achieved in this rock type.

3.6 Surveying of grouted areas

Geologists have surveyed the grouted trial areas using both ordinary ocular inspections and digital photography. Damp, drip and presence of grout at tunnel face have been observed. If the different concepts at the different fans should be compared, all surrounding conditions have to be stable and equal. The digital photography was performed a couple of mounts after the latest grouted fan At that time, all trial fans had achieved stable conditions after grouting and were, therefore, comparable. The ocular inspection of the grouted areas was graded 1 to 5, where 1 means that the area is very dry, and 5 very wet. Fans 1 and 2 were by the ocular inspection graded 5. The grouted areas are here much dryer than the surrounding areas, grouted with the ordinary concept for the tunnel. Fans 1 and 2 were grouted with micro cement (Rock U, Rheocem) and the surrounding areas with ordinary grout cement (Injektering 30). For this rock type, micro cement was consequently more efficient. Fans 3 and 4 were by the ocular inspection graded as 4. The areas are relatively dry and had less wet spots than had the surrounding areas. Fans 3 and 4 are grouted with the grout cement Injektering 30 and the micro cement RockU. Fans number 5A, 5B and 6 are graded as 3. The areas are OK, but have the same intensity of wet spots as have the surrounding areas. The grouts used at Fans 1 and 2 contained red and yellow iron oxide, in order to follow the grout path ahead of the tunnel face. The colored grout was observed in a nearby tunnel, confirming at least 20 meters' spread ahead, from the bottom of the grout hole. In a parallel tunnel, with 5 meters' of rock between the tunnels, the colored grout was observed from the bottom to the top of that tunnel, confirming at least 15 meters' of spread perpendicular to the grout holes.

4 Conclusions

Our conclusions from the laboratory and field investigation are summarized as follows:

- Joints with a hydraulic aperture of $\leq$0.1 mm were not possible to seal, neither with conventional injection cement (30μm) nor with micro cements (9.5μm).

- The trials with a higher stop pressure, 45 bars, did not give a better sealing result than the trials with 25 bars. Rather, an opposite effect, with poorer results from the observation and spreading holes, compared to the other concepts.
- Based on survey of damp and drip on the tunnel face, micro cements (Fans 1 and 2) performed much better than did other concepts.
- Measured Lugeon values between 0 and 0.15 are below the accuracy of the equipment and could therefore not be determined.
- For the micro cement, Rock U, it was not possible to reproduce laboratory grout parameters, not even outside production, when mixing strictly to the recipe.
- Lugeon sums below approx. 0.3 give, in general, only hole filling (the water pressure during pumping tests were 20 bars).
- The available grout mixing equipment restrains the choice of suitable grout.

4.1 Recommendations for further grout works at South Link

- The investigation has shown that, with ordinary grouting equipment, and for tight rock mass situations, here equal to a joint intensity of about 2.3 joints per meter and apertures between 0.05 to 0.15 mm, micro cement do not perform better than conventional injection cement.
- Micro cement may be used for more fractured rock, with wider joint apertures (>0.5 Lugeon), and will then fill the joint better than conventional injection cement, and thereby give a better sealing result. At such rock mass conditions, micro cement could be used, even with traditional mixing equipment, with poor dispersion ability. But naturally, the sealing effect will be much better if the equipment is exchanged to a high dispersion equipment or method.
- Improve hole cleaning before grouting.
- Train the personnel in grout mixing.
- Use more flexible sleeves, which can handle a larger variation in hole diameter.

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TUNNELS FOR THE CITIZEN – A NORWEGIAN R&D PROJECT

Miljø- og samfunnstjenlige tunneler – FoU-prosjekt i Norge

Helen Andersson, Geoteknisk Spiss-Teknikk AS
Kjell Inge Davik, Statens vegvesen Vegdirektoratet
Torkild Åndal, NVK Norsk Vandbygningskontor AS

Abstract

Planning and constructing tunnels in urban areas requires an environmentally friendly approach. This challenge is the focus of a comprehensive research and development programme, inititated in 1998-99 by the Research Council of Norway. The project includes preliminary investigations, environmental concerns, and sealing techniques. In short, the aim is to make transport tunnels more cost efficient and to minimize the environmental consequenses [1]. One activity called "Grouting strategy for pre-grouting of tunnels" has involved the compilation of grouting experiences from a selection of tunnel projects. Six tunnels constructed between 1995-2001 were studied, all but one in Norway. The selected tunnel projects provided a wide spectre of sealing situations in varying conditions. This paper briefly presents data and results for the tunnels, as well as some of the conclusions drawn from the study.

Sammanfattning

Planlegging og driving av tunneler i tettbygde strøk forutsetter en miljøriktig fremgangsmåte. Dette fokuseres i et omfattende forsknings- og utviklingsprosjekt, initiert av Norges Forskningsråd 1998-99, FoU-prosjektet er delt in i følgende delprosjekter; Forundersøkelser, Sampill med omgivelsene og Tetteteknikk. Målet er å gjøre transporttunneler mer kostnadseffektive og å minimere konsekvensene på miljøet. Aktiviteten "Injeksjonsstrategi ved forinjisering av tunneler" omfattet innhenting av injeksjonserfaringer fra et utvalg av tunnelprosjekter. Sex tunneler ble studert, drevet i perioden 1995 till 2001, samtlige i Norge unntatt én. De utvalgte prosjektene gir et bredt utvalg av injeksjonstilfeller under varierte forhold. Artikkelen sammenfatter data og resultater for tunnelene, og også noen av konklusjonene fra undersøkelsen.

1. Main project "Tunnels for the citizen"

The Norwegian transport system includes appr. 700 km of road tunnels, 250 km of railroad tunnels, and 40 km of metro tunnels. The problems during the construction of Romeriksporten raised questions on how to plan and construct tunnels in urban areas in an environmentally friendly manner. In 1998-99, a comprehensive research and development programme was therefore initiated. This effort is financed by the Research Council of Norway, owners, contractors, and consultants. In short, the aim is to develop and improve Norwegian tunnel technology, to make transport tunnels more cost efficient and to minimize the environmental consequences. A preliminary project [2] established following areas to be studied further in the main project:

- Sub project A "Preliminary investigations" – different test methods are tried out for several projects and the optimal extent of preliminary investigations is sought
- Sub project B "Environmental concerns" – the correlations between tunnel leakage / change of pore pressure / damages are studied, and classifications of accepted limits for tunnel leakage and vulnerability of vegetation / water sources are assembled
- Sub project C "Sealing techniques" – cements and procedures used for grouting are studied, both for normal conditions and adapted to difficult conditions and strict sealing demands, as well as natural sealing processes and water infiltration

More information about objectives and status for the different activities within each sub project can be found on our web-site www.tunneler.no or in internal reports from the Norwegian Public Roads Administration [1] & [3].

2. Grouting strategy for pre-grouting of tunnels (Activity C2/5)

Two of the activities within sub project C were joined. The activity C2/5 – "Grouting strategy for pre-grouting of tunnels" – aims to develop grouting procedures both for routine grouting and for grouting adapted to difficult conditions where the routine grouting is inadequate. The objectives for the two activities are described below:

C2 – "Standard grouting procedure for time efficiency":

- compile experiences of both satisfactory and unsatisfactory results from grouting of tunnels under normal conditions (including moderate sealing requirements where a routine time efficient pre-grouting has been used).
- develop a grouting procedure for optimisation of time and quality; *i.e.* fast progress combined with sealed rock, preferably after only one round of grouting. Methods based on conventional as well as innovative techniques (concerning procedure / grout materials / equipment / organisation etc.) will be studied.

C5 – "Grouting procedure adapted to difficult conditions and strict sealing demands":

- compile experiences of both satisfactory and unsatisfactory results from grouting of tunnels under difficult conditions (including strict sealing demands combined with limited rock cover, poor rock quality and / or complex tunnel design).
- develop sealing methods that are efficient at such difficult conditions. Methods based on conventional as well as innovative techniques (concerning procedure / grout materials / organizational structure etc.) will be studied.

Following sealing situations were the focus of the C5-project:

- High permeable rock mass and strict sealing demands for parts of the tunnel.
- Low permeable rock mass and strict sealing demands for parts of the tunnel.
- Zone with varying permeability and possibly demands on critical stability.
- Limited rock cover, possibly with overlying material sensitive to settlements.
- Two parallel tunnels or crossing over / under / close to other rock cavern facilities.

Phase 1 in activity C2/5 thus involved compilation of experiences from a selection of tunnel projects. The selection of tunnels was made according to the following criteria:

- Representative modern grouting strategy and range of methods.
- Grouting operations and results well accounted for.
- Relatively strict sealing demands.
- Extensive sealing efforts.

The six tunnel projects that were selected were all road tunnels, excavated in or close to urban areas, and often with limited soil- and rock cover or other difficult ground conditions. The projects were carried out between 1995-2001, when the focus on the consequences of possible groundwater depletion has been increasing. Finally, the selected tunnels had a high level of documentation, and it was possible to find data as well as representatives from the involved parties for supplementary information.

In short, the compilation of experiences was performed according to the following:

- Existing data and reports for selected tunnel projects were assembled and studied.
- Owner / contractor / consultant were interviewed to get complementary data.
- Preliminary report was compiled and distributed for double-check.
- Final report with concluding discussion was completed.

In the following, data and results for the selected tunnels will be presented. Further details can be found in a report from the Norwegian Public Roads Administration [4].

2.1 Tåsen tunnel, Oslo

The two Tåsen tunnels in Oslo are examples of tunnels with limited rock cover and a varying, sometimes highly permeable rock mass. Mainly sedimentary rock such as clay shale and limestone, interfaced by highly fractured igneous dykes (*e.g.* syenite porphyry and dolerite). In parts of the tunnel, grouting was performed based on the water leakage from probe holes, but in the vicinity of the syenite the pre-grouting was systematic. Sealing demands were relatively strict to moderate (10-20 l/min/100m). The documentation of the project was good, including for example a master thesis on grouting. An NGI report concerning the grouting, with emphasis on the pore pressure situation and settlements, summarized that systematic pre-grouting was performed for a total of 800 m in the two Tåsen tunnels. In some places, however, a substantial decrease of the pore pressure level over the tunnel had been observed (at most 7m).

Permanent water infiltration from wells in the tunnel was chosen to uphold the pore pressure. Key data for the Tåsen tunnel is given in the table below.

Tunnel length	933 m Ø – 937 m V
Tunnel design	Two parallel tunnels and several ramps, cross-section between 65-80 m^2
Excavated	1997-1998
Total leakage	25,7 l/min/100 m
Leakage restriction	When systematic grouting 10 l/min/100 m, or else 15-20 l/min/100 m
Average cement consumption	478 kg/hole – 24 kg/m hole – 802 kg/m grouted tunnel –26 kg/m^2 grouted tunnel – 870 kg/time

The sealing demands were met in the less difficult areas where a certain leakage from probe holes initiated grouting. In the highly fractured syenite, severe problems were experienced; *e.g.* drilling difficulties, grout outflow in the tunnel, limited grout penetration in the rock, and, finally, problems with rock support. Several changes were made for the systematic grouting programme, such as:

- leakage from probe holes instead of water loss measurements as criterion, but no change of the criterion despite high leakage and pore pressure decrease
- reduced length of grout fan, from 24m (outer) to 10-18m (inner) in difficult areas, grouting of the face through 10 m holes when large outflow of grout in the tunnel
- grouting pressure initially 20-25 bar, max. 35(-45) bar when low grout take
- larger drill bit diameter (64mm) to reduce drilling problems, but to no avail
- grouting cement (Rapid), w/c ratio 2,0-0,5, micro cement was tried but no clear conclusions on the result, post-grouting with polyurethane reduced the leakage

2.2 Svartdal tunnel, Oslo

The Svartdal tunnel in Oslo was selected due to limited rock cover (down to 2,5 m) and difficult geological conditions through the Ekeberg fault. The rock mass quality was poor; to the west highly fractured clay shale, then alum shale of very poor quality – "fragmented rock". The rate of excavation was very low in these areas, as a result of the need for substantial rock support. After having passed the zone of alum shale, there was gneiss of relatively good quality. Ironically, this is where grouting was required due to much water and uncertainty concerning the foundations of the buildings located over the tunnel.

The sealing demand that was placed on the tunnel by the owner in sensitive areas was quite strict (5 l/min/100 m). The principle for grouting was largely "Design as You Go", based on leakage from probe holes. In the two Svartdal tunnels, systematic pre-grouting was performed for a total of only 260m and the data was evaluated from the original grouting protocols. Key data for the Svartdal tunnel is given in the table.

Tunnel length	1700m N – 1450m S
Tunnel design	Two parallel tunnels with adjoining ramps, cross-section appr. 65m^2
Excavated	1998-2000
Total leakage	150 l/min, *i.e.* 4,3 l/min/100m
Leakage demands	5 l/min/100m
Average cement consumption	1358 kg/hole – 80 kg/m hole – 1719 kg/m grouted tunnel – 50 kg/m^2 grouted tunnel – 978 kg/time

The final total leakage measured for the Svartdal tunnel, including the critical passage of the Ekeberg fault, showed that the sealing demand was met. Since the leakage was not measured for sections of the tunnel, the distribution of the leakage is unknown and hence the grouting results cannot be correctly evaluated. At times, the systematic grouting programme was altered to a great extent, due to very limited rock cover and very poor rock mass quality. The main changes involved the direction and length of the grout holes due to the steep rock slope, other factors of interest were:

- leakage from probe holes initiate grouting, but the measurement of leakage from holes in the sole is difficult and may have led to underestimation of grouting need
- blast holes served as control holes for establishing the sealing result after grouting
- drilling difficulties necessitated shorter grout holes than the planned 21m

2.3 Lundby tunnel, Göteborg

The rock cover in the Lundby tunnel in Göteborg is from 5-35m, usually over 15m. The rock mass was mainly foliated granite of varying quality and permeability, with amphibolite and pegmatite dykes. Many of the buildings over the tunnel are founded on settlement sensitive ground. Key data for the Lundby tunnel is given below.

Tunnel length	2060m N – 2060m S – 238m ventilation tunnel
Tunnel design	Two parallel tunnels with 13 connecting tunnels, cross-section appr. 86-92m^2
Excavated	1994-1998
Total leakage	38 l/min, *i.e.* 0,9 l/min/100m
Leakage demands	600-2660: 0,5-2,5 l/min/100m
Average cement consumption	79 kg/hole – 5,9 kg/m hole – 476 kg/m grouted tunnel – 12 kg/m^2 grouted tunnel – ? kg/time

A water court order allowed a total of 135 l/min water leakage for the project, but the sealing demands placed by the owner were even more strict and varied for different parts of the tunnel (0,5-2,5 l/min/100m). Both pre-investigations and documentation of the grouting work had a very high level. Systematic pre-grouting was performed throughout the tunnel, appr. 4400m, and the average leakage for the tunnel 3 years after opening was measured to 38 l/min in total. During the excavation, the leakage was measured for sections of the tunnel. The grouting programme was planned in detail in advance, and had three sealing classes with a distance between the ends of the grout holes of 1,0-2,0m (62-30 holes) and hole lengths between 10-17m. With two 4-5m blasting rounds between grouting, this resulted in a substantial quantity of drilled meters. The grouting programme was also characterized by:

- water loss measurements of all holes, grouting starts in holes with most leakage
- two complete grouting rounds, *i.e.* every second hole in the grouting fan is drilled, water loss measured, grouted, then the cycle is repeated for the remaining holes
- leakage from probe holes or blast holes initiated supplementary grouting
- the direction of boreholes and leakage for sections of the tunnel was measured
- grouting cement, starting with w/c ratio 3,0, grouting pressure 25 bar+overburden

At Lammelyckan, with a rock cover of 5m overlain by clay in the surface depression and a very limited aquifer, a zone with poor rock mass quality and high permeability proved to be a big challenge. The sealing demand was 0,5 l/min/100m and permanent water infiltration was planned to secure the groundwater level. The grouting efforts included at least two complete rounds, grouting through control holes, and systematic post-grouting, but the sealing demands could not be met here.

2.4 Storhaug tunnel, Stavanger

The Storhaug tunnel in Stavanger was excavated in a rock mass that consists of different types of phyllites with very low permeability. For parts of the tunnel, the overburden was low, down to 3-4m in the area where grouting was performed. The leakage restriction was set to 3-10 l/min/100m; the lower value was specified under a peat moor area. In that area, systematic grouting was performed for appr.165m of the tunnel. A lot of documentation was available, key data is given in the table below.

Tunnel length	1260m
Tunnel design	One tunnel, cross-section appr. 85m^2
Excavated	1998-2001
Total leakage	1,6 l/min/100m
Leakage demands	1250-1550: 3 l/min/100m, 750-900: 10 l/min/100m
Average cement consumption	112 kg/hole – 8 kg/m hole – 1014 kg/m grouted tunnel – 26 kg/m^2 grouted tunnel – 273 kg/time

Leakage measurements in the tunnel in the summer '99 showed an average leakage of 1,6 l/min/100m under the peat moor. The satisfactory result was obtained by use of the systematic grouting programme, which involved the following elements:

- initially both water leakage from probe holes and water loss measurements, later water loss measurements left out due to lack of consistent information
- optimal amount of grout holes was found to be 62 (including 12 holes in the face) for the 85 m² tunnel and the hole length was 14m, with two 3m blasting rounds this resulted in a double cover and a substantial quantity of drilled meters
- micro cement (U12), Grout Aid and SP, low w/c ratio (1,1-0,4, usually 0,9-0,7), grouting pressure varying between 30-50 bar, maximum 70 bar in the sole

2.5 Bragernes tunnel, Drammen

The Bragernes tunnel in Drammen was excavated through volcanic rock mass, such as basalt and porphyry. The average rock cover varied from 10-150m, but was on average 100m. The rock mass was regarded highly permeable and the tunnel was located close to existing rock facilities. The maximum leakage was recommended to 120 l/min over 1200m, and the sealing demands were differentiated over the tunnel between 10-30 l/min/100m. Systematic grouting was performed for more or less the whole tunnel and the objective was to achieve acceptable a sealing result during <u>one</u> grouting round. The grouting work was well documented and data was also evaluated from the original protocols. Key data for the Bragernes tunnel is given in the table.

Tunnel length	2310m
Tunnel design	One tunnel, ventilation and escape tunnels, cross-section between 72-83m²
Excavated	1999-2001
Total leakage	10,1 l/min/100m, *i.e.* 240-1730: 8 l/min/100m and 1730-2540: 25 l/min/100 m
Leakage demands	400-800 and 1700-1900: 30 l/min/100m, 800-1700 and ventilation tunnel: 10 l/min/100m
Average cement consumption	2257 kg/hole – 68 kg/m hole – 1125 kg/m grouted tunnel – 38 kg/m² grouted tunnel – 2774 kg/time

The systematic pre-grouting gave an adequate sealing of the tunnel, the water leakage was measured to 8 l/min/100m in the critical areas with the demand 10 l/min/100m. Active design was used throughout the project, which led to several changes of the grouting programme based on mostly geological factors. To begin with, 21 holes with a length of 22m and two 2-3m blasting rounds between each grouting round, changed to only 7 holes of 27m for the same cross-section (72-83 m²) and 4 blasting rounds of 5m between. The grout take was extreme and the high pressure and volume capacity of the grout pumps was utilized in full. This facilitated the grouting to be performed in a highly standardised manner, which resulted in time efficiency. The grouting programme was characterized by:

- water leakage from probe holes was described as criterion for grouting (5 l/min from 1-6 holes), but grouting was performed for more or less the whole tunnel
- notice was to be given if the grout take in a single hole exceeded 1000 kg (later changed to 5000 kg, and after 3000 kg the W/C ratio should be lowered to 0,5)
- amount of grout holes was down to 7 for the 72-83m² tunnel, but in the last part of the tunnel (permeable with low overburden) the amount of holes was increased
- grouting cement (Rapid) and HP, low w/c ratio (1,0-0,5), high grouting pressure (80-90 bar with sufficient overburden, 20 bar in the last part of the tunnel)

At the Bjerringdal fault, with highly fractured and permeable rock, several grouting rounds were necessary from the same face and only 1-2 blasting rounds could be executed before the next grouting round. The grouting efforts also included longer holes in the first round in order to block off the water and 90 bar grouting pressure. In this zone, the grout take amounted to at most 60 ton in one grouting round.

2.6 Baneheia tunnels, Kristiansand

The system of Baneheia tunnels for the E18 in Kristiansand was excavated in a rock mass consisting of low permeable gneiss with dykes of pegmatite. The rock cover is 10-40m, and the tunnel with complex design passes only 19m below a very popular recreational area with three small lakes. The project attained large interest in media and extra focus was placed on the sealing of the tunnels, including pre-investigations. The leakage demand was set to 60 l/min in total (*i.e.* 2 l/min/100m), below the lakes either 6 l/min/100m for a 500m stretch or 12 l/min/100m for a single stretch of 100m.

Initially, the grouting was performed based on results from both water leakage from probe holes and water loss measurements. The water loss measurements were left out due to inconsistent information, and early on, systematic pre-grouting was performed. The documentation of the project was available and the data had been studied in two master theses on grouting. Key data for the Baneheia tunnels is found in the table.

Tunnel length	3000m in total
Tunnel design	Two parallel tunnels and several adjoining ramps, cross-section between 44-87m^2
Excavated	1999-2001
Total leakage	1,7 l/min/100m
Leakage demands	60 l/min in total, 6-12 l/min/100m near the lakes Stampene
Average cement consumption	256 kg/hole – 15 kg/m hole – 514 kg/m grouted tunnel – 14 kg/m^2 grouted tunnel – 755 kg/time

The total leakage measured for the Baneheia tunnels, 1,7 l/min/100m, indicated that the sealing demand was fulfilled. The leakage was however not measured for sections of the tunnel since the excavation was descending. Sporadic grouting was found to be insufficient and systematic grouting was performed for 95% of the total length. This led to a standardised grouting procedure with high capacity, well incorporated in the excavation cycle. Following elements in the grouting programme can be highlighted:

- optimal amount of grout holes was found to be 30 (including 4-7 holes in the face) for the 50-80m² tunnel, the hole length was 21-24m, with three 5m blasting rounds this resulted in an overlap of 9m and a large quantity of drilled meters
- objective was to achieve an acceptable sealing result during one grouting round, and the overlap made it possible to blast one round before drilling control holes
- micro cement (U12), Grout Aid and SP, low w/c ratio (0,9-0,7), varying but high grouting pressure, 50-80 bar maximum, lower close to the Stampene lakes

At a few zones with highly permeable rock, several grouting rounds were necessary from the same face and only 1-2 blasting rounds could be executed before the next grouting round. The grouting efforts also included reduced length of grout fan, from 17-24m (outer) to 8-15m (inner), and grouting of the fast-hardening Thermax. The distance between the two parallel tunnels was 4-22m and possibly the difference in excavation rate made the grouting easier; with lower grout take in tunnel no 2.

2.7 Concluding discussion

Data and experiences from six different tunnel projects, excavated during the last five years, have been compiled. The tunnel projects, all but one in Norway, were selected to provide a wide spectre of grouting situations at varying conditions. The rock mass / hydrogeology, and the overburden varies, as does the level of the pre-investigations performed. The sealing demands for the projects differ, as well as the reasons for the restrictions placed on the leakage.

Parallel with the increasing focus on possible consequences due to the excavations of tunnels in urban areas, the development of systematic pre-grouting has been evident over the last few years. The study clearly showed that, even for tunnels with moderate sealing demands, sporadic grouting efforts based on leakage from probe holes were insufficient. The use of a standardised grouting procedure throughout the tunnel, was pointed out as most advantageous for the excavation cycle. The development of a well-incorporated grouting procedure with high sealing capacity was characterized by the following elements:

- increased capacity and precision of the drilling helps provide the large quantity of drilled meters necessary for the new standard of optimal amount of grout holes
- enhanced use of superplasticizers and silica additives seems to have increased the penetrability and pumpability for both ordinary grouting and micro cements
- better penetrability has made it possible to use low w/c ratios, which in turn has improved the quality of the grout, and increased pumping capacity in dry cement

- mostly use of 2(-3) complete grouting lines (pump, activator, and mixer), better capacity for transport, weighing, mixing (limiting factor?) and pumping
- tendency towards use of higher grouting pressure in several projects, as much as 90 bar, resulting in better penetrability and grouting capacity

Further development of the elements described above, may provide a possibility to limit the number of grouts and mixes at the site for routine grouting. This can make the grouting process more time efficient and in itself contribute to increased capacity. The objective being to achieve an acceptable sealing result during <u>one</u> grouting round.

As for the grouting adapted to difficult conditions and strict sealing demands where the routine grouting is inadequate, the interim report [4] describes several interesting situations. Four of the projects have parallel tunnels and it may be an advantage with a difference in the excavation rate, not only for stability reasons but for the grouting as well. In the cases with limited rock cover, the arrangement of the boreholes and the grouting pressure are of course adapted. In rock of poor quality, or for tunnels with very strict sealing demands, the grout holes tend to be shorter. Details such as the unsolved problems with leakage around packers or through bolts (and a plan for how to deal with this) were discussed. Finally, the study placed some attention to general aspects like documentation, and there is a big potential for improvement of factors like; checking the rheology of the cement grout, measuring of water leakage in the tunnel, and evaluating the sealing results as well as the demands.

3. Future work

In view of the increased focus on the need for watertight tunnels in urban areas and environmentally friendly approaches, the timing for performing this study was good. The sealing strategies and range of methods for modern pre-grouting of tunnels have been established. Now, some of these experiences are being tested in the Metro Ring tunnel in Oslo and the results of this will be useful in future projects.

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PREPARATIONS FOR A LABORATORY STUDY ON THE PENETRATION OF FINE CEMENT GROUT IN SMALL JOINTS

FORBEREDELSER FOR EN LABORATORIESTUDIE AV FINKORNIG INJEKSJONSSEMENTS INNTRENGNINGSEVNE I TYNNE SPREKKER

Author: Anders Beitnes, research director, SINTEF, Norway

Abstract

Cement based grouting does still have possibilities for improvement, both in cost efficiency and in permeability reduction. Among other important efforts, it is sought to find the crucial parameters in grout composition, material grading and mix procedures. At SINTEF, a physical model of a fine rock joint is under construction. The joint is designed to be representing the limit for to-days cement grout penetration. It will be used to test the penetration of different grouts, changing single variables in each run. The paper describes this physical model, the considerations taken during construction and the objectives of the test programme.

Sammendrag

Berginjeksjon med sement kan og bør enda forbedres både mht. kostnadseffektivitet og tettingsevne. Det arbeides med forbedringer på flere felt, ett av disse er selve injeksjonsmaterialet, dets blandeforhold og tilberedning. Ved SINTEF bygges nå en fysisk modell av en bergsprekk, som antas å representere det som i dag er grensen for inntrenging med sementsuspensjon. Den skal brukes til å teste effekten av en og en variabel på inntrengingsevnen. Artikkelen beskriver selve modellen, de hensyn og krav som ligger bak utviklingen av den og det prøveprogram man tenker å gjennomføre.

Background for the study

Tunnel and underground projects are more and more facing new and strict regulations aimed at preventing inflict on the ground water balance. At the same time, for the reason of HES, stronger awareness has risen towards the use of resins or other chemicals. Cement grouting is still accepted when due care is taken to working conditions and disposal, but it is also widely accepted that there is a limit to the extent of tightness that can be achieved through contemporary methods. In some demanding situations, pre-grouting therefor seems to fail or be ruled out as an effective sealing concept. Much more expensive solutions or alternatives may be the result.

A lot is still to be improved, e.g. to utilise to its full the potential high grout pressure and perhaps foremost, denser hole patterns as effective means. In order to promote a further development of cement grouting results in demanding situations, it should also be possible to improve the performance of fine cement grouts substantially.

There is, however, a general lack of knowledge about the parameters in a cement grout mixture being significant for its penetration in fine joints. The "penetration ability" is not even an agreed parameter as such. Experiences from performed and suggested field studies certainly do give indications in direction of what works well and what does not. Field conditions do however prevent the obtaining of documented effects of single variables. Furthermore, it is not fully understood what goes on during curing in a thin cement mortar film at average rock temperature of 6 - 8°C. On the theoretical side, one has developed a growing understanding of fluid mechanics of suspensions in thin joints as well as theories on other subjects of probable importance to this matter, but they are only valid if they can be proven in practice. Also, a lot of practical tests for laboratory and field use are developed. A characteristic feature of these tests is, however, that they poorly reassemble the real thing during penetration in complex rock joints under high pressure. As long as the "science" of rock mass grouting is relying upon undocumented experiences and complex theory, substantial room will prevail for doubt, belief and personal opinions, as is in fact the case.

Hypotheses

Tests in a laboratory set-up should give the ability to prove a number of hypotheses put forward in recent discussions by a number of people who have both hand-on experience and scientific understanding of challenging rock grouting:

- Good penetration in finer joints is first of all achieved by use of raised grouting pressure, not by high w/c-ratios.
- Mortar suspensions with high w/c-ratios (>1) easily separate at high gradients and towards smaller channels, causing blocking and thereby damage to further penetration.
- Low w/c-ratios and stabilising ingredients/additives give the possibility to build up pressure and to enhance the distribution of grout to both smaller and larger joints at the same time.
- Efficient penetration of suspensions is only achieved when their maximum nominal grain size is about 0,2 times the nominal aperture of the joint.
- Low w/c ratios (<~0,8) and adequate cement types do give a sufficient early "strength", even at low temperatures, based on "setting", long before real (and slow!) hydration takes place.
- The limit of what is possible to penetrate with cement grout by use of to-days methods and materials, is an equivalent joint aperture of 0,1mm.
- Stability against "bleeding" is important, if not even crucial, and it is improved by use of e.g. micro-silica. But it is not agreed whether laboratory test methods do reflect the influence of a high pressure gradient.

Goal for the study

By imaging penetration in thin channels or joints under high pressure and controlled conditions, there are mainly two objectives, namely:
1): One wants to observe how factors like cement type, grain size distribution, w/c ratio, preparation of mix, additives and temperature separately (and together) act on penetration and on development of strength. Thereby it shall be possible to improve on materials and preparation procedures of grout mixtures.
2): By simultaneous tests and comparison, one wants to choose or further develop those field methods being most adequate for testing of grout mixture. Through this, it is a

goal to achieve one (or as few as possible) parameter(s) representing the penetration ability, and to be able to use it for specification and for QA/QC during grouting works.

Execution of the study

The physical performance of tests will be executed at SINTEF Civil and Environmental Engineering, by Dept. of Soil and Rock Mechanics and by Dept. of Cement and Concrete. Closely co-operating will be the consulting company NOTEBY.

Dept. of Soil and Rock Mechanics is hosting the physical set-up of the model. Development of the mechanics and the instrumentation has been going on for almost one year, partly integrated as a diploma work at the university NTNU. A back yard allows for set-up of actual full-scale mixer and pump equipment. The documentation of material parameters will take place in the neighbour laboratories of dept. of Cement and Concrete. That laboratory will also execute tests on strength development.

After hydraulic testing and preliminary grout suspension tests during the fall of 2001, a commercial programme will be offered to suppliers and other parties. Parameters and test specifications will then be further developed together with their R&D people. Development of field tests may be a mutual endeavour.

Requirements of the physical model (test set-up)

Confinement stress: The planar discontinuity or void representing a rock joint must be contained inside a body which is firmly held together by an outer force or stiff frame. A perpendicular and evenly distributed stress should represent a real effective stress. Tailored variation of this confining stress and its response character (elastic/elasto-plastic) is preferred.

Joint wall material: Chemistry and water affinity should be close to a variation of normal rock mineral properties. Furthermore, it must allow preparation of geometrical form at high accuracy and reproduction. Transparency will allow visual observation.

Joint aperture and roughness: It is important for the representation of a real rock joint to have some roughness. Firstly, the detailed roughness should be of some scale, say 1micron. Secondly, the gap should vary and undulate, forming wider and narrower channels in at least two directions. As a first part of the study, it is an important experiment to find a good layout or pattern. Expected variation could be within 20 and 200 micron to represent a rock joint, which is at the edge of to-days "groutability" by use of cement. Accuracy and reproduction must supposedly be within 20 micron. As a third requirement, it must be a firm stress-transferring contact between the two walls such that no crushing or permanent deformation takes place.

Global dimensions: For a realistic model, one would want a size representing an area of rock joint to be grouted from one or two drillhole intersections in practice. On the other hand, one must be able to reproduce the model's walls in an available grading bench. Furthermore, reasonably high grout pressures (which we certainly want to study) will

give huge suspension forces in a large model. As a compromise, it is sought to have a model of a joint area around 0,5 –1,0 m^2.
The model should also be symmetrical.

Boundary conditions: The model should allow beginning all tests with realistic saturation as well as in-situ pore pressures and gradients. This pore pressure should be controlled during execution of tests. Collectors at the rims must be part of the channel pattern, allowing through-passing grout to be measured and inspected. An advantage would be to observe the difference in water conductivity from one rim of the model to the other, before and after grouting.

Temperature: It is very important to govern the temperature of the whole model body throughout a test programme. No less variation than +/- 2° C from actual rock temperature should be allowed. For instance, it will be sought to represent Nordic conditions by an average of 8° C. Other temperatures will be introduced dependant on the purpose of specific tasks.

Monitoring: Accurate and true-time measurements of pressure distribution is of course a necessary and important features of the set-up. One should be able to monitor the distribution of pressure in several points around the joint plane. Measuring range may be 0 – 5 MPa, with high resolution. The flow rate at in- and outlet are also interesting values, even if the figures may be very small. A couple of "spy's" to control joint opening close to the centre hole as well as monitoring of stress and strain in the suspension frame are suggested. That will give the possibility to make controlled "forced" grouting.

Test layout

After careful discussions and consideration, one has made a choice of model set-up with almost no discrepancy to the requirements. In addition, one has chosen to use put an effort in being able to visually observe the penetration, and to measure the degree of joint filling after the grout has set and the hydraulic testing is finished for each test run.

At the SINTEF laboratory, a facility developed for rock bolt testing conveniently was at hand. In a rigid steel girder frame, the joint model is mounted between two separate cubic concrete blocks of 1m^3 each. A workshop winch and in-built rails in the frame make it easy to lift and slide the heavy parts into position. By use of threaded bolts, a chosen confinement stress is applied through the concrete blocks.

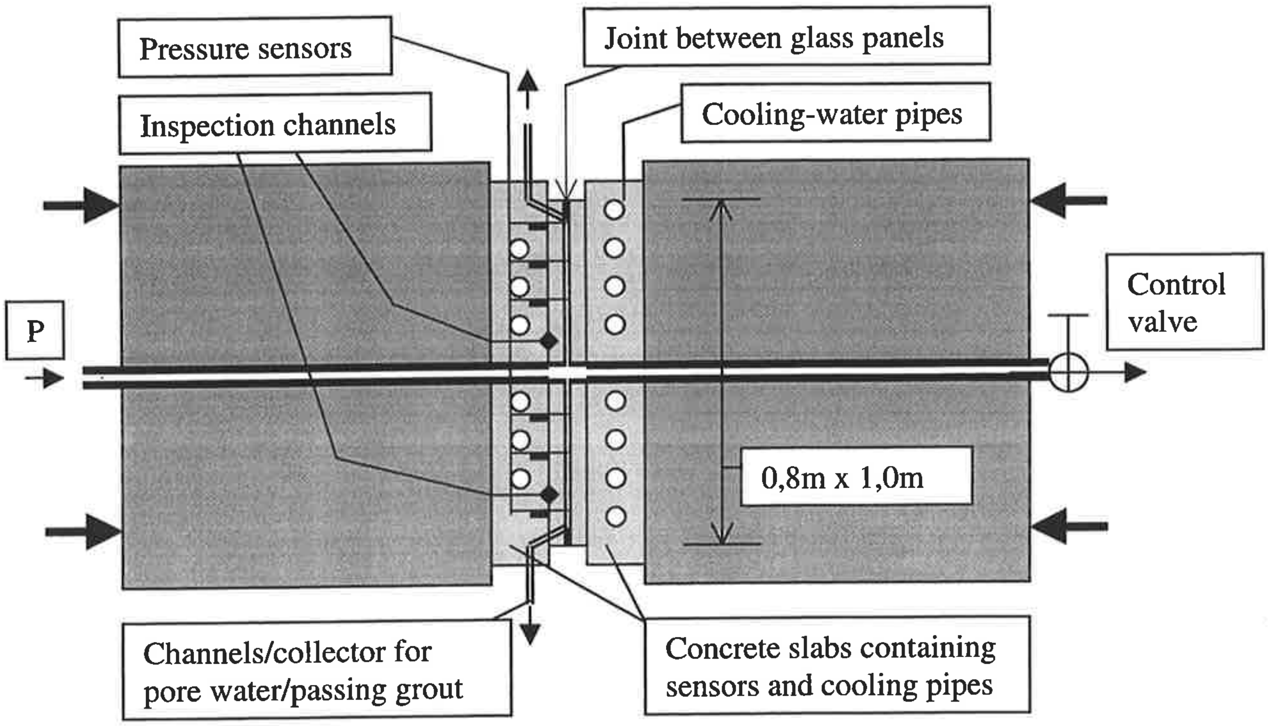


Figure 1. Schematic sketch of main test set-up

The model joint is a distinct rectangular plane between two planar, prepared glass panels. One is planar and equipped with sensors and rim collector channels along the short sides. 10 pressure sensors and 10 light-diodes are being mounted. Channels for visual inspection are prepared on the rear side of this panel. The countering plane will be prepared as a "landscape" with "valleys" graded into the surface by a diamond tool in a data-operated bench. Limitations in this machine has led to a dimension of the panel of 0,8 times 1,0 m. The pattern will leave more or less open channels in two diagonal directions when the panels are put together. Approximately 30% will rest ungraded and act as stress contact areas.

For each test run, the panels are assembled as a "sandwich", with each of the two concrete slabs containing circulating cooling water in fixed positions.

The packer design represented a special challenge. Ordinary packers are approximately 150mm long, and one such mounted from each side, would scarcely reach into the glass panels. By tightening, they might burst open a crack between the glass panels and the concrete slabs. It was considered to weld on glass manchettes or tubes as elongated hole sections to house the packers, but a better idea came up. An ordinary packer was split into two sections, and a free-moving double washer with distance slabs was set in the middle. It corresponds to openings in the centre tube. By tightening now, it will act more like an extra confinement.

Test series

In an early stage, experimental trials will be run, where good procedures and optimum design of glass panels, monitoring and other elements are developed. After that, standardised test will be performed for different grout mixes and other variables at a pace of at least one run per day. As direct measures of penetration ability, one will be able to obtain factors like gradient build-up, flow rate (?), degree of void filling and change in grout consistence. Also sealing success can be measured by changes in transmisivity across the joint model before the test and after the grout mortar has undergone sufficient setting.

Interesting factors by the cement mortar to vary for the research may be:
- cement mineralogy, also modified with other mineral substances
- grain distribution, tail of fines, maximum grain size
- water to cement ratio
- additives for plasticity, stability and acceleration
- mixing and preparation procedures and equipment

All test runs will start with water testing, both in and back trough the centre hole, and by transverse flow through the model.

Grouting will be performed through a centre hole running all trough the Sandwich" and both blocks. By gradually closing this flow at the far end for pressure build-up, a fair resemblance of real grouting in permeable rock is obtained.

In order to investigate the effects of "forced injection", some tests will be run by use of pressures that will actually split the model joint open.

Parallel tests

Two types of parallel tests should be run for all tests in the main research programme:
- tests of possible field apparatus and procedures,
- tests to measure the development of shear strength.

Field apparatus may be existing or new types under development in Sweden.

A simple shear box for measurement of strength development at the actual joint dimension and temperature should be easy to develop.

GROUTING IN SALMISAARI UNDERGROUND COAL STORAGE

Injektering i Salmisaari underjordiska kollager

Juha Salmelainen, Kalliosuunnittelu Oy Rockplan Ltd
Jarmo Roinisto, Kalliosuunnittelu Oy Rockplan Ltd
Jyrki-veli Aartolahti, Kalliosuunnittelu Oy Rockplan Ltd

1. The need for a new coal storage

The project was initiated due to the short-age of suitable land for office and business buildings in the Salmisaari-Ruoholahti area in Helsinki. The city decided to make a new town plan and to re-zone the current coal storage field of the power plant in Salmisaari for office and business facilities, a marina and sports facilities. The focus was in the future to create a new, dynamic centre of expertise in the heart of Helsinki.

Starting in 1999, the preliminary phase of the project was carried out by a working group, with participants from Helsinki Energy and the Real Estate Department, City Planning Department, City Office and Environment Centre of Helsinki. The most effective storage solution out of a range of proposals was designed by Kalliosuunnittelu Oy Rockplan Ltd.

2. The new storage solution

The new construction consists of four underground vertical silos which will house 250.000 tons of coal. The silos will be 40 meters in diameter and with the height of 64,5 meters. The volume of one silo is 76 900 cubic meters. Around the silos a network of belt conveyor tunnels, service tunnels and shafts make the storage operation possible. In the course of the project, some 3 000 metres of tunnel will be excavated. The total length of shafts will come to over 400 metres.

Excavation for the four underground coal storage bunkers will take place within 150 meters of three power plants, metro tunnels, coal transport and additional service tunnels. This is one of Finland's largest underground projects, a unique excavation, with 625.000 cubic meters of rock to be removed. From the ordinary person's point of view the imposing mountain of coal by the side of the busy highway running west from Helsinki will disappear into four underground silos.

Each silo will have the capacity for storing around 60 000 tons of coal. The total consumption of coal at the Salmisaari plant is around 450-500.000 tons annually.

When the silos are completed, ships carrying coal will unload their cargo at the Kellosaari coal terminal in to the drop shaft, whence the coal will be transported by conveyors to the silos, filling them from the above. Tranportation of goal directly to the plant is also possible. Inside the silos special storage equipment will spread the coal into a level bed. From the bottom of the silos, the coal will be moved by a horizontal belt conveyor to a vertical conveyor, which will hoist the coal up from a depth of about one hundred metres to the power plant itself. The movement of coal is fully automated. Similar silos are in use around Europe above the ground, but this is the first time that such spaces have been used for storing coal underground. Special measure will be taken to reduce the risk of spontaneous combustion will be reduced, increasing fire safety. The silos will also have foam as well as inert gas (N_2) extinguishing systems to maximise safety.

The new storage facility will consist of two additional parts: a eletrical switching station and a district cooling plant. The refrigeration plant will be important for the businesses in the area, as it will produce cooling air for their equipment. The three parts of the project will progress separately but in step, which places great demands for the management and control of all operations.

About 300 million marks (over 50 million euros) have been budgeted for the project of which 210 million will come from the City Real Estate Department and the rest from Helsinki Energy.

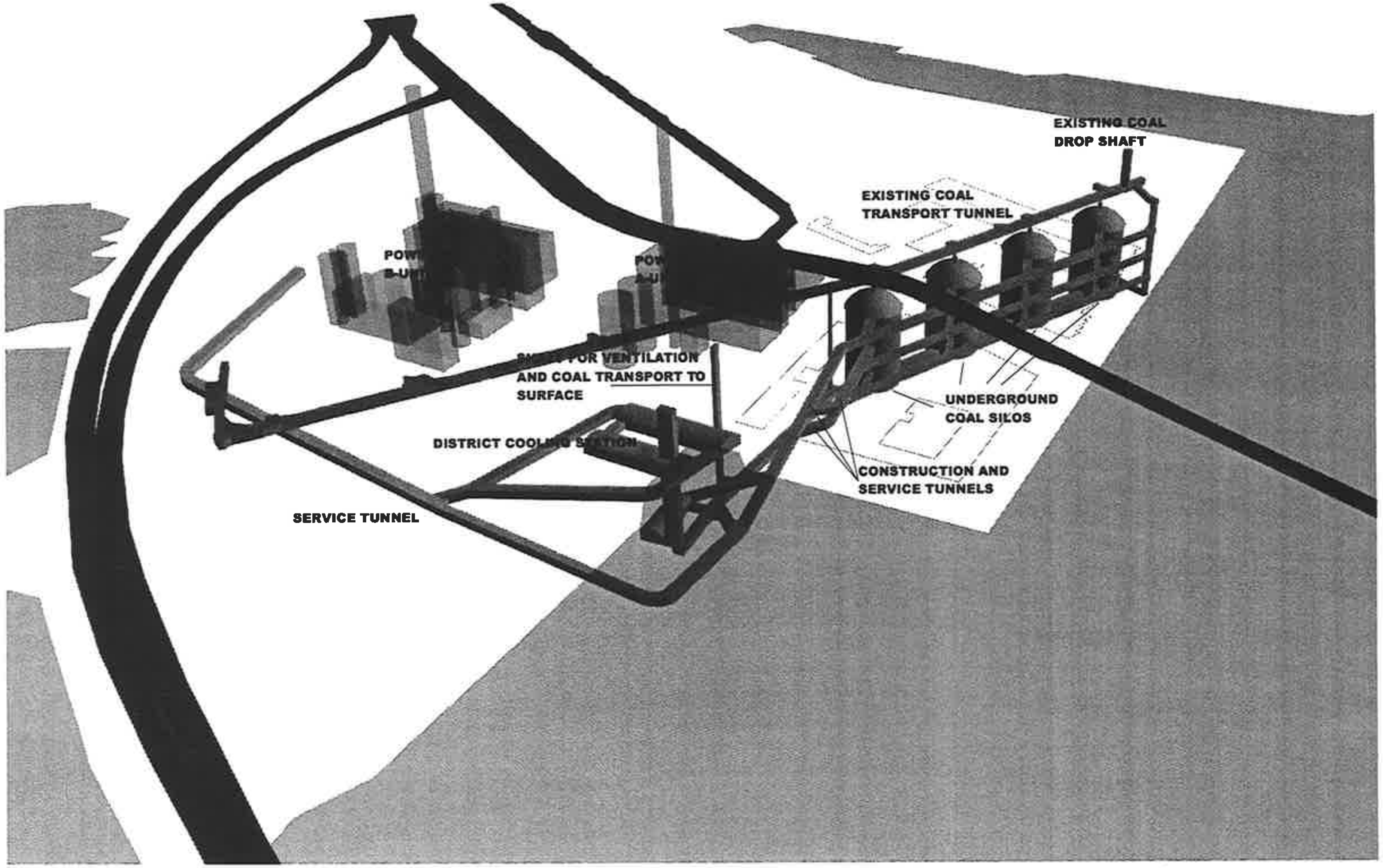
Helsinki Energy has commissioned Kalliosuunnittelu Oy Rockplan as chief designer, and Hiili, a consortium comprised of Lemminkäinen Construction and YIT, to began excavation. The excavation, construction and installation work will be carried out during 2001-2004.

3. A great excavation task

The contractors are expecting the excavation of the four silos to be a challenge. The size of the silos is very impressive, with a diameter of 40 metres and a height of nearly 70 metres. Their large volume also places great demands for ventilation. Pressure naturally increases the deeper one goes underground. In the silos, correct pacing of the work, reinforcement and optimal scaling of entrances and terracing are very important, highlighting once again the importance of advance planning.

The work started with ordinary excavation work of the future switching station, even though the underground oil tanks and a functioning power plant are located right next to the site. Accordingly, blast charges had to be planned with extra care. As the excavation progressed, the planned metro tunnels as well as a joint service tunnel are to be

SALMISAARI UNDERGROUND COAL STORAGE



18.1.2001

Helsinki Energy

KALLIOSUUNNITTELU OY
ROCKPLAN LTD

excavated. Site investigation of the rock mass showed that the rock to be excavated is less easy to drill at the beginning of the service tunnel than around the silos. In the course of the project, the service tunnel will branch out on four different levels. In practice, the uppermost level, the work tunnel, will also serve as the silo top service tunnel. The two tunnels in between are required for excavation, and will in the future house ventilation ducts for the silos and provide a route for maintenance.

The silos will be excavated by a crawler drill rig using in either 64 mm or 76mm hole diameter with extension rods and guide tubes as well as drill bits from Sandvik. Water flushing in bench drilling using Tamrock's Ranger rig will add an inter-esting extra dimension to the work. Excavation work will be completed in June 2003.

Various tests were performed prior to the start of excavation, which included risk analysis of borehole blasts to review how vibrations would be transmitted throughout the neighbouring tunnels, businesses and residences; these provided blasting limits that were passed onto the construction contractor, Hiili. As a result of the tests, vibration protection for approximately 2000 computers was required.

4. The need for grouting

4.1 Environmental demands

The city authortities were worried about the ground water conditions between excisting oil caverns and the planned silos. Currently the western end of Helsinki metro line in Salmisaari is between the oil caverns and the new silos. Later in the future the metro tunnels will be excavated further westward to city of Espoo. There was need to give require consideration to water tightness of rock joints caused by the excavation of silos.

Another issue was task is in the operations side: leaking groudwater brings radon in to the tunnel air. To minimize this radon problem the amount of leagage water must be minimized, open water surface be avoided and ventilation must be level while people are working in tunnels.

4.2 The rock conditions

Site investigations were carried out in several stages and by a wide rage of methods. The most widely used method was core drilling with the water pressure tests. Althougt the main focus was in the silo area, an intensive drilling program was carried out also in the site of the district cooling plant caverns. The results indicated high permeability figures (over 20 Lugeon) in several core drilling holes with otherwise firm and good rock nearby. The reason was few incline, thin fracture zones conducting water from the Baltic Sea.

The silos are located under the original Salmisaari island and in that area rock joints are exceptional watertight. Every silo was crossed with several core drillings and no notable values of permeability were found. The same observation was made by inspection of the existing conveyor belt tunnel where the southern area is very dry and northern area suffers water ingress.

4.3 The grouting solutions

The water proofing is to be mainly carried out using pregrouting technics. Systematic pregrouting is designed for all silos, all shafts and all building services spaces. Further more at the start of the service tunnel there was a thin rock cover to the underside of a water channel for the power plant and that area was also designed to be pregrouted. In tunnels probe holes are to be drilled ahead of the face, and in the case water entry, the tunnel face will be grouted.

In total it is planned that over 10.000 meters pregrouting holes will be drilled from the surface and tunnels around 30.000 meters from the tunnels. The grounting material is to be cement based.

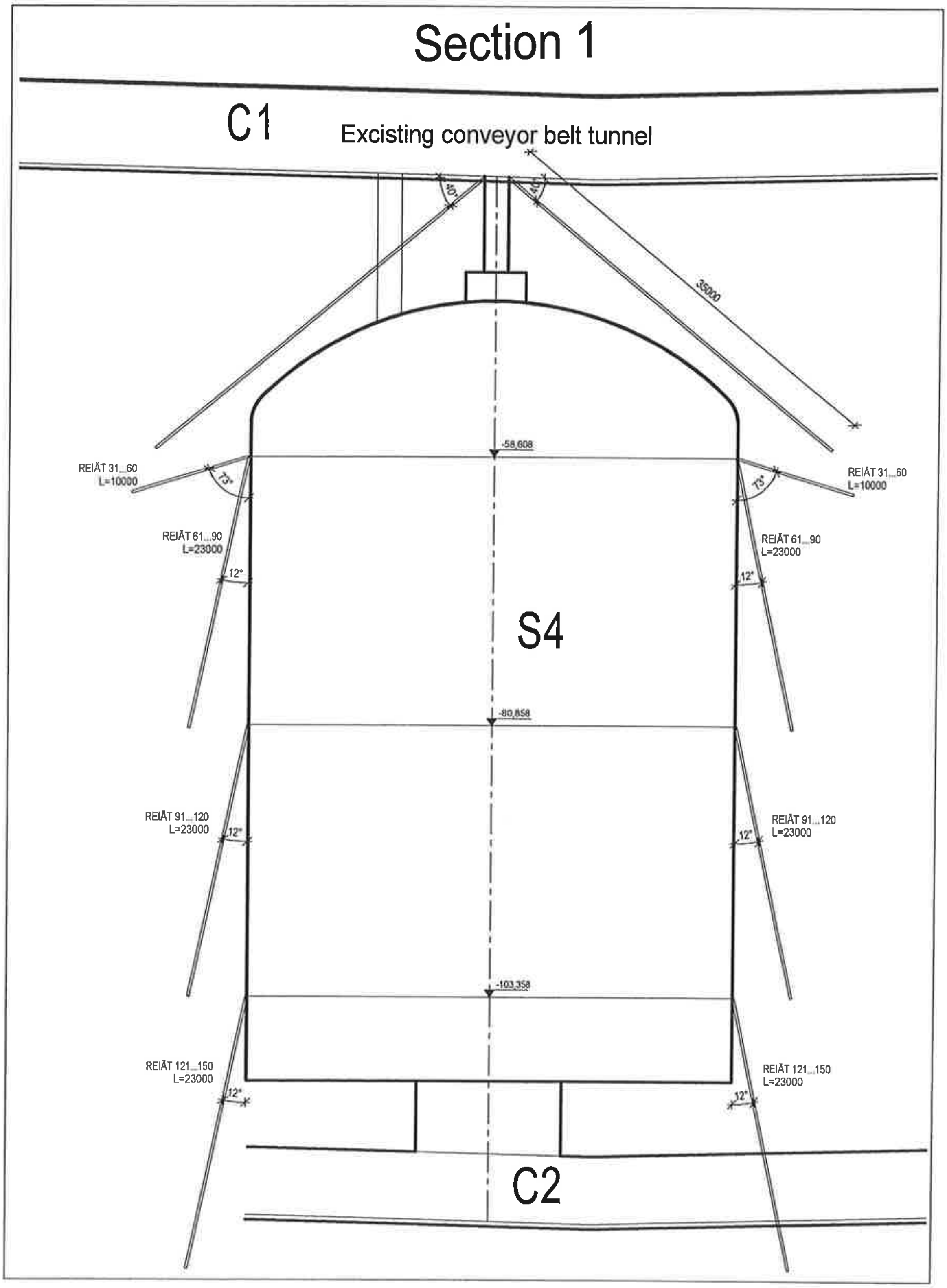
The contractor is using extension rods in injection drilling to avoid the need for conversion couplings. The hole diameter in injection drilling is 54 mm.

4. The current project status

During the first 400 meters of tunnel excavation considerable difficulties were faced. The reason for this was fracture zones of high permeability in the tunnel roof for a distance of 70 meters. Probe holes were some places impossible to drill 24 meters long. The holes were collapsed. Part of grounting holes in some spans were uncompleted. The rock face took grouting material normally. Totally the grouting rasult was not satisfactive and dropping waterpoints were left to be handled by postgrauting. Even the drilling and charging of shot holes left incompleteat some location. Result was that the contractor was about 50 days late on the schedule after 6 months period.

More sofistigated methods for grouting was designed. The grouting pressure was raised up to 4 MPa, a finer grout and appropriate working technics was introduced. The graut was changed to use ultrafine cement, GrautAid and SP40.

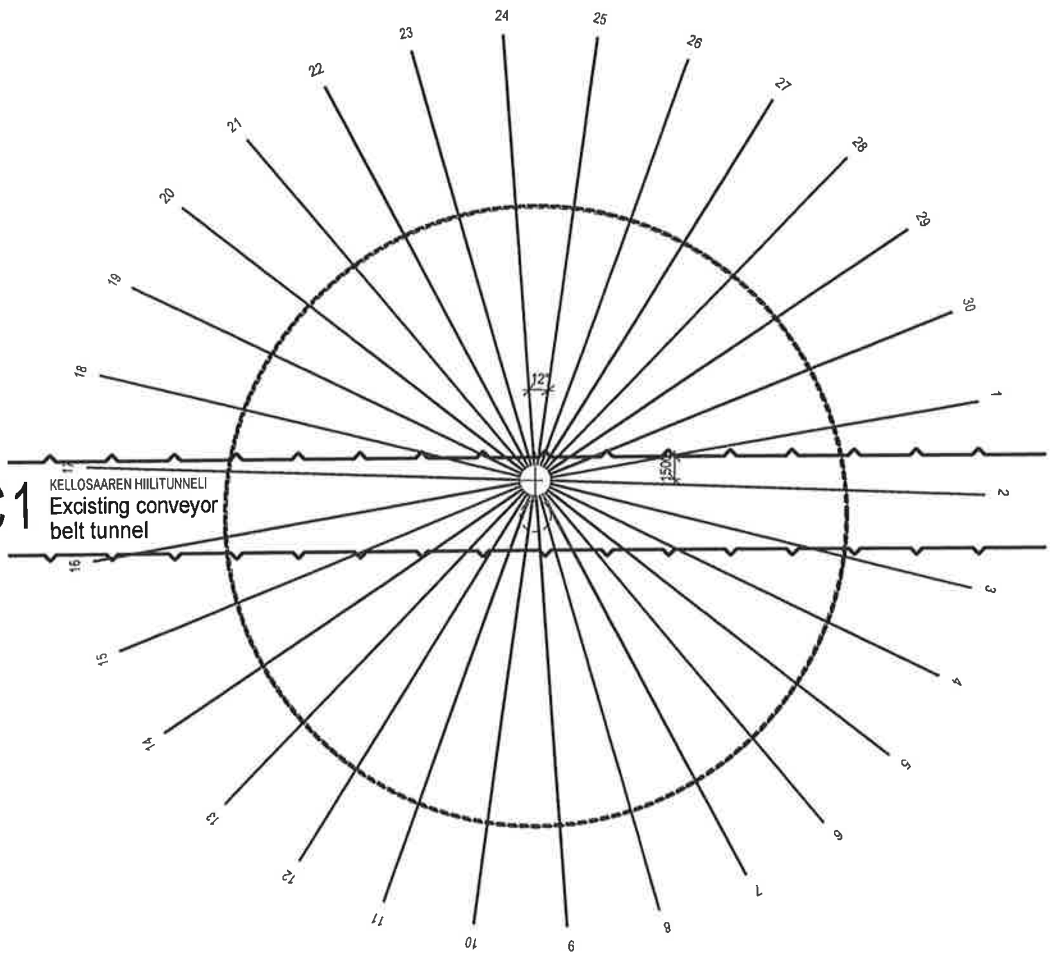
So far over 800 metres of tunnel and about 75 % of the district cooling plant caverns have been excavated. After the first 400 meters, the rock has been good as it was predicted, witout water ingress along joints. And now the first silo top is in 100 meters.


Section 1
C1
Excisting conveyor belt tunnel
40°
40°
35000
-58,608
REIÄT 31...60
L=10000
73°
REIÄT 31...60
L=10000
73°
REIÄT 61...90
L=23000
12°
REIÄT 61...90
L=23000
12°
S4
-80,858
REIÄT 91...120
L=23000
12°
REIÄT 91...120
L=23000
12°
-103,358
REIÄT 121...150
L=23000
12°
REIÄT 121...150
L=23000
12°
C2

PLAN 1

Grouting of rock roof

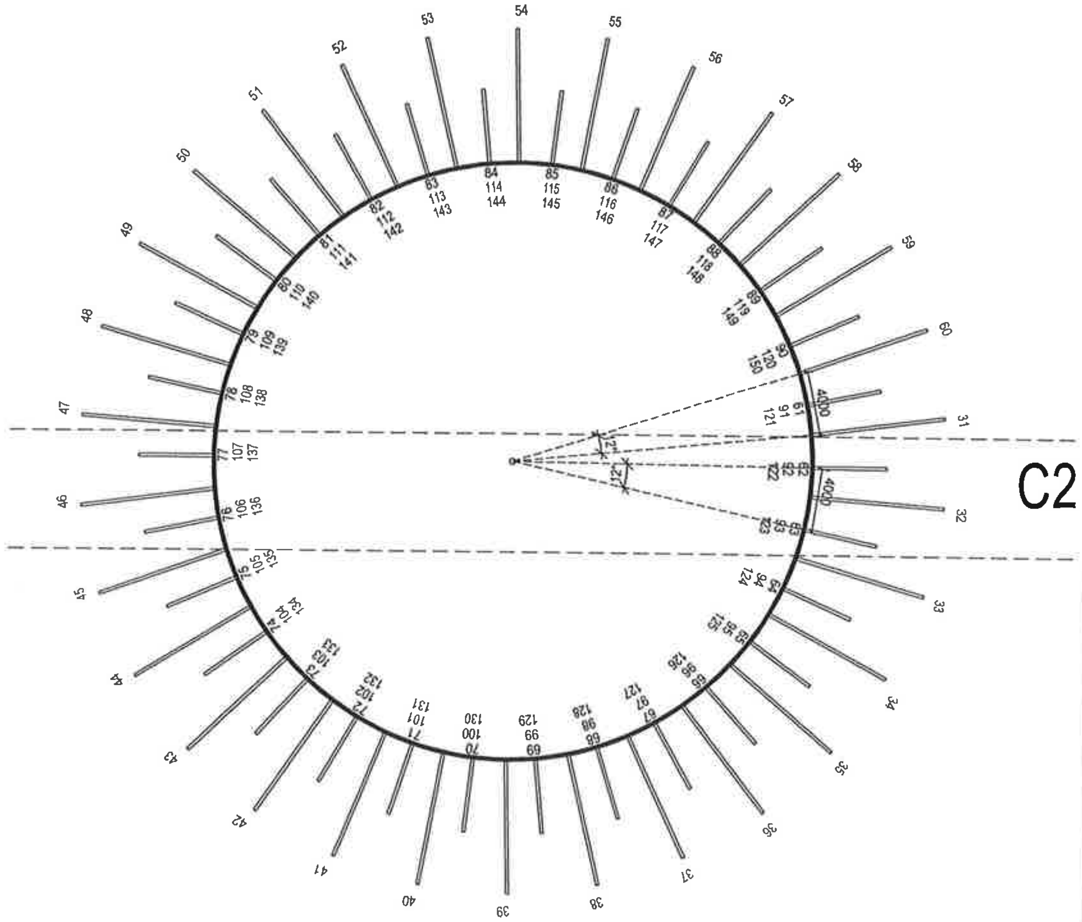
HOLVIN INJEKTOINTI



PLAN 2

Grouting of walls

SEINIEN INJEKTOINTI



REPAIR GROUTING OF SHOTCRETE COVERED ROCK WALLS IN LARGE ROCK CAVERNS

Efterinjektering av sprutbetongtäckta bergväggar i stora bergrum

Ingvar Bogdanoff Geo Flex AB

Abstract

After the excavation for the Royal Library in Stockholm city was completed, leakage was experienced from the high vertical walls. The grouting repair became difficult since the leaking spots in the high walls were situated behind the shotcrete support. Different methods to establish hydraulic contacts with the leaking paths were tested, such as dense percussion drilling, light core drilling, hydraulic fracturing and long hole core drilling. Long hole core drilling in combination with slow setting microcement turned out be successful. A lot of repair grouting was performed from the narrow space between the rock and the book storage walls. From the beginning only PU grout was used, but later on, the microcements dominated. Grouting operations in the 150 m long narrow space between the building and rock walls required long grouting hoses. To avoid clogging in the long hoses, a slow setting cement grout was preferable. The long core drilled holes were all grouted with microcement, combined with PU resin in order to repair surface leakage to the rock wall. The PU resin was used to seal visible leaking holes and surface leakages from the remedial holes. The PU grouting was often performed with high grout pressure, 8 - 14 MPa. The high pressure is possible in case of small leakages. After the remedial grouting the leakages was reduced to acceptable level.

Sammanfattning

Efter att bergentreprenaden av bokmagasinen under Kungliga Biblioteket avslutats konstaterades läckage från bl.a. bergrummens väggar och golv under bokmagasinen. Läckagepunkterna i väggarna var ofta täckta av sprutbetong och svåra att lokalisera. Efter ett omfattande injekteringsarbete kunde slutligen konstateras att läckaget minskat till en acceptabel nivå. Och att bergrumsväggarna i stort var torra. Till stora delar måste arbetet utföras i det trånga utrymmet mellan de höga byggnaderna och bergrumsväggarna. Detta arbete möjliggjordes av bl.a. en kärnborrutrustning som kunde borra långa hål uppåtriktade hål från det trånga utrymmet. Försök gjordes att hitta läckagepunkterna bakom sprutbetongen med tät handhållen bergborrning och med korta smala kärnborrhål borrade med lätta utrustningar. Den svåra åtkomligheten gjorde vidare att cementinjektering måste utföras via långa injekteringsslangar med igensättningsproblem av slangar. Detta ställde krav på långa geltider för de använda cementsuspensionerna. Från början användes polyuretan som injekteringsmedel. Efterhand kom mikrocementer att dominera efterinjekteringen. Injekteringen utfördes också som saminjektering med polyuretan och cement. Polyuretaninjektering utfördes ofta med höga tryck. Denna teknik har betraktats

som kontroversiell. Emellertid ger små läckagevägar och snabbt gelande injekteringsmedel utrymme för höga tryck utan att skapa stabilitetsproblem. Polyuretaninjekteringens fördelar var framförallt vid reparation av synliga injekteringshål, reparation av ytläckage från efterinjekteringshål samt injektering i schakter. Cementinjekteringens stora fördel var dess förmåga att sprida sig över stora ytor från kärnborrhål som sträckte sig utanför den förinjekterade zonen. De långa kärnborrhål som borrades snett uppåt från bergrumsgolvet gav lyckade injekteringsresultat.

Introduction

Two large rock caverns for book storage were excavated under the Royal Library in Stockholm city. Figure 1. After completion of the excavation, severe leakage was observed mainly in the walls but also in the floors. The leakage in the walls appeared to originate from the former bench levels. After closer examination it was obvious that part of the leakage was due to leaking grout holes, now concealed behind a 30 mm shotcrete layer on the walls. The leakages displayed as a range from moist areas on the shotcrete, to spots of heavy flow from particular grout holes.

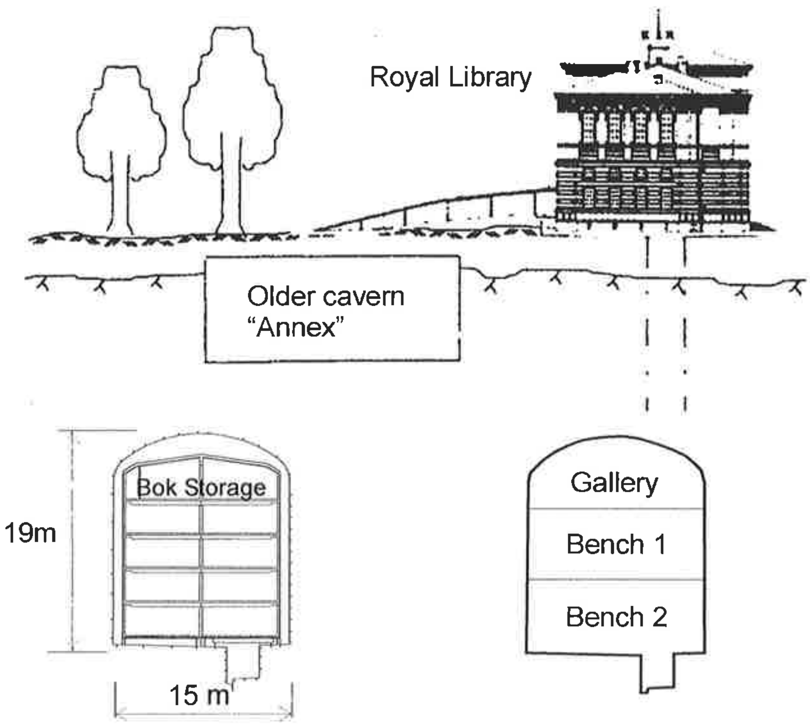


Figure 1. Section of the caverns. The two book storages are 150 m long

Excavation and Grouting

The galleries were grouted with in a more or less standard manner, grouting with three rounds before the front with checkpoints after each grout operation. The result became in most parts very good. With a few exceptions, the rock conditions were very good. The excavations of the benches were performed with horizontal drilling in same manner as is normally used for tunnelling. The grouting was designed for horizontal drilling covering four rounds before the front and blasting two rounds thereafter. Thus the grouting overlaps two rounds. This method has a well-known disadvantage. The grouting interferes with the excavation work.

To gain time a new grouting variant for benches was proposed. The main difference was that the grout holes were designed downwards instead of horizontal. Figure 2.

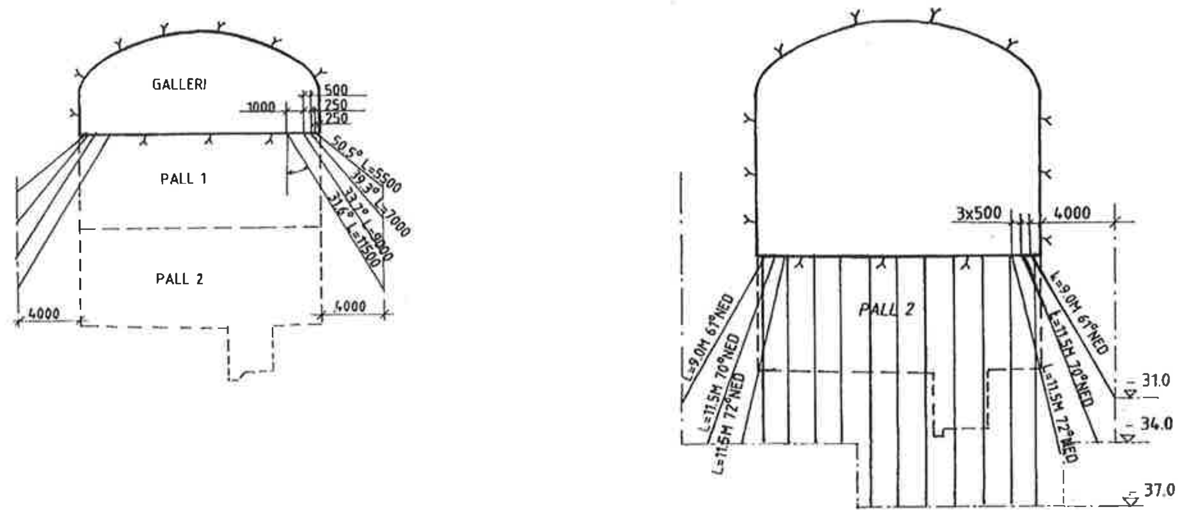


Figure 2. Section of the grout holes for the benches during excavation stage.

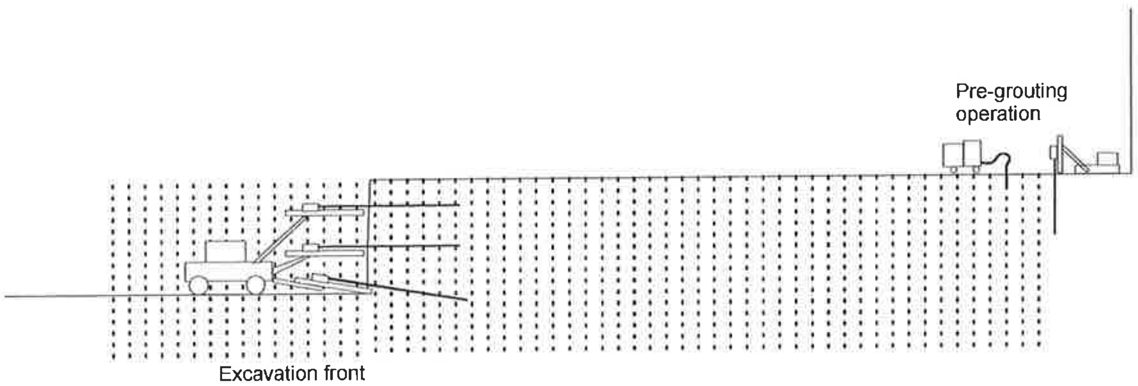


Figure 3. With vertical drilling for grouting, the excavation can be performed without interfering with the grout operation.

Vertical drilling of the grout holes for the benches made it possible to excavate the benches independent from the grouting. The grouting operations could be performed far ahead of the blasting operations without interference. As the method was implemented to the project leaking grout holes were not observed and sealed before shotcrete covering of the walls.

There can be different reasons for leaking holes. But it was a fact that hole fillings could be found partly destroyed. As a result of this, the holes acted as effective drains outside the grouted zone. Figure 4.

The cement used for grouting was the SH-cement, a fast setting Portland cement with grain size up to 0,125 mm. This cement has been used for tunnel grouting in Sweden for decades, before the use of microcements. The cement grout seals however not all thin fissures, which form leakage paths to the grout hole. Therefore, hole fillings after the grouting operation must be watertight.

After the packers were dismounted from the downward inclined leaking holes, it was almost impossible to detect the leakages in the bench floor, in the dark environment with mud and water.

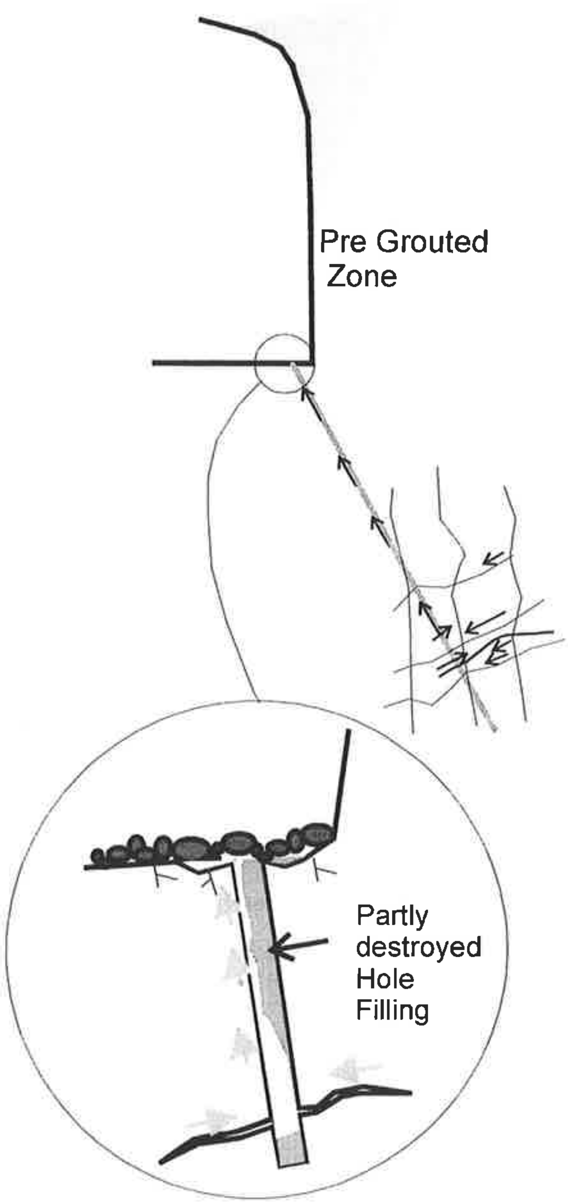


Figure 4. Grout holes with leaking hole fillings convey water from the area outside the grouted zone via thin fissures not penetrated with the cement grout. The leakage is difficult to observe on the floor covered with mud and water from drilling for grouting and bolting operations

When the excavation of the benches continued the leakages in the bench floors became located in the cavern walls.

Shotcrete applied on a rock wall has a very little water tightness. It may however have the effect to lead the water to a nearby crack in the shotcrete through the interface between the shotcrete and rock wall. The leakage appears then in other places than the hole opening. This may occur as well for a leaking fissure undulating through the pre-grouted zone.

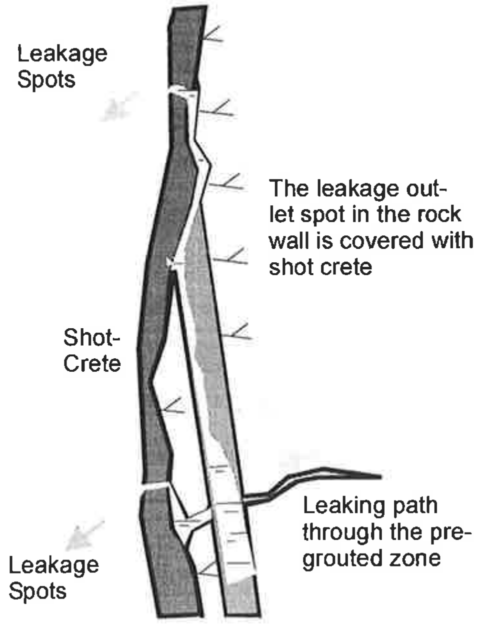


Figure 5. In case of leakage to the shotcrete covered rock wall, the shotcrete may steer the water flow from the leaking grout spot in the rock wall to a place situated far from the hole opening. Thus the leakage outlet in the rock surface is difficult to locate.

Possible Reasons for Leaky Vertical Grouting Holes

Vertical drilling and grouting is well known from curtain grouting of dams. But it is seldom used for underground facilities. As the technique was applied in this case, it differs from the horizontal grouting, where the result is visible after the first blasting round, or even before, if control holes are drilled before blasting. Supplementary grouting is performed if leakage is found.
With the vertical grouting far ahead of the front such controls were very sparse. Working operations such as drilling for grouting and bolting and shotcrete operations on the bench, above and ahead the front produced lot of water to the benches. Part of this water could probably flow to the front. Water on the excavation front may then have not been regarded as leakage, but to originate from other operations on the bench.

The reasons for leaky grout holes may have been that, the packers can have been dismounted before the grout was strong enough to stand against the ground water pressure. Early dismounting of packers may have been made after the holes were filled with low w/c = 0,5 cement suspension after reaching the prescribed grout pressure. Figure 6. If a packer is dismounted from a vertical hole filled with grout, the groundwater builds up a pressure and slowly makes intrusion to the hole. This process takes long time and is probably not visible on the floor, before the surrounding area to the hole is covered with mud, stones and dirty water.

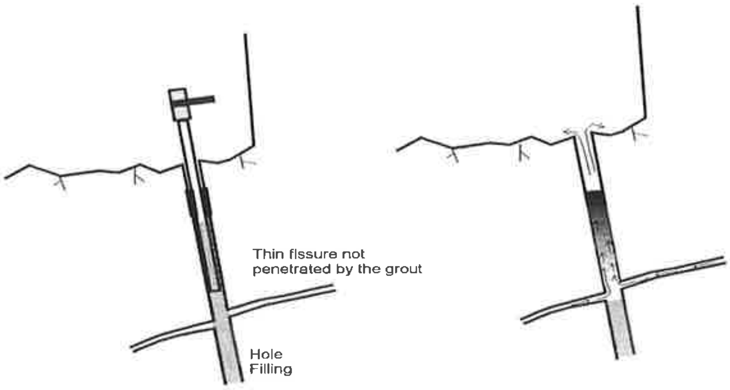


Figure 6. The hole filling destroys if the packer is dismounted before the grout has set to gel. The hole-filling column of grout cannot resist the ground water pressure. Due to counter pressure from the grout, the intrusion of water to the hole is slow.

Remedial Grouting Technique for Cavern Floors

Before the building was constructed in cavern 2, figure 2, leaky holes could relatively easily sealed by inserting a packer into the hole. But in cavern 1 and later on in cavern 2, the buildings prevented access to cavern floors. The cavern bottom parts were then grouted from via horizontal holes from the trenches below the buildings. To prevent surface leakage to the floor, cement grout was combined with 1-component PU resin. The PU resin was injected via a T- valve on the packer.

Remedial Grouting Technique for Visible Leaky Holes

Sealing of visible leaky hole openings was performed with PU resin injected via short 13 mm inclined holes to high pressure, 8-14 Mpa. The PU holes were drilled from the side to the larger leaky percussion holes. The hole openings were sealed first with wooden plugs. This process was fast and effective and was by routine performed to the horizontal remedial holes, which often became leaky after the end of the cement grouting.

Remedial Grouting Methods to Seal Leakages Covered by Shotcrete

When the remedial grouting started, the building in cavern 1 was already erected. The construction of the building in cavern 2 started a few weeks after the remedial grouting. During the first weeks, the grout operations could be performed in the high rock walls from sky lifts. But after a few weeks, the building construction was ahead of the grouting. The grouting work for the walls was then forced to be performed in the narrow space between the building and the rock wall. The space was designed to be 0,8 m, but could in some parts be somewhat wider, due to rock over break.

From the very beginning it was obvious that it was extremely difficult to get hydraulic contact between new grout holes, old leaky pre-grouting holes or leaking fissures in the pre-grouted zone. As experienced above the leakage openings in the rock walls were seldom located at the leaking spots in the wall.

When a leaky hole opening could be found behind the shotcrete, it could be repaired by applying a packer into the hole. PU grout was used in most cases. The grout pressures were often high, up to 8-14 MPa. Such high pressures were possible since the grout only acts in thin thread like fissures in the holes or fissures forming water paths to the holes. To detect hole openings behind the shotcrete meant also that shotcrete sometimes must be taken down.

A lot of drilling was performed as percussion drilling with 3 -5 m long holes. For hydraulic contact it was necessary to hit the hidden leakage behind the shotcrete. This work became often extremely difficult and dense drilling was necessary. Percussion drilling was possible before the building was erected in cavern 2, from the trenches under the building and the relatively wide space at the caverns ends (gables) where the space was 3-5 m.

Small Core Drilling with Light Machines

As an alternative to percussion drilling, light core-drilling equipment with 20-25 mm diameter was tested. Only short, 1-2 m holes could be drilled. This equipment could be used from scaffoldings between the buildings and rock walls. Even here, the goal was to drill to the leaking grout holes. This method suffered from the low drilling capacity and the limited hole lengths. It became obvious that this technique did not work at all at large walls.

Hydraulic Fracturing Between the Leaking and the New Hole

To overcome the problem to find the leaking holes behind the shotcrete, hydraulic fracturing with water was tested. Figure 7. It was assumed after dense drilling that the new boreholes in many cases must close to the leaking hole or a leaking fissure in the grouted zone. But hydraulic contact cold not be established. Therefore it was tested to set the new holes under pressure with water. The small high-pressure PU grout pumps were used. These pumps could increase the pressure more than 15 MPa if needed. The new holes were set under increasing water pressure until hydraulic fracturing occurred. Figure 7. The pressure for fracturing in sound rock was often around 8- 12 MPa. A second pump with PU resin was then connected to the hole.

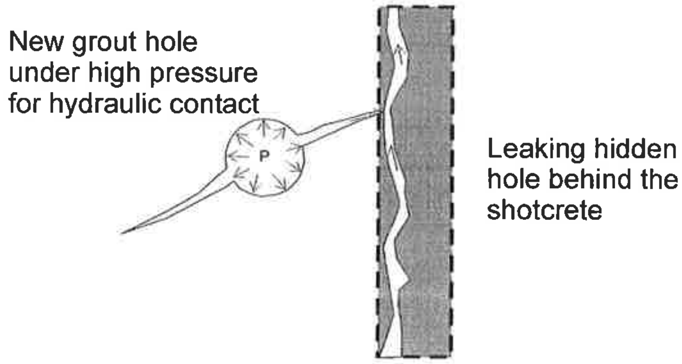


Figure 7. New grout hole pressurized to hydraulic fracturing for contact to the leaking grout hole.

It is often believed that high pressure grouting may cause rock falls or stability problems. In this project rock fall never happened due to high grout pressure. When the water acts in the borehole and when the hydraulic cracking occurs the crack propagates only a short distance and then the pumping pressure decreases momentarily. The small grout pumps cannot preserve the high pressure after a fracturing.

It is well known that the fissures in the rock seldom are planar. The leakage paths are more like small undulating pipes. It is the author's experience that the pressure then acts on very small surfaces and cannot cause the high forces needed for rock fall. In case of large fissures, it is mandatory that in all grouting the pumping begins with as low pressure as possible. After the voids are filled the pressure can be raised to penetrate fine fissures. Hydraulic fracturing with small pumps can only work during conditions without leakage paths to the hole the high pressure cannot be raised. Grouting with high pressure PU could work in many cases when small leakages were grouted. A serious problem was to receive safe anchorage of the packers for percussion drilled holes, during high pressure. In that way high pressure may be a severe safety risk, especially when the grout is chemicals.

Long Hole Core Drilling

After the first grouting round in cavern 2 a second round was performed in cavern 1. Leakages here occurred up 13 m above the floor level. Drilling with long drill rods could not be used. Drilling with small core drilling machines was also rejected due to bad drilling capacity experienced before.

It was decided to test heavier core drilling. An experienced core-drilling contractor with a compact high-capacity drilling machine with capacity to drill 36 mm holes was engaged. If inclined, this machine could operate in the narrow space between the building and the cavern.

Upwards directed 13 – 16 m long core holes were drilled in the cavern wall to the gallery level. The drilling continued until water broke into the hole, which often occurred when the drill hole reached the gallery level and outside the grouted zone. Figure 8.

For grouting, microcements were used, mostly Microdur R-F with d95 = 24 - 16 µm particle size but also finer grain size 6 µm was tested. A superplasticizer, Tricosal, was used as an additive. The concept with long core holes grouted with microcement worked surprisingly well. It was obvious that the core holes had much better sealing capacity than the percussion drilled holes. In case of surface leakage of grout, a fast shift to PU resin mostly halted the surface leakage and the cement grouting could continue.

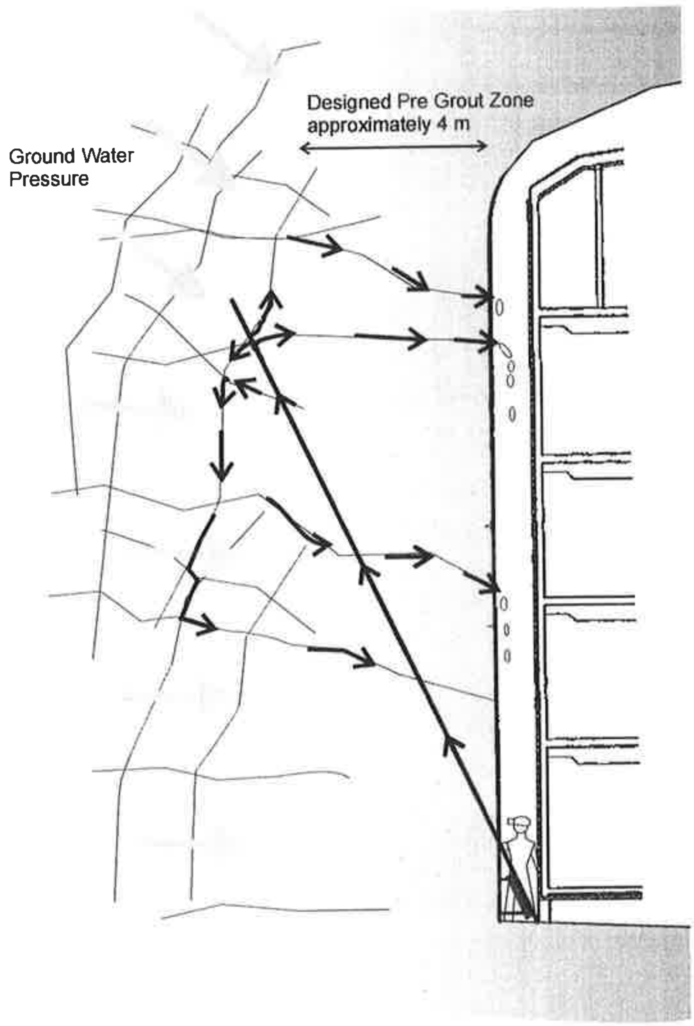


Figure 8. Long hole core drilling outside the pre-grouted zone

A hypothetical explanation to part of the success may be that the grout probably reached outside the pre grouted zone and was forced back to the large cavern by the ground water flow directed to the cavern. During this return it sealed the waterways. Large surfaces were sealed with this technique. A remaining impression was that the core holes compared with percussion drilling had a far better acceptance to grout.

The long core drilling was also used for remedial grouting in a part with bad rock conditions at one end part to cavern 2. This part had been remedial grouted already during the excavation stage and further during the later repairing processes, but with bad results. The upward inclined holes with long core drilling became even here successful.

Differences Noted Between Microcements

The used Microdur cements in combination with the Tricosal superplasticizer have a long set time. This was considered as an advantage. It was believed that the long set time also prevented the cement grains to clog into aggregates, an effect which during field conditions is difficult to control. Long time treatment in the agitator was also believed promote aggregate growth. Hence, operators were instructed to mix small batches only, max a half cement bag at a time.

The grouting in the 150 m long narrow spaces between the building and rock wall required long grout hoses. Trials with Swedish Portland microcements caused clogging in a long grout hose, which was lost. The slow setting microcement concept was adapted to avoid further tube failures.

Trials with the extremely fine grain size 6 μm grout displayed no significant better sealing effect than coarser grain size microcements.

During remedial grouting the major part of the grout usually was lost due to surface leakages, gel processes in the grout, and cleaning of hoses, mixers and agitators. A very minor part remained in the rock fissures. Despite the often very long setting time of the microcement grouts, no unusual squeezing out of grout occurred due to ground water pressure.

Conclusions

The relatively expensive long core holes had far less blocking effect for cement grout than the percussion drilled holes and were considered worth the additional costs. The microcements with superplasticizers open possibilities to seal large rock walls containing fine fissures.

Access to fast setting grouts, preferably chemical resins is necessary, for repairing surface leakage from the remedial grout hole in a large rock wall.

Chemical resins may be a good help for fast sealing of small leakages from cement grout hole openings, which in the project occurred from the remedial grout as well from the pre grout holes.

The total water inflow to the cavern system was reduced with around 75% and walls became dry to an acceptable level.

DEVELOPMENT OF GROUTING EQUIPMENT IN SCANDINAVIAN TUNNELLING DURING THE LAST DECADE

Utvecklingen av injekteringsutrustningar inom Skandinaviskt tunnelbyggande det senaste decenniet

Sten-Åke Pettersson, Atlas Copco Craelius AB, Stockholm

Atlas Copco Craelius is probably the oldest commercial manufacturer of cement grouting equipment in the world.
Its production of grouting equipment started in 1921, which means that the company celebrates 80 years in the grouting business this year (Fig 1).
At a brief glance, it might look as if the equipment has not changed much since then. The equipment (Fig 2) from 1925 comprises the same fundamental pieces as the equipment of 1980, 1990 or 2000: a mixer, an agitator, a pump and recording equipment. Even as far back as 1925, equipment was hydraulically driven.
A rough comparison can be made with a car from 1925 and a car of today. The features are the same, but the performance characteristics have increased considerably and have also become more user-friendly since then.

This paper will not go into details on how the basic pieces of grouting equipment are designed and the principles of grouting, as that is done elsewhere. But it can be said: "When it comes to mixing and grouting, the quality of the mixer is more important than the quality of the cement".
The paper describes the development and experience, which are gained from complete or almost complete grouting equipment from Atlas Copco Craelius.

Up to the beginning of the 1990s, the equipment was mostly sold piece by piece and customers mounted the different pieces together themselves. The new complete equipment was based on old machinery and equipment available in the store, supplemented by new pieces needed for the specific job in mind.

The grouting equipment for tunnelling in the Stockholm Subway and the extensive works at the Oslo sewage tunnels or the Fjellinien tunnel during the 1970s and 1980s were manufactured this way.

During these years, the demands were not so big on the final result and much equipment was made with the intention of being as cheap as possible.
Grouting equipment for less than SEK 100,000 could be discussed over and over again and was considered expensive at the same time as the same management bought a drill rig for 100 times that amount without any hesitation at all.

This method of producing grouting equipment started to change around the 1990s, due to different circumstances.

Firstly, it was the increased demands from authorities and consultants in relation to the finished job. Many of the previously-built tunnels had to be regrouted in order to make them better sealed. Leakage had to be reduced from, for example, 40 litres/min for 100 metres of tunnel to below 5 l/min, 100m.
Secondly, the limited time available from when the tender was received to when the work should start and/or from when the work started until it should be completed, meant that the equipment needed to be ready to go into operation within a short period of time. At the same time, the capacity of the complete equipment had to be increased.
The new labour laws and international safety regulations also meant that it began to be complicated to produce machinery locally. Atlas Copco had the necessary know-how to do all this and had done so for many years for dam foundation works worldwide.

In the past decade, the design and material as well as capacities have changed considerably due to the abovementioned circumstances. (Fig.3)

Some examples of projects and equipment during the last decade

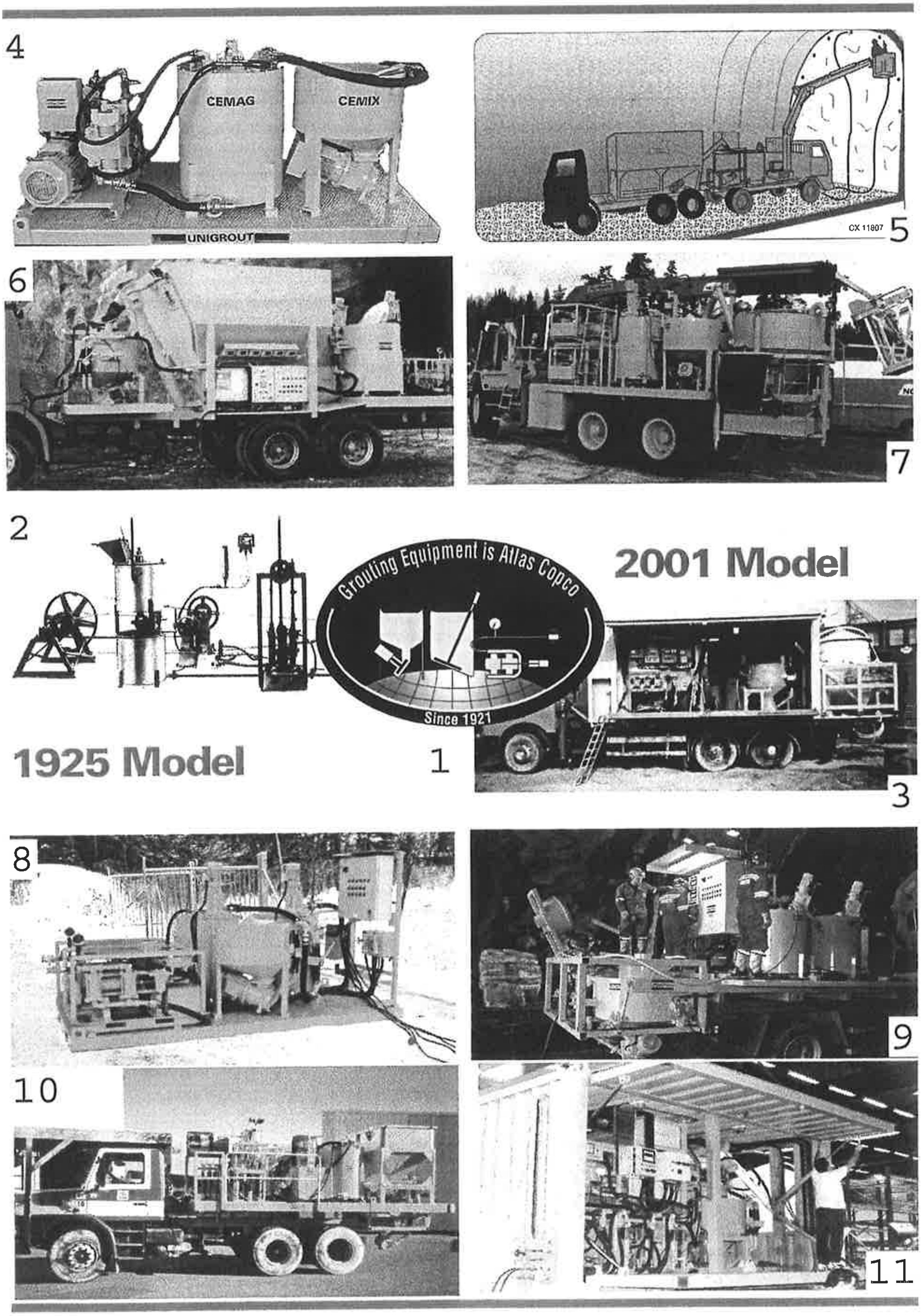
By 1990, it was mainly standard grouting equipment. Cheap paddle mixers could still be seen at work. Colloidal type mixers, with no better performance of mixing grout than down to a w/c ratio of 0.5, were abundant. The agitators were just a common barrel with a slow rotating paddle in the middle with no intention of trying to resist settlement. The pumps were mainly old and the flow and pressure on the pump could not always be set independently. The service and maintenance was difficult to do and the wear on valves could be significant.
Automatic weight batching equipment had started to appear. Recorders were rather simple printers connected to magnetic induced flow meters and the standard pressure meters had a very simple and inaccurate gauge saver.

Around 1990, Atlas Copco Craelius had started to assemble Unigrouts from its standard equipment. The complete units are denominated Unigrout. The name is only valid when the equipment is complete with mixer, agitator and pump and ready for immediate operation (Fig.4).

Hallandsås-1991

Atlas Copco is one of the companies, which have been involved in the Hallandsås project from the very beginning. We made the first quotation in 1991 and the picture (Fig.5) shows that proposal.

As can be seen, the equipment was split in two parts: A main part on one carrier with a mixer, an agitator, two pumps with two or more grout lines from each pump, as had been the practice since the 1970s, and a sky lift for the packer man. A second carrier for the cement silo, the cement screw and the admixture barrel. By splitting the equipment, the intention was to obtain better availability.


4
CEMAG
CEMIX
UNIGROUT
CX 11807
5
6
7
2
Grouting Equipment is Atlas Copco
Since 1921
2001 Model
1925 Model
1
3
8
9
10
11

There are two things to be noted: First, there is no recorder equipment on the main carrier and it was not requested. Second, the normal procedure up to then was to use a standard mixer and place it on load cells directly on the floor. This was cheap and easy, but the disadvantages were too great. The spilled grout and other material moved around on the floor, which caused problems.
Our experience was that it is much better to place the mixer container on load cells above the floor and fixed onto a special stand .The load cell signal cables are then connected to the weight batching processor (WBP). The WBP automatically operates the electric motors of the feed screws and pneumatic valves for water, admixtures and the distribution system and also controls the mixing time.
All these different functions are actuated by the grout mix formula stored in the processor's memory. The WBP can handle and store a number of different formulas. We always try to reserve one recipe for the cleaning of the equipment.

This start of the project was delayed and, later on, there were more rounds for further discussions concerning the grouting equipment.
The contractors at Hallandsås have, during the years, composed and built the equipment themselves to suit their own purposes. Atlas Copco has only delivered parts such as mixers, pumps, recorders etc.

During the different stages, we have carried out a large number of trials in co-operation with the different contractors. This was to improve the equipment and produce better mixers, pumps, valves, and recorders. From Atlas Copco´s point of view, it was said by one of our managers: "It is worth its weight in gold to be able to develop the equipment in a continuous, long-run operation and under similar conditions."

Without exaggeration, it can be said that the interest in grouting, grouting equipment and grout material has increased as a result of the tunnelling between Förslöv and Båstad and the Gardemoen tunnels in Norway. This interest is displayed by users, consultants, authorities and the general public.

Strängnäs 1994 and Unigrout E 400-100 WB

In 1994, Lemminkäinen Construction won the Strängnäs railroad tunnel project. This is a single-track tunnel, driven conventionally through 2,200 metres of hard rock beneath the town of Strängnäs. Parts of the tunnel have only 4-5 metres of ground coverage, but most of it is beneath groundwater level. The tunnel is 8.5 metres high from the crown to the invert and 7.5 metres wide, which gives a cross section of about 60 m^2.
The demand from Banverket, the Swedish railway authority, was that the water seepage into the tunnel was not to exceed 5-litres/min per 100 metres of tunnel.

Lemminkäinen had already started its computerised drilling operation and wanted professional grouting equipment to fully match their drilling methodology. One of the prerequisites was that the grouting equipment should only be two-man operated and discussions started on the basis of the equipment earlier proposed for Hallandsås. Lemminkäinen did not want a sky lift on the grouting equipment, saying: "It takes too much space where it is located and there is already a working platform on the drill rig, which has to stand by during grouting in case it is needed."

This was the first time we had experienced a contractor who utilised a drill rig only for grouting. Our previous experiences had always been that a drill rig was only used for drilling the grout holes and these should be drilled as fast as possible. Later on, the very important blast holes would be drilled and the tunnel advance should not be held up by the simple grouting operation.

Lemminkäinen also decided that, since it was only a rather short tunnel, they wanted to have the cement bin on the same carrier as the equipment.

Our previous experiences were that it was complicated to have the silo on the same carrier. In a lot of cases, the front axis had been damaged or broken by the heavy load from the cement.We now decided to move the 11 metric-ton silo further back from the standard solution with the silo close to the cabin. (Fig.6) The mixer was then placed in the created space. The mixer impeller makes a pressure of approximately 1.5 bar, so the produced grout could be pumped to either of the two agitators at the other side of the silo. Lemminkäinen also considered it to be important that the mixer was equipped with a covered opening and a dust collector – both of which now were introduced on the Unigrouts.

Each pump should have three grout lines and its own separate agitator. Due to the limited take from each hole and the nearness of the ground surface, it was not possible to utilise all lines simultaneously. But with two agitators, it was possible to have two different grout recipes available at the same time. This had not been done on moveable equipment before.

Lemminkäinen also decided to have the recording equipment on the same operation panel as the weight batching processor and the electrical cabinets. All panels were fully visible for one man to operate from the same position. The Grout Printer allowed the user to read the data file and print the data recorded on a 75 mm paper strip during actual grouting. The operation was then: one packer man up in the basket moving the packers from hole to hole and one grouter supervising the whole operation by setting the pump pressure and mixing the different recipes according to stipulated specifications, and also recording flow and pressure for every single grout line.

We had some problems with the equipment at the beginning and the main one was the cement. It was difficult to feed the mixer properly. The cement clogged and formed lumps at the bottom of the silo, which were almost impossible to move. This was despite both a horizontal screw at the very bottom feeding the askew feed screw and an agitator rotating just above the horizontal feedscrew.

The reason was that, in the beginning, the cement was delivered in 25-kilo bags and the grout truck was driven up to the surface to load the cement. But there was a different moisture content in the silo caused by the movement from the tunnel and going up on the surface and back again. The cement was also sometimes rather warm when it arrived and stored on the surface during the rainy autumn. Later on, the cement bags were stored down in the tunnel and the silo was fed with cement in the tunnel. Both Lemminkäinen and we learned a lot about cement storing and handling.

Two years later, the same equipment did all the grouting at the Norrala railway tunnel, the longest in Sweden at 3, 850 metres. Some 578 tonnes of cement were grouted in grout holes totalling some 60,500 metres. The equipment is now operating in Finland.

Lemminkäinen had a very professional attitude to the complete drilling and grouting operation. It was the first company we had come across that put aside a drill rig exclusively to deal with grouting operations. They also knew exactly how they wanted the equipment to look and we just added our experience of grouting equipment to their knowledge of tunnelling.

NCC Läggesta and Arlanda-1995

At the same time as the Lemminkäinen project, we also took part in the construction of grouting equipment aimed for Läggesta and Arlanda. NCC built this equipment in their own workshop on a dumper chassis (Fig.7). The dumper is easy to drive, sturdy and rugged in tunnels – and it can take a heavy load. But it is rather high to climb.
This equipment had two ZBE 100 pumps, two Cemag agitators and a complete recording system for flow and pressure. They used their own cement silo, which was equipped with a negative weighing. The weighing cells record how much is leaving the silo and the weighing is as accurate as the "normal" weight batching – but it is necessary to measure the water and admixture separately. The silo was fed with big bags of cement, which is an advantage both from an economic and manual point of view. The hard work with 25-kilo bags is eliminated, but there were also some problems with moisture in the cement, which took some time to handle.
The grout lines were brought forward at the front of the dumper, where a sky lift was used for the operation of the packers. For the electric power supply, a cable reel was mounted on the dumper. This is a normal mounting on a drill rig but had not been used for grouting equipment.

Romeriksporten-11 pcs of equipment-1994

At the same time as these grouting developments in Sweden, Scandinavian Rock Group (SRG) started the 13.4 km-long tunnel project Romeriksporten, in Norway.
We delivered three specially-designed Unigrouts, which did the initial grouting work. They were all different and mainly manually operated (Fig.8). The units were designed

to work continuously, 20 hours a day at a pressure of 50 bar. Experience later proved that up to 60 bar was preferable.

At a later stage, when the grouting operation was extended considerably, Atlas Copco was involved in 11 of the 12 grouting rigs on the job. Some of them were standard Unigrouts – but others were manufactured by Stabilator with parts, mainly pumps, from Atlas Copco. We did studies on seats and ball valves and learned how a seat should look in order to be sustainable. Tests were also made on other vital parts for our planned new pump.
Most importantly for a grouting pump is that the flow and pressure can be independently set. But almost as important is the maintenance and daily servicing. It must be possible to change the four sets of valves as quickly and easily as possible and they should last, despite heavy wear. The variety of material is large and the cement quality varies from OPC through injection cement and all types of micro cement. Pumpac, the latest pump, is built on a system of components. The single user can change the set of valves in a few minutes. For rough handling, the set of valves is of the ball valve type and, for a bigger and more even flow, the disc valve type. At present there are two sizes on the cement cylinders –110 and 150 mm. The first gives 100 bar pressure and a flow of 130-litres/ minute, the latter a pressure of 55 bar and a flow of 235 litres/minute. But these are not achieved at the same time. To change from one cylinder size to the other takes approximately 30 minutes and the change is facilitated by using the pump's hydraulic system and power for dismantling.
There are three motor alternatives: 7.5, 15 and 22 kW. The different power alternatives do not give higher or larger single capacities, but result in a larger pressure-volume area in such a way that a higher pressure can be utilised at the same time as a larger flow.
The daily service of the Pumpac is normally reduced to pumping clean water through the fluid end at stops during work and shift changes, when shifting personnel. At shift changes, the greasing point and sets of valves should be attended to. So daily servicing is limited to 5-10 minutes of maintenance every shift change, plus a couple of minutes of water flushing during breaks. At short breaks can the grout be re-circulated

FATIMA and Unigrout E 800-100 WB -1995

Fatima stands for "Fastlandsförbindelsen till Mageröya". This is the sub-sea tunnel to the island of Mageröya in Norway, where Nordkapp is situated. It was by then the world's longest undersea tunnel at 6.8 km and was constructed with its deepest point below 212 metres of rock and 140 metres of water to improve accessibility to Nordkapp. As with most subsea tunnels in Norway, the water inflow is not of the same magnitude of importance as tunnels in urban areas. Studies showed that the rock on the Mageröya side was very foliated and fissured and probably needed a lot of support. At the planning stage, this indicated that high mixing capacities were of the outmost importance and the equipment must be very mobile. This meant that two 400-litre mixers were necessary and that the unit had to be mounted on a truck.

Together with the contractor, Veidekke ASA, we designed the Unigrout800-100 E (Fig.9). To be able to operate in a tunnel with limited height, the two mixers were lowered as much as possible. They were then mounted on a special frame at the very back of the platform, which made the mixers very easy to clean and service. The intention was to have a double silo of 2 x 10 m^3 placed on a low trailer behind the Unigrout. The silo could be fed either by big sacs or moved in and out by a tractor for refilling.
The platform was equipped with flaps on the sides, which were hydraulically operated, and in this way the working platform could be extended considerably. There were two separate grout handling and mixing lines, which could be individually operated. Each mixer had its own separate weight-batching processors and the mixers were also equipped with dust collectors. There were two agitators, each connected to a pump. It was planned to add a third agitator and pump so that each of these three grout handling lines could be switched on and off to either of the two mixers.
The operation panel was placed under a small roof with neon lights.
There was no need for recorders on this Unigrout and we believed we were now getting a good grasp of how to design and build grouting equipment.
The low-placed mixer is such a good solution that we think there will be many more to come in the same style.

Underverket and Unigrout E 400-100 HWB-1996

The next project we were involved in was Underverket under Sundbyberg.
Underverket is a tunnel that will take care of contaminated water from rainfall and snow and keep it until it can be treated in a sewage plant.
The tunnel is 3,252 metres long and has a cross-section of 15 m^2. Two 23-m^2-access tunnels totalling 365 metres were also excavated in the compact gneiss-granite.
In order to avoid lowering of the groundwater, very strict limits for the water inflow were specified: The water seepage for the first 1.9 km of the tunnel should not exceed 4 litres/min. and the remaining part should not exceed 2 litres /min. Pregrouting should do the sealing and three different grouts were specified:
1-microcement with a w: c ratio =0,8
2-OPC based grout with a w: c ratio=0,6
3-OPC based grout with a w: c ratio=0,5
Each type was to be used in accordance with the rock.
The dosage of water is within 5% accuracy and the admixture dosage should be within 2%. Automatic recording of pump pressure, flow and volume for each pump line must be done. Maximum pressure must be preset. And, notably, the drilling should be done with a maximum deviation of 3%.

YIT Bygg AB got the job and we started discussions. YIT wanted to have a mobile Unigrout, which should be completely electro-hydraulically driven. This was the first completely electro-hydraulic equipment we had made and is another solution that we believe will stay for the future.

Since the tunnel was very narrow, we could not build the equipment as we did with the previous Unigrouts. YIT decided to use the lorry as an operating platform and mounted a working platform above the driver's cabin (Fig.10). From the platform, they could do all the upper face part grouting. On this unit, we used the same solution as with the Fatima Unigrout with the low mounted Cemix. We also added another solution for the silos, since three different types of grout were specified. Together with YIT, we decided to use removable silos. In order to save space, the silos were square and in two different sizes. The truck can carry two silos with different types of cement simultaneously. Each silo was quick-connected to a feed screw at the base of the silo.

Hitra-Fröya and Unigrout E 400-100 WBC-1998

Hitra and Fröya are two islands west of the Norwegian coast. They are essential parts of the fish farming industry in Norway. The tunnel between these two islands was the last leg of the communication system, replacing the ferries from the mainland. The tunnel is 5.3 km long and was built by Selmer ASA. Before the work started, Selmer approached Atlas Copco to acquire top-of-the-line grouting equipment for this job and for the future. They knew what they wanted and the design process started. This process is the same for every piece of equipment. A discussion starts until common and jointly-agreed ground is reached. Then a drawing is made, based on these discussions, followed by a detailed study of the drawings, and the necessary corrections are made. This procedure can go on with more revisions, such as a cable reel needs to be bigger and because of that it must be moved and because of that...etc, etc.
For this project, Selmer had the following basic requirements: the complete equipment to be mounted inside a container which must be transportable on public roads and have a maximum width not exceeding 3 metres; silos for two different types of cement to be used alternatively and at the same time; two pumps with two grout lines each, both with their own agitator and a mixing capacity of at least 4.5 m^3 of grout /hour. Everything should be completely automatic and recorded.

A specially designed and equipped container, 2.99 metres wide and 6.375 m long, was made (Fig.11). The mixer, the two agitators, the two pumps and all control equipment were placed inside the container. In order to have enough space for the operators – and also as protection from water etc. – we decided to divide the side of the container. The lower part became the floor and the upper part became the protective roof. The cable reel plus the two silos were placed on the back of the same base platform, but without a roof or walls. This means they are accessible from the outside and the silos are easy to refill with cement. One silo is intended for 1.2-ton drop bags and the other one for small standard bags of micro – or injection – cement. The cable reel, the operating platform and the protective roof are operated by the hydraulic system of the grout pumps.

With this equipment, we also used the new computerised recording equipment LOGAC. The computer has a lot of advantages over the previous standard recorder and printer and its development is leaping ahead. There is no problem in making a very modern and sophisticated recorder. In fact, it is very easy and we can get any help we want. The problem is to make an easily-operated recorder, capable of being operated by people who are not familiar with or interested in computers. Besides being easy to operate, it also has to be capable of working in a dark and very dusty environment with extremely limited light.

Drammen and Unigrout E 400-100 WBC-2000

The same equipment was later moved to the Drammen Northern Bypass, a 2,300 metre-long tunnel under the Bragernesåsen .The production figures are impressive and Selmer was also there working according to the North Sea System. This is a three-shift rotation system with two weeks on duty. Each shift comprises three men plus a technician (mechanic and electrician alternating) taking care of the whole operation. They drilled the 5.2 m-long blast holes and then charged and blasted the round. After every third round, they drilled the 27 m-long, 51 mm- diameter grout holes, which also served as investigation holes.

This system gives the crew full knowledge of what is ahead of them and no information about the rock is missed. They know when the rock is changing and where the fissures are, which is almost as important for blasting as grouting. This extensive knowledge is unique and a good example for others. The crew, together with the consultant, make the decision immediately on whether they should change the grout recipe or be careful with contour blasting, etc.

The nominal mixing capacity of the equipment is 4 tonnes of dry cement per hour. But at especially bad and wet areas, they grouted more than 66 tonnes of dry cement over a 14-hour period, giving an average mixing and pumping capacity of 4.7 tonnes of cement per hour. The pumping capacities were 100-120 litres/minute at 50-60 bar and 80 bar was the set stop criterion – but up to 90 bar was used occasionally.

Norway is leading the technical progress in using high or very high-pressure levels in standard grouting operations. An interesting observation was made during the passage of bad and "clayey" zones. There was no response until about 60 bar pressure was reached. After having grouted the zones at these high pressures, the ground became stable and sealed – and this is probably something that can be used at other locations, too.

Veidekke at T-baneringen Unigrout E 400-100 HWBC-2001

T-baneringen is at present an ongoing project in Oslo, where Veidekke ASA is responsible for the grouting work. Together with Veidekke, we have made a grouting container, which leads the present grouting equipment technology (Fig.3). It is the same base container as specified above for Selmer ASA above, but two hydraulic power packs have been added for the operation of all machines including a third pump line. And a lot of design and service details have also been improved.

A few comments.

Mixing with high-speed mixers.
Some years ago, there were requirements in many tender documents for high-speed mixers with specified speeds of: "at least 3000 rpm". I believe this was a misunderstanding from a laboratory study made for Hallandsås.The equipment used for the study was a small desk mixer which could mix around 1 kg of cement each time at very high speed.
We did a thorough study together with Vattenfall Utveckling AB and Lambertsson Sverige AB at different speeds. From these studies, we could not draw any conclusive conclusions regarding the general necessity of high speed mixing. The improved mixing from the desk mixer was probably caused by the small batches and not from the high speed. But there is a difference in mixing 1 kg or 300 kg of cement per batch.
Our experiences, from tests over the years, are that the best mixing speed is around 1,700-1,800 rpm for normal batches. This is best achieved by using hydraulically-driven equipment.

One pump per grout line
One pump per line is also something that has been a prerequisite in some tender documents. I do not see why. If one pump is specified for each grout line, then a minimum pump capacity and a limit of pumping time should be set. These specifications should then be in relation to the condition of the actual tunnel face and the quality of the grout mix. And this has not been the case-yet.

Capacities
More realistic figures regarding both mixing capacities as well as estimated time capacities for many grouting sequences are needed in order to avoid unpleasant surprises.

Final words
The increased technical and environmental demands on the finished tunnels have been the driving force behind the thrilling development of grouting equipment over the last decade.
Atlas Copco is proud to have had the opportunity to take a leading part in this development.

References
Pettersson, S-Å.,Molin,H. (1999) *Grouting& Drilling for Grouting*- Atlas Copco
Various On-the Job-Reports –Atlas Copco

PREPARATION OF CEMENT-GROUT

Tillredning av injekteringsbruk

Tommy Ellison, AB BESAB
Staffan Hjertström, Cementa AB

Abstract

Cement-grouts prepared in laboratory were compared to cement-grout from an ongoing tunnelling-project in Stockholm. Tests made with the "NES-apparatus" showed that under these conditions penetration-properties were equal for laboratory-prepared cement-grout and grout made under realistic production conditions.

Filter built up by clogging in the crack also showed equal behaviour while using optimal dosage of super plasticizer. However cement-grout mixed with laboratory equipment seemed to be more robust to variations in dosage of super plasticizer.

Sammanfattning

Injekteringsbruk tillverkad i produktionsutrustning jämfördes med bruk tillverkat i laboratorieutrustning. Mätningar med NES apparaten visade att vid detta tillfälle var inträngningsegenskaperna likvärdiga för laboratorieblandat kontra produktionstillverkat bruk.

Bildningen av filterkaka studerades också. Laboratorieblandat och produktionsblandat bruk var likvärdiga vid optimal dosering av flytmedel. Dock tycks bruk blandat med laboratorieutrustningen vara mer robust för variationer i dosering av flytmedel.

The Southern Link-project

The Southern Link is a 6 km traffic-tunnel in the southern suburbs of Stockholm. The Southern Link connects the Värmdö-link in the east to the E4-motorway in the west. The completed Southern Link solves a traffic-infarct in this highly populated area, as today's intensive car-traffic will go underground.

Main-tunnel consists of two parallel tunnels, each 4,5 km long. Each head-tunnel consists of double lanes and surplus lanes for traffic turning on or off. Several underground grade-separated junctions, installation-rooms, shafts and emergency-exits complete the project, and total tunnel-length is approximately 17 km.

Tunnel-sealing in general

In the last decade sealing-requirements in tunnelling have strongly increased. In Scandinavia normally nearly total sealed and drop-free tunnels is required, especially in urban areas. This requires methods to design and predict sealing-methods and grout-properties in order to achieve systematic and totally sealed tunnels.

Sealing in the Southern Link

The maximum permitted in-leaking amount of ground water is stated by judgement in the Environmental Court. Due to the judgement designers describes drilling-patterns, hole-length, water-loss measurement and grouting-procedure. After first grouting-round test-holes are being drilled, and if results are not satisfying a second round is done. This can be repeated until the result is good enough.

In the Southern Link-project the grout-properties are described in detail. The cement-grouts are to be pre-tested and also testing during construction is needed. One of the most important requirements is ability to withstand clogging even called "filtration-stability", where 50 cm^3 of each grout are supposed to pass through a 125 μm filter. The test is supposed to show the ability of the grout to penetrate and seal minor cracks.

These are the test-methods used in the Southern Link.

- Filtration-stability or ability to withstand clogging. The method is described above.
- Marsh-time are supposed to show floating-property of the fresh grout
- Water-separation shows the stableness of the emulsion
- Yield-point
- Viscosity
- Shear-strength of the hardening grout 1, 2, 4 and 12 hours after mixing

	Unit	Pre-testing demands	Demands during construction
Filtration-stability	cm^3	≥ 50	≥ 50
Marsh-time	seconds	≤ 60	≤ 60
Water-separation	%	≤ 1	≤ 1
Floating-limit	Pa	3-8	3-8
Shear-strength	kPa	≥ 30	≥ 30

Table 1. These are the required properties of the cement-grout in the Southern Link.

Grouting-equipment

The equipments used by BESAB in order to perform cement grouting in the Southern Link-project are made by Swiss firm Häny. The mixer has an extremely turbulent process in two directions and the pumping capacity is 1.000 litres/min. The materials are proportioned by weight and receipts are pre-programmed. Agitator, 2-4 grouting pumps and registration computer complete the equipment.

Test performance

In order to compare the penetration-properties the "NES-apparatus" were used. The "NES-method" is meant to be comparable to real grouting, as grout is pressed through a parallel opening hereafter named crack, with certain width by using over-pressure. The flow of passing material is continuously measured by weight. The flow is a measurement of the penetration-properties. In these tests crack-width were 75 μm and pressure were 20 bars over atmospheric pressure. All test-samples were composed of Cementa Injektering 30 and plasticizer Cementa HPM.

Field laboratory roomed in a usual container was placed in the tunnel during time for construction. The testing equipment consists of NES-apparatus, mixer "Ultra turrax" good for 20.000 rpm, Marsh-cone, Brinell-tester, viscosimeter and other equipment that was needed.

Test-samples from production were taken from the agitator right after mixing, and were brought to the laboratory in a plastic bottle. The time from mixing to start of test varies from 10 minutes. The samples were shaken before testing but no surplus mixing was being done. Laboratory-mixed samples were prepared in the laboratory with the same materials and proportions as samples from production.

Achieved sealing-results

Achieved sealing-results in the Southern Link are quite satisfying due to stated levels on in-leaking groundwater. Less than 4 litres/100 m, minute are being measured in general except for a few sections. In spite of that there are still some post-grouting made, mainly caused by rock bolting through grouted zone. Some typical experience from the pre-grouting in the Southern Link-project, site SL 03:

- Approximately 1.5 rounds was needed as a mean-value
- Cement-consumption was around 300 kg cement per meter tunnel
- Very little post-grouting was needed
- Some 20 % of the post-grouting is due to 5 m long rock-bolts that punctuated the grouted zone

Results from penetration-testing with NES-apparatus

Results from tests in Southern Link SL 03 site Trädskolevägen. Penetration is measured by NES-apparatus.

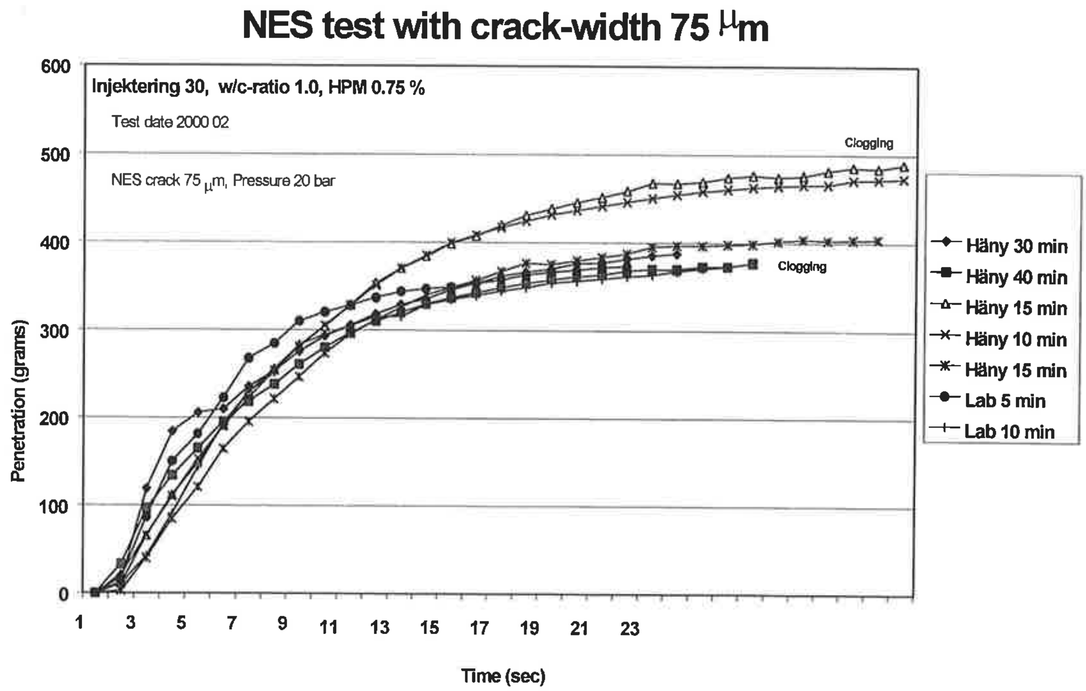


Diagram 1. NES-tests comparing mixer-type and age after mixing.

NES-tests described in diagram 1 shows that properties of grout based on Cementa Injektering 30 w/c-ratio 1,0 and 1 % Cementa HPM is not deflected by aging up to 40 minutes. Grout made in real production by Häny-mixer and grout made in laboratory mixer "Ultra turrax" is equal due to penetration-properties.

Results from filtration-tests (clogging) with NES-apparatus

Creation of filter-cake hereafter called clogging, can be studied with the NES-apparatus. After running test to a complete stop you can take the crack apart and the thickness of the clogging can be measured.

Diagram 2 and diagram 3 shows that the thickness of the clogging is equal for grout from production in Häny-mixer and grout made in laboratory if the dosage of plasticizer is optimal. However if dosage is not correct, grout made by production-mixer gives significant thicker clogging than grout made by laboratory mixer.

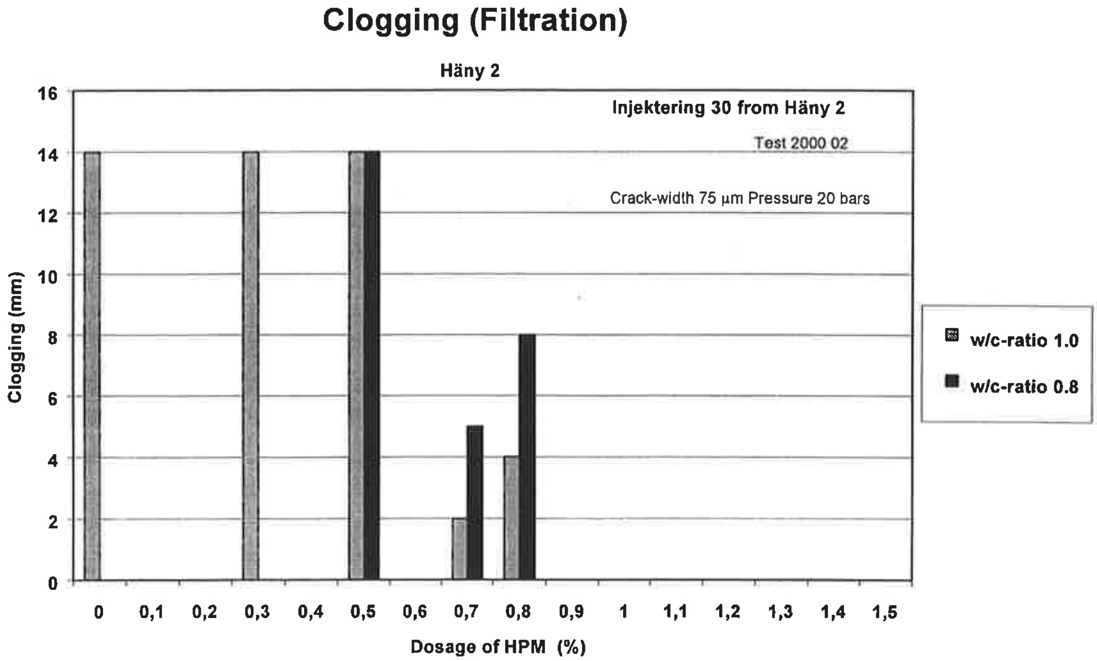


Diagram 2. Clogging of grout made by production mixer Häny measured by NES-apparatus.

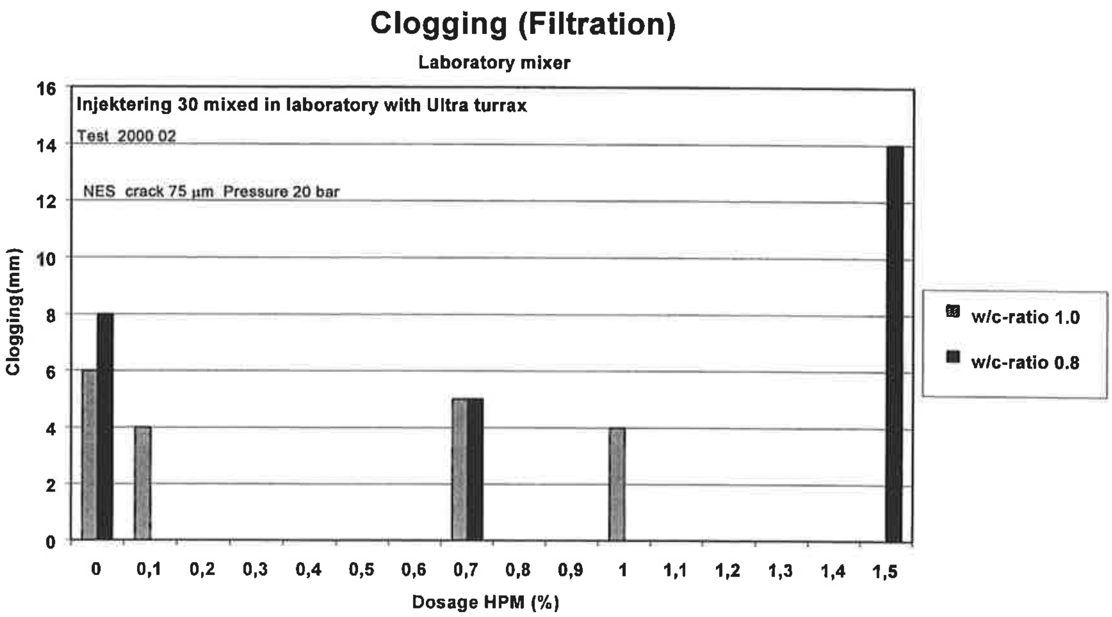


Diagram 3. Clogging of grout made by laboratory mixer "Ultra turrax" measured by NES-apparatus.

Results of various w/c-ratio with NES-apparatus

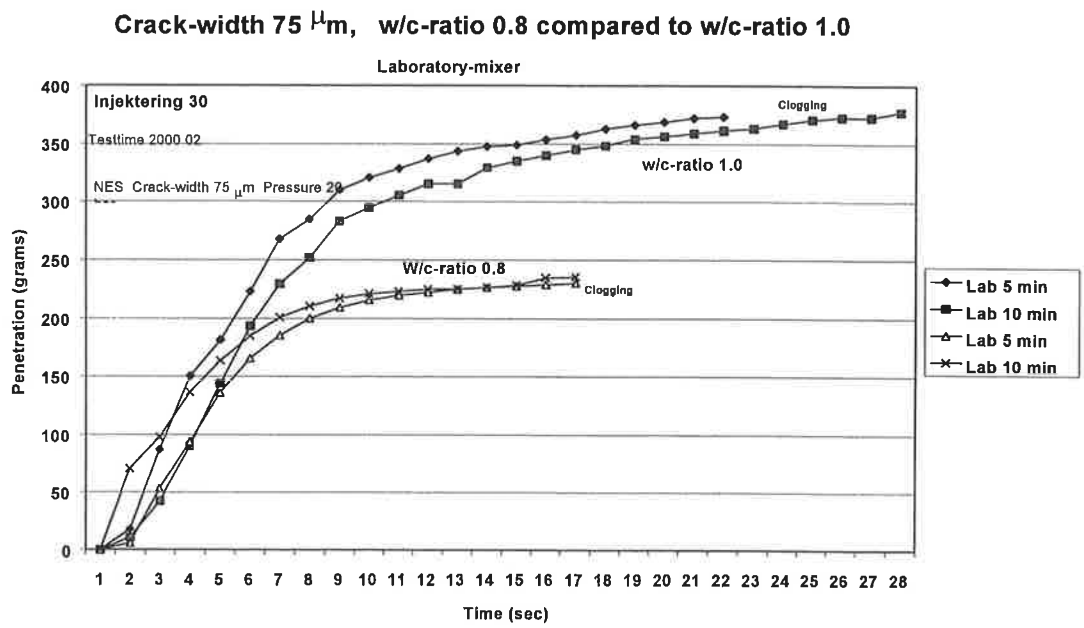


Diagram 4. Penetration-tests with two different w/c-ratios

Different w/c-ratios are supposed to affect penetration-ability quite significant. This is verified by the tests in Southern Link-project. See diagram 4.

TESTING OF THREE TYPES OF PACKERS IN CONNECTION WITH CEMENT-BASED GROUTING

Provning av tre typer av injekteringsmanschetter

Thomas Janson, Golder Associates AB
Lars Österlund, Bergab
Per Delin, Department of Soil and Rock Mechanics, Royal Institute of Technology, KTH
Bo Karlsson, Swedish National Road Administration, Stockholm Region

Abstract

During the sealing works – pre- and post-grouting – in connection with the rock tunnels for Södra Länken (one of the Stockholm Ring Road links), use has been made of what are referred to as "one-off packers". On completion of grouting, the packer tube is disconnected and the packer remains in the grouting hole. Two variants of one-off packers have been used. Back-leakage through the packer has been noted after completed grouting works. The cause of the back-leakage has been the subject of discussion between the contractor and the client. In order to shed light on the function of the one-off packers used, the Swedish National Road Administration has had the packer laboratory tested.

Both packer variants have been tested and, in order to provide reference data for tests on the one-off packers, tests have also been carried out on multiple-use packers based on the same testing procedure as that used for the one-off packers. The procedure includes systematic testing of the function of the packers and their effect on the grout.

A series of tests have been carried out with one-off and multiple-use packers. The pressures (10 and 20 bar) and grout mortar (Cementa Inj 30, water/cement ratio 1:1) used in the tests are the same as those used for the grouting works on Södra Länken rock tunnels.

At the time of writing (beginning of October 2001), not all the tests have been completed. Initial observations indicate that multiple-use packers maintain their sealing function and pressure in the test arrangements. The one-off packers show indications of back-leakage and give pressure losses in the test arrangement after completed pressure testing.

Sammanfattning

Vid tätningsarbetena, för- och efterinjektering, av Södra Länkens bergtunnlar har så kallade engångsmanschetter använts. Efter avslutad injektering monteras manschettröret bort och packern blir kvar i injekteringshålet. Två varianter av engångsmanschetter har använts. Bakåtläckage genom packers har noterats efter avslutat injekteringsarbete. Orsaken till bakåtläckaget har diskuterats mellan entreprenör och beställare. För att klargöra de använda engångsmanschetterna funktion har Vägverket Region Stockholm låtit testa engångsmanschetter på laboratorium.

Båda varianterna av engångsmanschetter har provats och som referens till testerna med engångsmanschetterna har även flergångsmanschetter provats enligt samma testprocedur som för engångsmanschetterna. I testet görs en systematisk provning av manschetternas funktion och påverkan på injekteringsmaterialet.

En serie tester har utförs med engångs- och flergångsmanschetter. Tryck (10 och 20 Bar) och injekteringsbruk (Cementa Inj 30, vct 1,0) vid testerna är desamma som har använts vid injekteringsarbetena vid Södra Länkens bergtunnlar.

Provningarna är inte är helt klara, vid dags datum (i början av oktober 2001). De första observationerna visar att flergångsmanschetten är tät och håller trycket i försöksuppställningarna. Engångsmanschetterna visar på bakåtläckage och ger tryckförluster i försöksuppställningen efter avslutad provtryckning.

1. Introduction

1.1 Background

During the sealing works – pre- and post-grouting – in connection with the rock tunnels for Södra Länken, different types of packers have been used. During the course of the work, discussions have been held on the advantages and disadvantages of the different types of packers. For this reason, the Swedish National Road Administration has arranged for laboratory tests to be carried out on the packers in different test arrangements.

Viewed from the outlet back to the inlet at the rear, a packer consists basically of a rubber packer that is forced up into the borehole creating a seal against the borehole wall, a pipe unit through which the grout mortar flows and which forces up the rubber packer, and a final section consisting of a connection and valve to which the grout tube is connected. Traditionally, use is made of so-called multiple-use packers for cement grouting. In the case of multiple-use packers, the grout flows straight through the entire packer. When the grouting is finished the valve is switched off at the inlet to the packer and the grout tube is disconnected. The entire packer remains in the borehole until the mortar has gelled. Once the mortar has cured, the packer can be disconnected and the next phase of the work started.

In order to introduce a more rational handling of the packers and be able to commence subsequent work at an earlier stage, so-called one-off packers have been introduced. A one-off packer consists of a rubber packing with a non-return valve made of rubber which is installed at the front of the packer. The idea behind the non-return valve is that when the grouting pressure is applied, the valve opens and when the grouting work is finished the valve is closed with the aid of the built-up pressure. Once the grouting operation has been completed, the pipe unit and connection are disconnected and the rubber packer remains in the borehole.

1.2 Purpose

The purpose of the testing has been to test the function and effect of the packer on the grouting process. Prior to testing, the following questions were posed:

- What is the functional reliability of the packer against back-leakage?
- What opening pressure is necessary for the one-off packer?
- What is the pressure change in the test arrangement after completed grouting pressure?
- What changes, if any, are there in the properties of the mortar when grouting through the packer? Do so-called filter cakes or material residues collect in the non-return valves of one-off packers?

2. Description of the tests

2.1 Preconditions

Manufacture of the test arrangement and testing of the various packers have been carried out at the Department of Soil and Rock Mechanics at the Royal Institute of Technology, Stockholm. The testing was carried out in three different test arrangements constructed from conventional pipe fittings and connections. Grout mortar mixing, grouting pressure and times, and the procedures used for pressure testing have been selected and executed in the same way as for the normal grouting operations in connection with Södra Länken.

The three types of packers that have been tested have been used for the Södra Länken rock tunnels, see figure 1. They consist of two so-called one-off packers (Alternatives I and II) and a multiple-use packer.

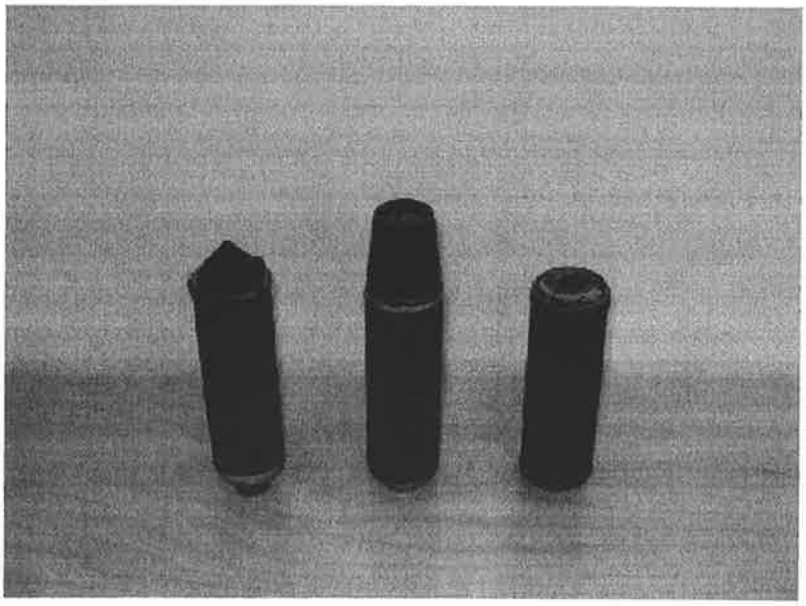


Figure 1 Photograph of the packers tested. From left to right – one-off packer Alternative I, one-off packer Alternative II and the multiple-use packer.

2.2 Test arrangements

Three different arrangements have been manufactured: a large arrangement with through-flow, see figure 2; a closed system without through-flow, and a smaller arrangement with through-flow over a longer period of time.

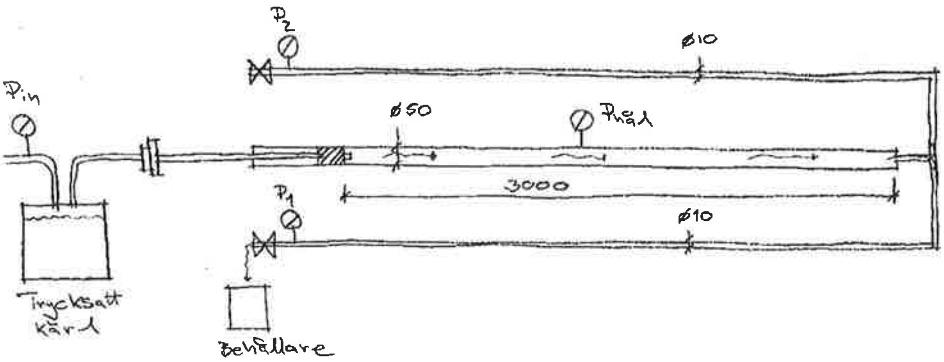


Figure 2 Sketch of the large test arrangement.

2.3 Testing methods

The testing methods have varied depending on the arrangement used. The testing started with the large test arrangement and the two latter arrangements were developed on the basis of observations from the first tests.

Large test arrangement

The basic idea behind the testing has been to imitate the grouting process as much as possible and to be able to answer the questions posed in Section 1.2. In the case of the large test arrangement, the following principle has been applied:

1. The system is filled with grout mortar under low pressure so that as much of the air as possible in the system is vented. The aeration screws are then closed and pressure testing commences with pressure increasing in stages until the final pressure is reached after approximately 25 minutes. After this, the outlet is closed and the system is pressurised for a further 5 minutes.
2. The pressure is switched off and the packer tube disconnected.
3. The system is kept closed for 2 hours, after which the packer is removed and the system emptied.

The following parameters have been measured or observed for the large test arrangement:

- Continuous pressure in the arrangement, at three different settings, both under pressure and after being pressurised for two hours.
- Outflow through the test arrangement.
- Application properties such as: density, March-cone time, filter pump with 75, 104 and 125 μm mesh and separation stability, at different stages of the testing.
- Back-leakage through the packer after pressurisation.
- Filter cake ahead of the packer in the test arrangement after 2-hour standstill.
- Separation of non-return valve on the one-off packer on completion of testing.

Closed test arrangement

The purpose of the test was to observe the pressure and any back-leakage after pressurisation in a completely sealed arrangement as well as to answer the question of whether there is any connection between pressure loss and back-leakage. In the case of the closed test arrangement, the following principle has been applied:

1. The packer is assembled and the system filled with water and pressurised. The system tightness is checked.
2. The water is drained from the bottom of the system with the packer remaining in place.
3. The system is filled with grout mortar and pressurised to its final pressure over a period of approximately 10 minutes. Then the pressure is switched off and the packer tube disconnected.

4. The system remains closed for 2 hours after which the packer is disconnected and the system emptied.

The following parameters have been measured or observed for the closed test arrangement:

- Continuous pressure in the arrangement, both under pressure and after being pressurised for two hours.
- Application properties such as: density, March-cone time, filter pump with 75, 104 and 125 μm mesh and separation stability, at different stages of the testing.
- Back-leakage through the packer after pressurisation.
- Filter cake ahead of the packer in the test arrangement after 2-hour standstill.
- Separation of non-return valve on the one-off packer on completion of testing.

Smaller test arrangement with a longer through-flow

In the two previous test series, narrow "wormhole" passages or flow paths had been observed in the non-return valve for one-off packer Alternative II. The purpose of the test was to see whether these "wormholes" affected the grout flow through the arrangement in the course of time. In the case of the smaller test arrangement with a longer through-flow, the following principle was used:

1. The system is filled with grout mortar and pressurised to final pressure.
2. The test continues until the flow through the arrangement stops or until approximately 2 hours have elapsed.

The following parameters have been measured or observed for the closed test arrangement:

- The continuous pressure in the arrangement during pressurisation.
- Outflow through the test arrangement.
- Application properties such as: density, March-cone time, filter pump with 75, 104 and 125 μm mesh and separation stability, for fresh mix and the last drops before finishing
- Separation of non-return valve on the one-off packer on completion of testing.

3. Observations during testing

Since the testing has not been completely finished at the time of writing (beginning of October 2001), it is only possible to present the results of initial observations in this article.

3.1 Observations from large test arrangement

A total of 12 tests have been performed in the large test arrangement as per table 1.

Table 1 Compilation of test results in the large test arrangement.

Packer alternative	I	I	II	II	Multiple	Multiple	I
Water cement ratio/ %HPM	1.0 / 0.65	1.0 / 0.65	1.0 / 0.65	1.0 / 0.65	1.0 / 0.65	1.0 / 0.65	1.2 / 0.81
Final pressure, Bar	10	20	10	20	10	20	10
No of tests	2	2	3	2	1	1	1

Figure 3 shows the results of the continuous pressure gauging. The following observations were also made concerning the large test arrangement:

- The packer alternative influences the pressure drop through the packer (back-leakage). There is also background leakage in the system (test arrangement), see figure 3. Alternative I loses the greatest pressure, and immediately.

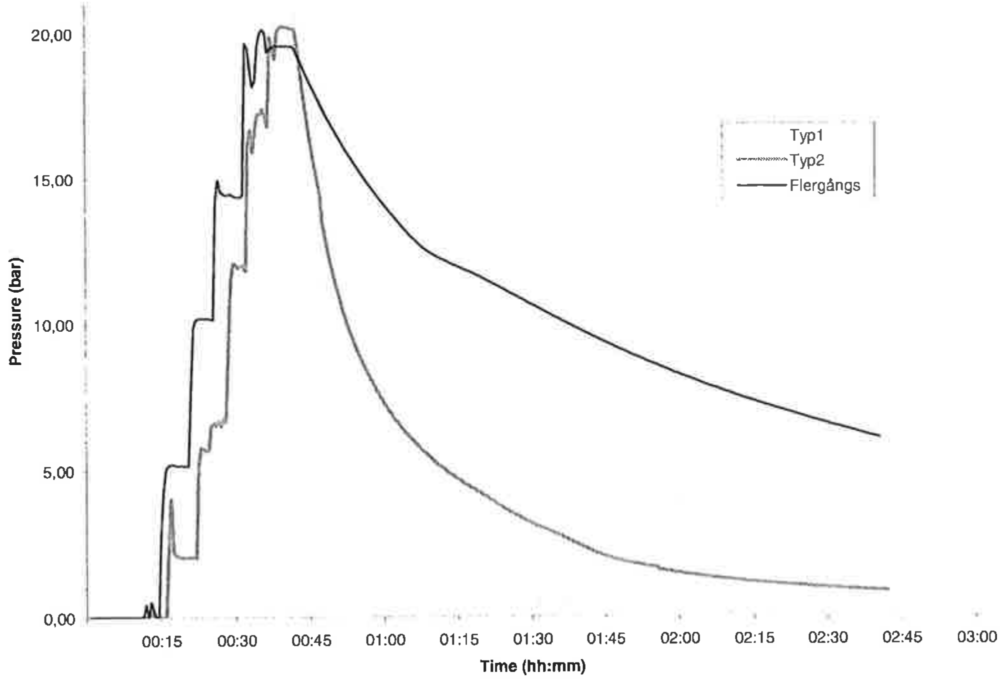


Figure 3 Example of pressure over time for Alternatives I, II and multiple-use packer for testing at 20 Bar

- The one-off packer in the case of Alternative II requires a starting pressure of 3 bar, whereas Alternative I and multiple-use packer allows mortar through directly.

- The grout mortar that has been forced through the system has the same properties regardless of the type of packer concerned (Alternative I, II or multiple).
- A large distribution of drop quantity (back-leakage). The tendency is for packer Alternative I to display a greater amount of leakage than Alternative II, and no back-leakage has been observed in the case of the multiple-use packer.
- In the case of the non-return valve on the one-off packer, a layer of mortar was found after testing. In the case of Alternative I, an approximately 0.5 mm layer of porous material was found, whereas for Alternative II, an approximately 1.5 mm layer of dense material was found with wormholes between the layers. The layer of grout mortar in Alternative II has a high density of approximately 2.26 ton/m^3 (corresponding to a water cement ratio of approximately 1:4.5).

3.2 Observations from the closed test arrangement

A total of 7 tests have been performed in the closed test arrangement at final pressures of 10 and 20 Bars and a water cement ratio of 1.1. Figure 4 shows the results of the continuous pressure gauging, for testing at 10 Bar, and figure 5 from the drop of the back-leakage.

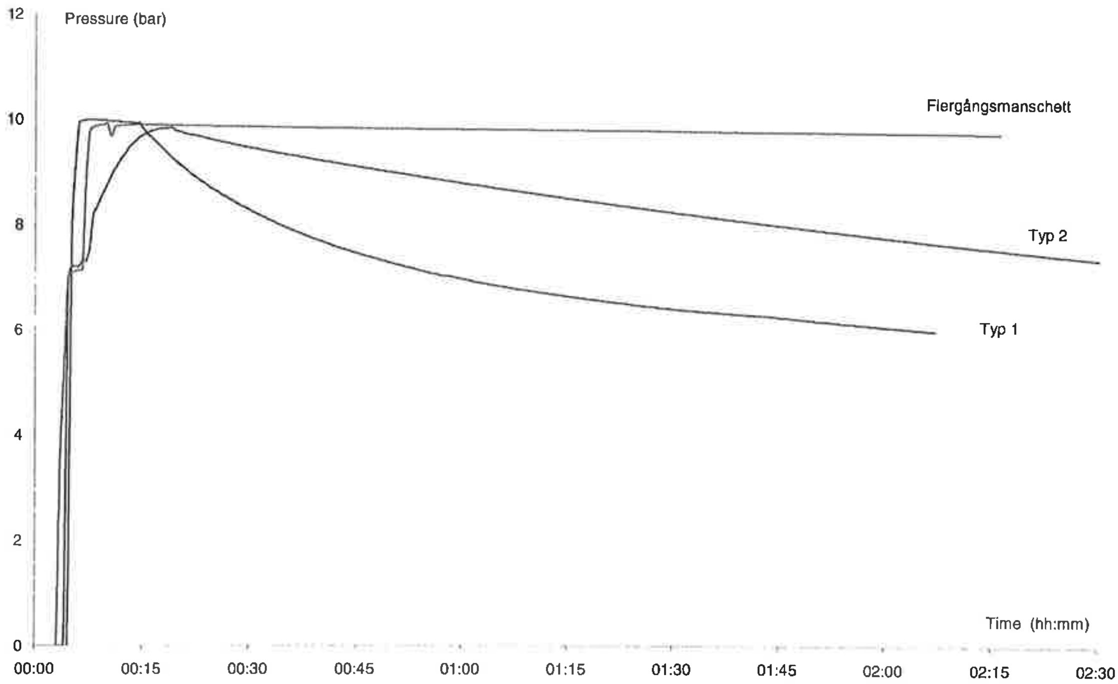


Figure 4 Graph of pressure drop for testing at 10 bar

The following observations were also made during the course of the testing:

- There is no pressure drop in the case of the multiple-use packer.
- For one-off packer Alternative I, at a pressure of 10 bar, an immediate pressure loss is obtained and after 2 hours' rest the pressure was found to have dropped by 40 %, see figure 4.
- In the case of one-off packer Alternative II, at a pressure of 10 bar, a declining pressure drop is obtained and after 2 hours' rest the pressure was found to have dropped by 20 %, see figure 5.
- The back-leakage at a pressure of 10 bar is approximately double the amount as for Alternative I (195 ml) compared with II (100 ml). No back-leakage is obtained in the case of the multiple-use packer.
- In general, enclosed air is compressed under pressure. At a pressure of 10 bar, the air quantity is reduced to approximately one tenth (10 %) of its original volume. This ratio concurs well in the case of the test at 10 bar if the pressure drop and drop quantity are compared after 2 hours.
- The pressure drop at a final pressure of 20 bar is not as large as it is at a final pressure of 10 bar – for Alternative I preliminarily approximately 20 % and for Alternative II preliminarily approximately 16 %.

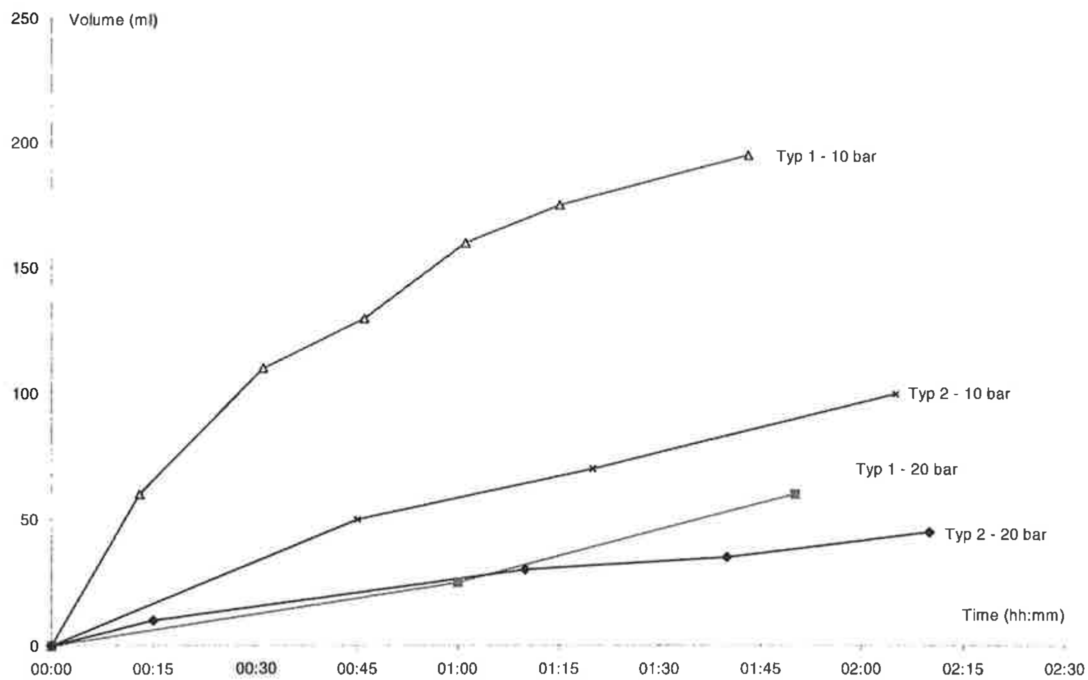


Figure 5 Preliminary graph of drop quantity from testing at pressures of 10 and 20 bar.

4. Discussion

At the time of writing (beginning of October 2001), an analysis and evaluation have still to be made of the results since not all the tests have yet been carried out. A presentation is given below of initial reflections regarding observations on the tests carried out so far at 10 and 20 bar final pressure and using Cementa Inj 30 with a water/cement ratio of 1.1 as grout mortar:

- There is a need to develop a one-off packer without pressure losses or back-leakage.
- There is no back-leakage with a multiple-use packer, irrespective of the test arrangement, and no pressure losses in the sealed arrangement.
- The connection between back-leakage and pressure loss has been observed in the sealed arrangement.
- One-off Alternative I accounts for a greater pressure drop and back-leakage than Alternative II. This difference is most obvious in connection with testing at 10 bar.
- The non-return valve on one-off packer Alternative II is more complex than for Alternative I. The non-return valve requires an opening pressure. Furthermore, the grout mortar that flows through the non-return valve forms a dense layer of material. This layer of material contains flow paths or so-called wormholes.