BeFo

STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

GROUTING THEORY AND GROUTING PRACTICE

Björn Stille

BeFo Report 157

STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

GROUTING THEORY AND GROUTING PRACTICE

Injekteringsteori och utförande

Björn Stille

BeFo Report 157
Stockholm 2024
ISSN 1104-1773
ISRN BEFO-R-157-SE

PREFACE

The Underground is often used during construction of infrastructure works such as railroads, metro, and roads. Tunnelling is however costly and stipulates high demand on safety and environmental consciousness. With regards to the environment the tunnels need to be sealed to reduce seepage of water into the tunnel and to avoid harmful lowering of the ground water table and potential damages to the surroundings. Sealing of the tunnels are usually performed with grouting, and to increase the possibility to achieve good results many research projects have been performed. What has not been studied to any great extent previously is the largescale variation of hydrogeological properties or the importance of the contractors works and process limitations.

In this project several field studies have been used with the objective to study the grouting process. In every case study hundreds or thousands of grout holes have been analysed. The case studies show that there is a correlation between seepage of water, grout take and most prominently the existence of zones. This means that there is a possibility to identify more conductive areas with for example grout take and consequently to increase the amount of grout works in these areas to reduce the seepage into the tunnel.

This project has also studied the grout theory and the application of ”The real-time grout control method” and shows that the theory is applicable to interpret the grout process and may also be useful to identify effects on the fractures/rock mass such as for example jacking.

In this work the hydraulic conductivity of the rock mass has been studied and concluded that the hydraulic conductivity is a lognormal distribution. It is also shown that the measurements are depending on scale where the geometric mean are depending on the measured interval length. The REV, representative elementary volume, is discussed and it is shown that for the presented data the rock volume needs to be about km sized for the variations in measurements to be negligible. This also means that for a larger scale, the hydraulic conductivity will go towards the arithmetic mean. This also means that many measurements that is performed in 3 m lengths is not representative and needs to be scaled up to at least grout hole scale to be able to describe both the hydraulic conductivity and the grout result as well as an alternative method to analyse seepage

The report is based on a licentiate thesis from Chalmers and the work has mainly been performed by Björn Stille (Chalmers), with advise and support from Gunnar Gustafson (Chalmers), Håkan Stille (KTH) and Shinji Kobayashi (KTH and Shimizu Corp.). Supervisor was Lars O Ericsson (Chalmers).

The project has been financed by Skanska, SBUF and BeFo.

Stockholm

Patrik Vidstrand

FÖRORD

Vid utbyggnad av infrastruktur nyttjas möjligheten att använda undergrunden för att dra väg och järnväg. Undergrunden och mer specifikt byggande i bergtunnlar ger stora fördelar och tillfälle att dra väg eller järnväg genom även tätbebyggda områden. Tunnelbyggande är dock kostsamt och ställer höga krav på säkerhet och miljöhänsyn. Med hänsyn till miljö och omgivning måste tunnlarna tätas för att inte inläckande vatten ska leda till grundvattensänkning och potentiella skador på omgivningen. Tätningsarbeten utförs vanligen med injektering och för att öka möjligheten att täta tunnlar har ett stort antal forskningsprojekt genomförts. Det som tidigare inte studerats lika utförligt är den storskaliga variationen av geohydrologiska parametrar eller vikten av entreprenörens utförande och dess begränsningar.

I detta projekt har data använts från ett antal fältstudier med syfte att studera injekteringsprocessen. I varje fallstudie har hundratals till tusentals injekteringshål studerats. Fallstudierna visar att det finns en korrelation mellan inläckande vatten, bruksåtgång och framförallt förekomst av zoner. Detta innebär att det går att identifiera genomsläppliga områden med till exempel bruksåtgång och att öka tätningsinsatsen i dessa områden för att minska inläckagen till tunneln.

Injekteringsteori med studie av tillämpningen ”The real-time grout control method” visar att den är användbar för att tolka injekteringsförloppet och även användbar för identifiering av påverkan till exempel i form av hävning eller jacking.

I arbetet har även genomsläppligheten i bergmassan studerats och det framgår tydligt att genomsläppligheten är lognormalfördelad. Vidare visas att mätningarna är skalberoende och där geometriskt medelvärde är beroende av mätintervallets längd. Vidare diskuteras begreppet representativ volym där bergvolymen måste vara minst ca km stor för att variationen i mätningen ska kunna anses negligerbar. Det innebär att i större skala går genomsläppligheten mot det aritmetiska medelvärdet. Det medför också att många mätningar som utförs i 3 m längder inte är representativa utan behöver skalas upp till minst injekteringsborrhål för att kunna beskriva både genomsläpplighet och troligt resultat efter injektering samt även en alternativ modell att beräkna inläckage med indata från tunneldrivning.

Rapporten baseras på ett licentiatarbete på Chalmers som huvudsakligen utförts av Björn Stille (Chalmers), med hjälp av Gunnar Gustafson (Chalmers), Håkan Stille (KTH) och Shinji Kobayashi (KTH och Shimizu Corp.). Handledare har varit Lars O Ericsson (Chalmers)

Projektet har finansierats av Skanska, SBUF och BeFo.

Stockholm

Patrik Vidstrand

SUMMARY

The data, which is the basis for this report, comes from several case studies that have been performed with the aim to study the grouting process. For each case hundreds to thousands of grout holes have been studied. There is a correlation between grout take and the largest water pressure test for each grout fan. Such a relationship could be used as a prognosis tool/classification method with regards to the extent of grouting and water seepage. The cases show that the geological and the geohydrological conditions clearly influence the grout take and the grouting process. The single strongest influence on the grout results seems to be the density of zones. The properties of these zones (shown by the extent, water pressure test and grout data) indicate that zones are very important for the effective hydraulic conductivity and the general flow regime. Where continuous geological structures indicate that parallel flow can be expected.

The geological conditions and the distribution of water pressure test results are indicative of the water seepage and grouting difficulty (grouting results). It is considerably more difficult to grout areas with zones and fractures that have clay/ partial clay filling. This should be considered when planning pre-investigations, describing geohydrological conditions and when procuring tender documents.

The grout take and the resulting water discharge indicate that a different approach to the grouting process could be suggested. Based on the large grout takes and the corresponding low grouting pressures that were common in the conductive rock mass (grout class C), use of two grout rounds is recommended. It was also clearly noted that the bottleneck in the grouting process was the grout mix capacities. The standard capacity for ordinary grout platforms was simply not sufficient to grout under higher pressure for a longer period to reach a stipulated penetration length for smaller fractures (calculated according to Gustafson and Stille, 2005).

For an ungrouted tunnel it is suggested that the effective hydraulic conductivity is represented by a mean value ranging from the 3D mean to the arithmetic mean with a scale (block size, relevant volume) that depends on the depth of the tunnel but not less than 20 m. In the case of a grouted tunnel, the grouting will significantly change the hydraulic conductivity close to the tunnel. The water flow for the grouted tunnel will be parallel and the effective hydraulic conductivity would therefore be represented by the arithmetic mean after grouting. It is recommended that the scale is equal to the grout hole length, i.e. around 20 m.

Grouting is to a large degree a special skill and experience is required to handle the rig decisions rationally. In the report 'Field tests with multi-hole grouting ' by Stille et al 2014 it was clearly shown that this was the case. Recurring issues, such as poor planning and poor communication, were often the case when inexperienced personnel handled the equipment compared to the relatively efficient grouting procedure with experienced personnel.

The case studies show that it is generally easier to achieve a better grouting result in good quality rock mass compared to poor rock. Furthermore, there are practical aspects with regards to the grouting process that need to be considered, such as equipment types and capacities, temperature, organizational competencies, and contractual arrangements, which may influence the grouting result.

The general conclusions with regards to the grouting process are

- The grouted fracture aperture will be about twice the hydraulic aperture.
- For the presented cases in generally good quality crystalline hard rock, about 50 - 70% of the rock mass can be considered ungrouted from a water seepage calculation perspective.
- Connected holes influence the grouting performance, resulting in areas that are not grouted or only partly grouted. After the third or fourth connected hole that is sequentially grouted, the holes are almost lost (small grout take, short grouting time and a resulting short fracture penetration).
- Mixing capacities may limit the grouting performance, especially in areas with higher seepage and with many connected holes.
- Elastic (and ultimate) jacking should be avoided.
- Two (or more) grout fans may be required to seal more seeping areas.
- For most grouting fans, one person at the face can only handle two hoses effectively. For more seeping areas, up to four hoses can be handled with grouting times of around 15-20 minutes per hole.
- An efficient grouting procedure requires experienced personnel and may limit the problems related to connected holes and grouting capacity problems.

SAMMANFATTNING

Underlaget till den här rapporten kommer från flera fallstudier med syfte att studera injekteringsprocessen. För varje fall har hundratals till tusentals injekteringshål studerats. Fallstudierna visar att det finns en korrelation mellan bruksåtgång och största vattenförlust i varje injekteringsskärm. Den typ av samband kan användas för att göra prognoser eller för att skapa en klassificeringsmetod för att bedöma omfattning på injektering och vatteninläckage. Fallstudierna visar att de geologiska och geohydrologiska förhållandena tydligt påverkar bruksåtgång och hela injekteringsprocessen. Den tydligaste singulära påverkan på injekteringsresultatet och inläckaget är tätheten, frekvensen, på zoner i bergmassan. Egenskaper på dessa zoner (identifierade med exempelvis utbredning, vattenförlust och injekteringsdata) indikerar att zonerna är mycket betydelsefulla för den effektiva hydrauliska konduktiviteten och även påverkar den generella flödesregimen. Större genomgående strukturer medför parallellflöden i större omfattning.

De geologiska förhållandena och fördelningen av vattenförlust indikerar vilket vatten inläckage och svårighetsgraden för injekteringen. Det är exempelvis betydligt svårare att injektera områden med zoner och sprickor med ler/ eller delvis lerfyllning. Det är därför viktigt att identifiera sådana förhållanden som påverkar både täthet och injekteringen med förundersökningarna, i beskrivningar av geohydrologiska förhållanden och när förfrågningsunderlag tas fram.

Bruksåtgång och vatteninläckage i fallstudierna visar att en annorlunda injekteringsstrategi kan rekommenderas. Baserat på att stora bruksåtgångar och medföljande låga injekteringstryck är vanliga i genomsläppligt berg, rekommenderas att två injekteringsomgångar utförs. Det kan också tydligt noteras att flaskhalsen vid injektering är mixerkapaciteterna. Standard kapaciteten för ordinarie plattformar är inte tillräcklig för att injektera med högre tryck under längre tid. Därför nås heller inte erforderlig inträngningslängd i mindre sprickor (beräknade enligt Gustafson och Stille, 2005).

För en oinjekterad tunnel föreslås att den effektiva hydrauliska konduktiviteten representeras av ett medelvärde mellan 3D medelvärdet till ett aritmetiskt medelvärde där 3D medelvärdet beräknas för en skala (blockstorlek, relevant volym berg) som beror av djupet på tunneln och inte är mindre än 20 m. För en injekterad tunnel kommer injekteringen att förändra den hydrauliska konduktiviteten av bergmassan nära tunneln. Vattenflödet kommer att vara parallellt och den effektiva hydrauliska konduktiviteten representeras av det aritmetiska medelvärdet efter injektering. Det rekommenderas att skalan och därmed resultatet efter injektering baseras på injekteringslängden d v s ca 20 m.

Injektering är till en stor grad en specialist aktivitetet där erfarenhet och skicklighet erfordras för att hantera beslut på injekteringsriggen rationellt. I rapporten 'Field tests with multi-hole grouting ' av Stille et al 2014 visades det tydligt att så var fallet. Återkommande händelser under utförandet som dålig planering och dålig kommunikation inträffade ofta när oerfaren personal hanterade utrustningen jämfört med den relativt effektiva injekteringsprocessen som observerades med erfaren personal.

Fallstudierna visar att det är generellt lättare att uppnå ett bättre injekteringsresultat i ett bra berg jämfört med dåligt berg. Vidare finns det praktiska aspekter med hänsyn till injekteringsprocessen som måste beaktas, som utrustningstyper och kapaciteter, temperature, organisationens kompetens och kontraktuella arrangemang, alla dessa kan påverka injekteringsresultatet.

De generella slutsatserna med hänsyn till injekteringsprocessen är:

- Den injekterade sprickans apertur är ungefär dubbelt så stor som den hydrauliska sprickvidden.
- Fallstudierna visar att i ett generellt bra kristallint berg kan ungefär 50 - 70% av bergmassan anses oinjekterad, vilket bör beaktas vid beräkning av vatteninläckage.
- Sambandshål påverkar injekteringsresultatet vilket resulterar i områden som inte är injekterade eller bara delvis injekterade. Efter det tredje eller fjärde hålet med samband som injekteras sekventiellt är hålet nästan alltid förlorat (liten bruksåtgång, kort injekteringstid och resulterande kort inträngning i sprickorna).
- Mixer kapaciteterna kan begränsa injekteringsresultatet speciellt I områden med större läckage och med manga sambandshål.
- Elastisk (och ‘ultimate’) “jacking” bör undvikas.
- Två (eller fler) injekteringsomgångar kan krävas för att tata mer genomsläppliga områden.
- För de flesta injekteringsskärmar, kan en person vid fronten bara hantera två slangar effektivt. I mer genomsläppliga områden kan upp till fyra slangar hanteras om injekteringstiderna är omkring 15-20 min per hål.
- En effektiv injekteringsprocess kräver erfaren personal vilket kan begränsa problem med sambandshål och kapacitetsproblem.

CONTENTS

1. Introduction 1

1.1 Background 1

2. Development of theoretical grouting methodology in Sweden 3

3. A review of the Namntall Tunnel project with regard to grouting performance 7

3.1 Introduction 7

3.2 Geological history of the rock mass in the county of Ångermanland (Local orogenesis) 7

3.3 Zones and description of the nature of joints in the area 10

3.4 Water pressure test results and grout classes 10

3.5 Summary 13

4. Experiences of the real-time grouting control method 15

4.1 Introduction 15

4.2 Grouting control using RTGC 15

4.3 Experience from different case histories 18

4.4 Summary 22

5. Distribution of rock mass hydraulic conductivity and its application to rock engineering problems 23

5.1 Introduction 23

5.2 Effective hydraulic conductivity and mean values 23

5.3 Evaluation of lognormal distribution parameters of section transmissivity in different cases 24

5.4 Influence of scale 26

5.5 Summary 31

6. Analysis of water seepage after grouting 33

6.1 Introduction 33

6.2 Grout penetration and grouting results 34

6.3 Transmissivity after grouting 37

6.4 Analysis of water seepage 39
6.5 Case comparison 40
6.6 Summary 41
7. Discussion and conclusion 43
7.1 Mean values and their use 43
7.2 The observational method as a design approach 44
7.3 The grouting process 45
8. References 49

1. Introduction

1.1 Background

Grouting is performed to reduce water seepage through a rock mass, under a dam or into a tunnel. In the big picture, however, grouting also limits the environmental impact caused by lowering the groundwater pressure or reducing damage to the object being constructed. As such, grouting is part of the construction and the process that starts much earlier and eventually ends with 'measuring' seepage or evaluating the grouting results.

The building process that involves the use of subsurface space starts with a societal need, regardless of whether it is related to infrastructure, energy or another area. Every such use will require an investigation of the influence on the groundwater situation or other issues related to seepage. These questions will eventually pass to the regulatory bodies for review and approval. One such body is the environmental court, which at an early stage will approve or regulate the permitted seepage or influence on the groundwater level depending on the situation and the phrasing of the application.

Societal needs and requirements will therefore regulate the construction process and the use of the subsurface space. The understanding therefore is that the grouting process starts, at least conceptually, at a very early stage with preliminary discussions about the needs and the many implications arising from the construction process, including water seepage, influence radius and so on. The level of detail is naturally lower at this stage and the estimates are more sweeping. Throughout the planning process the level of detail will increase up to delivery of the basic design (and possibly, but not usually in Sweden, during the detailed design stage). The basic design may be delivered at an earlier stage to ideally have the environmental court ruling available during the detailed design stage. However, the engineers or hydrogeologists may not be the same persons that carry out the grouting design or follow the grouting during the execution of the project. It is easy to understand the potential for miscommunication and conflicting interests, but the fact is that the data (e.g. from hydrogeological tests) and the interpretation of the data (e.g. by choosing different mean values) may be analysed differently, thus producing different results depending on the aim of the analysis.

The most detailed information on the ground conditions will be obtained during the construction phase. Unfortunately, the actual data, the hydrogeological distribution and the geological information related to these statistics, are often not collected or summarised for future reference. The organisation and/or contractual arrangements tend to limit the exchange of information between the environmental engineers and the design engineers and between the contractor and the client organisation.

This thesis provides information based on case studies dealing with the distribution of hydrogeological data, the mean values and the execution of the grouting. In addition, there is a discussion about the model used for analysing water seepage and the need for an open approach to controlling the grouting works by adopting an observational approach.

2. Development of theoretical grouting methodology in Sweden

Grouting theory spans almost three decades. The actual practice, however, has been carried out since the 1950s (or even earlier) with grouting of projects that included the Stockholm underground. The practice was often described, as by Morfeldt (1979), as a "dark art" and its methods were passed on verbally and through personal experience and experimentation. In this way, grouting methodology developed through 'wise men' and can be seen as a footprint of the general trends and analysis methods of the time.

In parallel, grouting theory evolved along four different paths:

1) Hydrogeology
2) Fracture theories
3) Knowledge of grout properties, including testing methods
4) Grout flow theories (Rheology)

Hydrogeology is in itself an old science, emanating from well theory and observations of aquifers, developed as a part of soil mechanics (pore pressure theory/effective stresses) and later for rock mass by Alberts and Gustafsson (1983) and others.

The description of fracture was, at least in Scandinavia, made relatively late by, for example, Hakami (1995) identifying fracture characteristics such as contact areas, matedness, fracture width distribution and so on, see also Olsson (1998). The distribution of fractures and zones in the rock mass is part of the science of geology, although up to the mid-1990s it was not exactly usable for grouting theory purposes. The definition of zones by, for example, Caine et al. (1995), and the quite similar idea by Munier et al. (2003) at SKB, made it possible to explain some of the grouting problems experienced in, for example, the Namntall Tunnel, as demonstrated by Gustafson and Stille (2010).

The first doctoral thesis focusing on grouting was performed by Hässler (1991), who studied grout flow in channels and pipes, including a numerical tool for calculating grout penetration. Some of the ideas presented in Hässler's thesis were not completed more generally until Gustafson and Stille (2005) and Gustafson and Claesson (2005) presented their theory of grout spread later published as Gustafson et al (2013).

Hässler's thesis was followed by Håkansson (1993), who studied the rheology of cement-based grouts and developed methods for grout tests. Håkansson tested a number of grouts and in doing so demonstrated the importance of additives, thus possibly opening up a new era in cement suppliers' research and development programmes. Today, the yield limit and viscosity of cement-based grouts are no longer unheard of properties and additives have been developed that can adjust these properties to the desired values.

Research is still of course being carried out in these four main areas although some significant attempts have been made to bring the areas together. The realms of hydrogeology and fracture characteristics have been studied through work by Fransson (2001), Hernqvist (2011) and Zetterlund (2014) on characterising the rock mass, interpreting water pressure tests and analysing fracture transmissivities and hydraulic fracture widths.

Gustafson et al. (2004) presented a paper analysing hydrogeological data (interval transmissivities) and fracture widths using a statistical model. The inflow calculation (based on the fracture distribution) and grouting effect by sealing some of the fractures brought together current ideas about geohydrology, fractures and grouting results. The ideas in the paper were further discussed in Gustafson (2009).

Gustafson and Stille (2005) demonstrated an analytical method to study grout flow based on grout rheology. In this paper, the diagnostic method used to identify dimensionality (planar or channel grout flow) combined grout rheology and ideas about fracture widths and implied that a design could be performed that included grout property and penetration length (grout hole distances) requirements. This paper was followed by several others, who studied the application of the theory to case records, including some suggested modifications to simplify use. The end product would be the Real-Time Grout Control Method (Kobayashi et al. 2007).

In his 'Hydrogeology for Rock Engineers' (2009), Gustafson summarises much of the research performed by the Swedish Nuclear Fuel and Waste Management Company. This book includes interpretation of various pre-investigation methods, groundwater flow in a rock mass, hydrogeological properties and groundwater modelling. Of particular interest for this thesis is the summary of statistical data for interpreting mean values.

The practical aspects of the grouting process were studied by Brantberger (2000) and Dalmalm (2004). Brantberger's study included a review of, for example, the GIN method, concluding that the method was developed for dam grouting and that the GIN design is based on a risk assessment/estimate of jacking in the rock mass. It may therefore not be relevant to grouting of tunnels in hard crystalline rock masses. Dalmalm (2004) made a detailed case study of the Southern Link project in his doctoral thesis as well as earlier work on, for example, the Arlanda Line, but had difficulty finding any significant relationship between the parameters in the Q-method and the grout take. Funehag's studies (e.g. Funehag and Gustafson 2004, Funehag 2007) include a silicate solution (silica sol) grout as well as practical aspects observed in the Hallandsås project and at the Äspö Hard Rock Laboratory. However, the practical application of the theory has often suffered from a lack of large sets of 'real world' data. In the paper 'Review of the Namntall Tunnel project with regard to grouting performance' by Stille and Gustafson (2010), they studied and summarised around 6,000 water pressure tests and items of grout hole data. Following this paper and after verifying some of the observations at the Namntall Tunnel, two master's thesis reports were published – Bohlin and Urtel (2008) and Bruno (2009) –

which studied the grouting process and performance in some of the Northern Link tunnels. These studies investigated both large-scale statistics and also revealed practical problems related to the grouting process in the field.

In the City Line project, the fracture width distribution theory (based on Gustafson and Fransson) and grout penetration theory (based on Gustafson and Claesson 2005 and Gustafson and Stille 2005) were used to calculate the required grouting time (Zetterlund and Eriksson, 2007). It was later shown by, for example, Holmberg et al. (2012) that although the required grouting time was based on not actual grout properties, the concept seemed correct and it was further developed in the project to take into account the actual grouting and the rock mass response (grout take, seepage etc.)

The determination of grouting pressure is still difficult considering possible negative effect of jacking of the fractures. In a study by Rafi 2014 the analytical solution in estimation of grout spread and distinguishing onset and mechanisms of elastic jacking were expressed. Both negative effects such as increase in grouting time and remaining transmissivity and positive effect on increase of penetrability were discussed.

In studies by Rahman 2015 and Shamu 2021 rheological aspects of grouting were studied with inline measurements and vizualisations using ultra-sound. Several interesting opportunities may arise from this works considering the present uncertainties of grout properties during a full grouting round. Identifying possibly aging of grout and consequences of mixing new and older grout in the agitator.

In Zhang 2021 a design methodology for dam curtain grouting was developed using the theories of rock grouting. The grout curtain is treated as a structural component of the dam foundation and the design is performed considering: (i) the reduction of hydraulic conductivity, (ii) prevention of erosion of fracture infillings and (iii) optimization of uplift reduction. Especially the erosion of fracture infilling has been studied using coupled numerical methods with promising results indicating that analytical solutions for laminar flow will give a good estimate of the risk for erosion.

The steps in the grouting process related to research and development are shown in Figure 2-1.

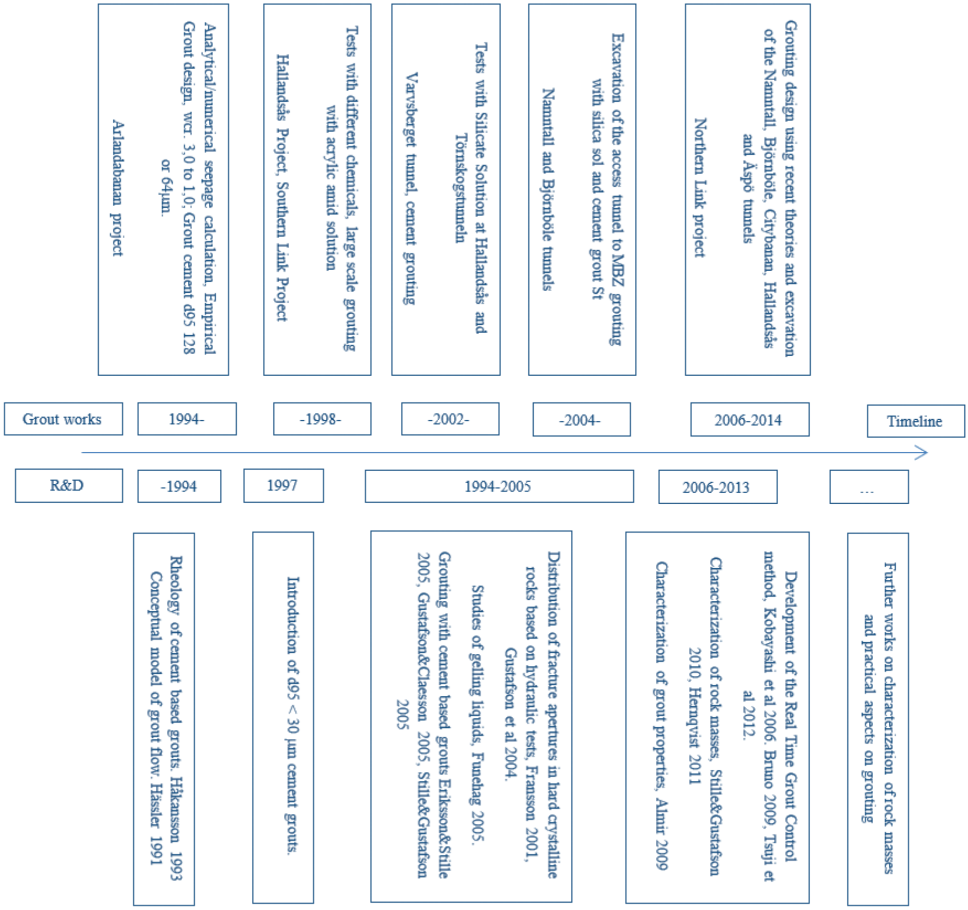

Figure 2-1. Research and development and a selection of the Swedish tunnelling projects discussed, modified after Sturk et al (2013).

Not only has the research and development process at Scandinavian universities contributed to the emergence of a new perspective on grouting design, suppliers has also followed the research and the requirements that have evolved over the years to create better and more finely grained cements of higher quality.

Stille (2015) summarised the research with regard to certain aspects of grout flow, grout properties, grout effect and inflow analysis.

3. A review of the Namntall Tunnel project with regard to grouting performance

3.1 Introduction

The six-kilometre Namntall Tunnel is part of the Bothnia Line project linking Örnsköldsvik and Kramfors. The tunnel was constructed as part of a design and build contract consisting of a single-track rail tunnel (65 m^2) and a parallel service tunnel (35 m^2). The client, Botniabanan AB (BBAB), was a partnership (90/10) between the Swedish Rail Administration and the municipal authorities in the area. The contract was awarded to Skanska Sverige AB as the main contractor for the civil engineering work. The total scope of the design and build contract included the six-kilometre Namntall Tunnel, several over ground parts and a five-kilometre second tunnel. The tunnels were excavated between 2004 and 2007 using the drill and blast method.

For the most part, the Namntall Tunnel, with a rock cover of 20-150 m, was excavated in greywacke and through a major intrusion of granite. The tunnels were excavated by drilling and blasting and using a normal grouting routine, including probe/grout hole drilling, water pressure tests (Moye, 1967), evaluation of grout class, cement grouting, drilling of control holes (water pressure tests) and supplementary drilling/grouting.

The chapter summarise information presented in Stille, 2010.

3.2 Geological history of the rock mass in the county of Ångermanland (Local orogenesis)

The geological history of the area can be said to have 'started' some 1,820-1,850 million years ago during the Swecokarelian mountain chain-folding process, Lundqvist et al. (1990). The greywacke bedrock was created during this process of intense tectonic events. Originally deposits of sedimentary material made up of sand and clay, the term greywacke is defined by its relatively large clay content (more than 15% in the matrix). In the Namntall area, several metamorphic stages in the greywacke have been observed, ranging from almost intact to a metamorphosed metagreywacke that is gneissic and even migmatised, resulting in, among other things, segregation of the mineral components into bands or strips.

The bedrock has been intensely folded, which is demonstrated by the steeply dipping foliation. The orientation/strike of the foliation is roughly NE-SW, indicating the probable direction of the local tectonic thrust NW-SE (perpendicular to the foliation) during this early phase of the orogenesis.

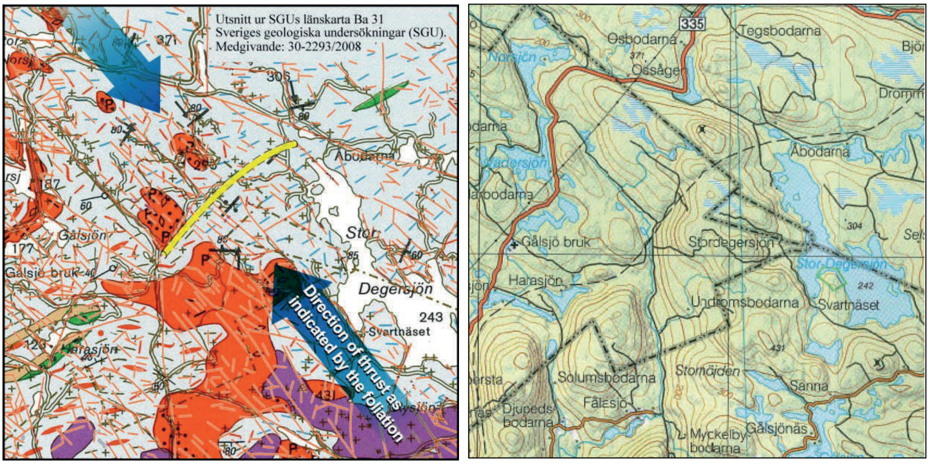

Figure 3-1. The topographical, geological and structural geological map of the Namntall area. The tunnel is marked in yellow, the foliation is indicated by the small blue lines and the lineaments are shown in brown. The bedrock consists of greywacke (grey) with intrusions of granite and pegmatites in red. The topographical map shows terrain features and the Bothnia Line railway, including the tunnel (from Stille 2016).

Since the mountain-folding process and the regional metamorphosis, the rock mass has been subjected to a number of faults, resulting in the local topography, Figure 3-1. The literature states that "valleys, the course of mires, steps in the terrain and straight lakeshores correlate to steep failure lines in the bedrock", Lundqvist et al. (1990). Furthermore, the erosion created by the moving direction (NNW – SSE) of the quaternary glacial ice has emphasised the weaker areas in the rock, especially where these coincide. It has also been observed that the deep failure lines also coincide with intrusive rocks such as granites, pegmatites and metabasites (which are considered to be the latest additions to the rock mass, originating some 1,200 million years ago). It has been observed that during the intrusions of metabasite dikes, the rock mass was subject to a considerable increase in fracturing, including the creation of horizontal crushed zones.

The large intrusion of granite between cross-sections 504+370 and 505+450, as well as numerous dikes, also support the theory that the area was subject to significant deep failure lines prior to the intrusion.

The general experience of the geological conditions during tunnelling led to a division of the geology into geological regimes or areas with similar characteristics. These are mainly related to water seepage, Figure 3-2, and the density of "zones", Figure 3-3, but as we shall see, they were also of consequence for the distribution of water pressure test results (grout class distribution) and grout take.

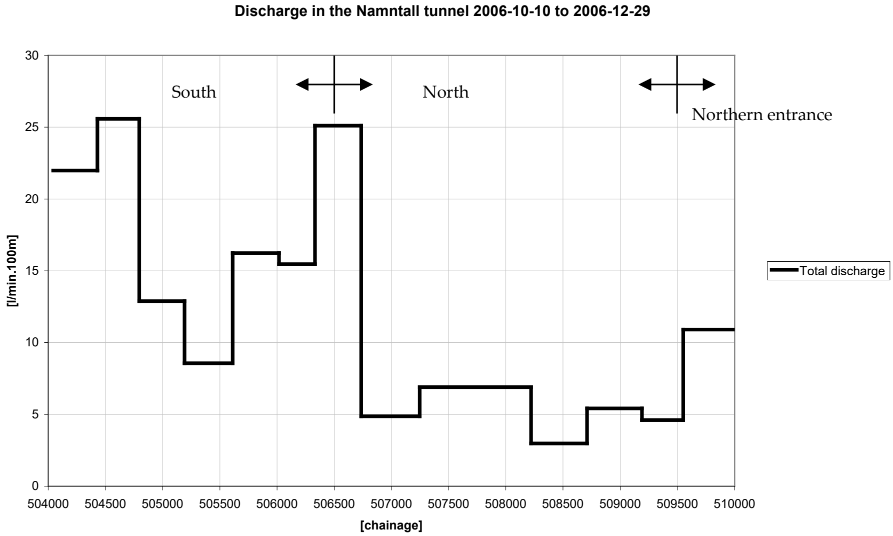

Figure 3-2. Total discharge in the Namntall Tunnel, including track and service tunnels. The measurement took place on October 10, 2006 and December 29, 2006 (from Stille 2016).

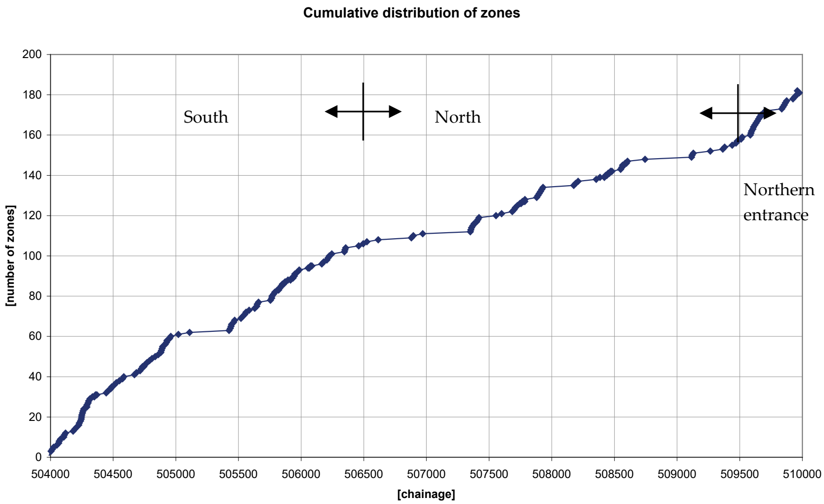

Figure 3-3. The cumulative distribution of zones over the tunnel length (from Stille 2016).

3.3 Zones and description of the nature of joints in the area

Herein Zones has been chosen as a designation to describe all areas that diverge from the surrounding host rock and which could be expected to have an impact on the water discharge. Zones include fracture zones (zones with a higher density of joints, e.g. 5-10 joints/m of a specific joint set), rock type contacts where the contact zone/area is more fractured, dikes that are either highly fractured or have a weathered or loosened contact area (a fractured, weathered dike is included whereas good pegmatites are not included) and shear zones.

The density/extent of zones is seen as an indication of where increased discharge could be expected. By adding the number of zones along the tunnel length (starting from the south) a cumulative distribution can be presented, Figure 2-3. The incline of the dotted line is indicative of the density of zones. A steep curve would therefore indicate a number of zones that lie close to each other. Figure 2-3 shows that the rock mass in the southern part of the tunnels has a high density of the zones (up to an approximate chainage of 506+500). The distribution of zones in the northern part of the tunnel (excluding the entrance area) shows a concentration of zones in certain areas but set some distance apart. The mean concentration of zones for the southern part is one zone every 23 m (2,500 m), for the northern part one zone every 62 m (3,000 m) and for the northern entrance part (500 m) one zone every 20 m.

Most of the zones have an estimated length of over 200 m. Most joints are 10-20 m in length although the length varies considerably. The foliation joints, for example, are in some areas very long, possibly over 100 m according to Lindström (2007).

The mapping performed by Stuge and Lindström (2007) shows that the joint systems in the area generally have two to three joint sets, often with one random joint set as defined in the Q-classification system (Barton, 2002). The spacing of the joints is generally 0.2-0.6 m, indicating fairly close spacing.

The filling in the joints varies along the tunnel although the joints are often coated with a thin, soft filling of clay, calcite or chlorite. In the areas with lower rock cover, the filling is generally thicker. In the area close to the southern tunnel entrance, some of the joints have a filling of swelling clays. For other parts of the tunnel, a partial clay coating has been noted, indicating difficult grouting conditions (Hässler, 2007).

3.4 Water pressure test results and grout classes

The hydrogeological conditions have been evaluated throughout the project by means of water pressure tests in the grout holes. The number of grout holes was generally between 10 and 20, arranged around the tunnel perimeter. The variations in number and length of the holes originated from the changes in grouting methodology that emerged from increased knowledge of the in situ hydrogeological conditions. After an initial period, 10 or 20 holes were generally drilled around the tunnel with a tested length of 21 or 23 m.

The water pressure tests were performed using digital water flow equipment with a measurement range of 2-38 l/min. The read-out range limited the water pressure test to between 0.2-4.0 Lugeon, l/min,m,MPa, (the values are for a test length of 20 m and 0.5 MPa overpressure). The water pressure test limits correspond to a hydraulic conductivity of K = $3.7 \cdot 10^{-8}$ and $7.5 \cdot 10^{-7}$ m/s, respectively).

The grout fans were classed as A, B or C fans after the highest water pressure test in each fan, Figure 3-4. The limits for the grout classes were based on the contractor's experience that a tight rock mass would produce a low water pressure test result (Grout class A: WPT < 0.5 Lugeon) whereas a conductive rock mass would have a significantly higher water pressure test result (grout class C: WPT > 2 Lugeon). Grout class B was defined as having a WPT of 0.5-2 Lugeon. For the most part the A fan included 10 grout holes, the B fan 20 grout holes and the C fan 20 grout holes plus additional control holes to verify the grout results.

The geological conditions along the length of the tunnel have been seen to vary considerably. A summary of the distribution of grout classes within each geological regime reveals some interesting statistics.

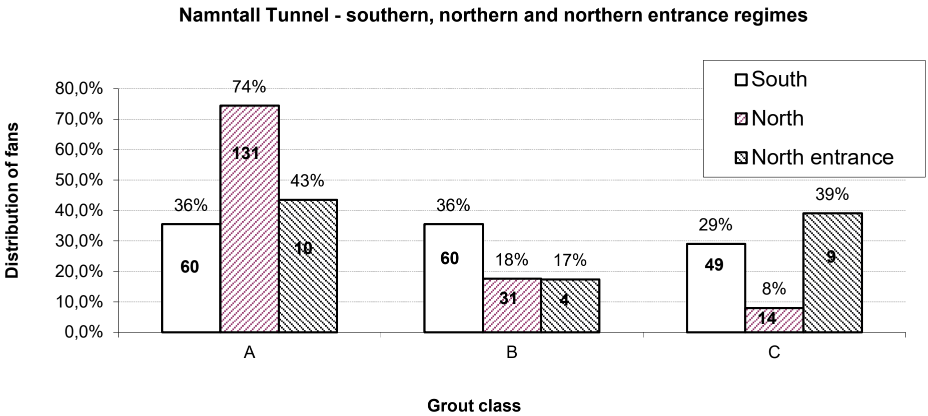

Figure 3-4. Distribution of grout classes for the south, north and north entrance regimes (from Stille 2016).

It is clear that the distribution of grout classes is influenced considerably the geological conditions. The water pressure test distribution for the southern part of the tunnel strongly indicates the grouting difficulty and the resulting water discharge, Figure 3-2. The difficulty obtaining a good grouting result is indicated here by the grout take in Figure 3-5. More specifically, the characteristics of the geological regimes can be expected to influence the grouting process and consequently the results of the grouting. A high concentration of zones and a combination of infilling and a high degree of jointing could be expected to be indicative of difficult grouting conditions. It is therefore reasonable to expect that for the southern part, where almost every grouting round crosses a zone, the grouting would be affected the most.

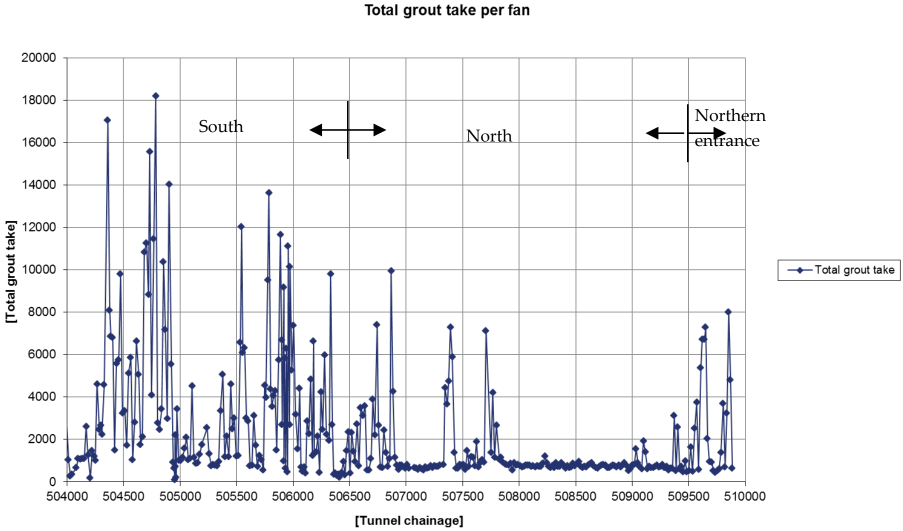

Figure 3-5. The total grout take for each fan, plotted against the location and tunnel chainage (from Stille 2016).

By arranging the grout data into the different grout classes, the distribution of grout take can be studied for each subgroup. The grout take per metre of borehole is studied to limit the influence of different hole lengths, number of holes etc. The data is presented in a histogram, where all three classes are represented, Figure 3-6. The Y-axis shows the percentage of holes and the X-axis shows the grout take. For example, around 13% of the grout class A holes have a grout take between 4 and 8 l/min. Furthermore, Figure 3-6 shows that there is an overall relationship between grout take and water loss in a fan. It also shows that there is a considerable number of holes in each fan that have a grout take approximately equal to the hole volume (3.2 l/m), although there is a significant difference between classes A and C.

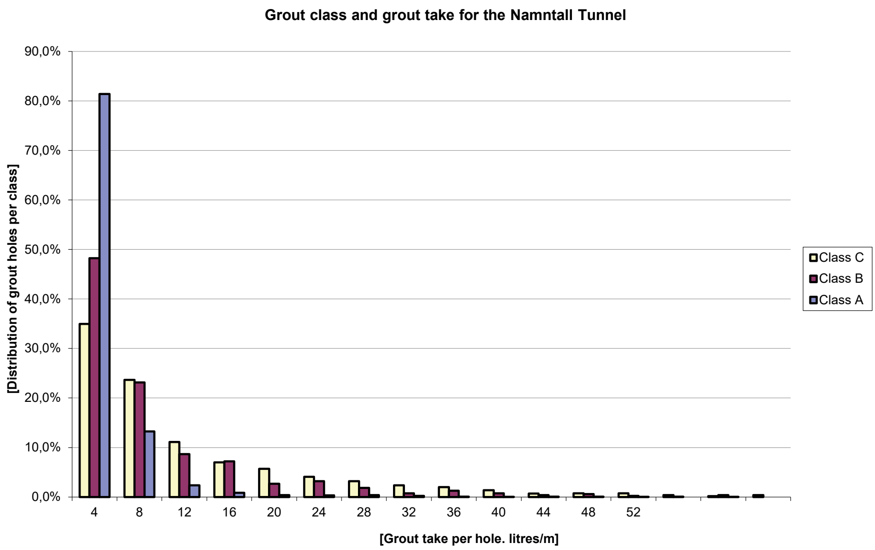

Figure 3-6. Histogram for the grout take for grout classes A, B and C. The figure shows the relationship between grout class and grout take distribution (from Stille 2016).

3.5 Summary

The grout take and the resulting water discharge indicate that a different approach to the grouting process could be suggested. Based on the large grout takes and the corresponding low grouting pressures that were common in grout class C rock, use of two grout rounds is recommended. It was also noted that the bottleneck in the grouting process was the grout mix capacities. The standard capacity for ordinary grout platforms was simply not sufficient to grout under higher pressure for a longer period of time in order to reach a stipulated penetration length for smaller fractures (calculated according to Gustafson and Stille, 2005).

The geological conditions that have the most prominent correlation with water pressure tests and grout take are the zone density. The properties of these zones (extent, water pressure test and grout data) indicate that the effective hydraulic conductivity may influence the general flow regime, further discussed in Chapter 5.

There is a correlation between grout take and the largest water pressure test for each grout fan. Such a relationship could be used as a prognosis tool/classification method with regards to the extent of grouting and water seepage.

The geological conditions and the distribution of water pressure test results are indicative of the water seepage and grouting difficulty (grouting results). It is considerably more difficult to grout areas with zones and fractures that have clay/ partial clay filling. The evaluation of the grouting process/results is discussed in (Stille 2009) and summarised in Chapter 4. This should be considered when procuring tender documents and when describing geohydrological conditions.

4. Experiences of the real-time grouting control method

4.1 Introduction

Herein it is shown how grouting theory can be used in practice through observations of grout flow and grouting time. Based on these 'real-time' data, predictions of grout penetration can be conducted, and the stop criteria can be adapted to the actual grouting process. The required penetration length needs to be defined at the design stage and is based on the hydraulic properties of the rock mass and a certain idea of grouting efficiency.

The real-time grouting control method concept involves calculating the grout penetration and controlling grouting in real-time by applying the development of the grout spread theories.

Grouting is completed when the grout penetration of the smallest fracture that needs to be sealed is above a certain minimum level (target value) or before the grout penetration for the largest fracture aperture reaches a certain maximum level (limiting value).

The spread of grout is governed by a number of complex relationships. This means that the issue of how or when the injection of grout should be stopped cannot be answered by simple rules of thumb. In recent decades there has been a substantial increase in the understanding of the mechanism behind the spreading of grout. Up until 1990, the understanding was more or less based on empirical knowledge as described by Houlsby, (1990). A deeper theoretical understanding of the mechanism, manifested by Lombardi (1985), Hässler et al. (1988), Gustafson and Stille (1996) and Eriksson et al. (2000), has had an impact on the development of both new stop criteria and new grouting materials.

Research in recent years has given us a better understanding of the water-bearing structures of the rock mass as well as analytical solutions of grout spread, see e.g. Gustafson and Claesson (originally submitted, 2005) later published as Gustafson et al (2013). In Hässler (1991), the concept of analysis of grout spread in real-time was discussed for the first time but was based on numerical calculations. The analytical solutions of the governing differential equations have made it possible to develop tools for analysing grout spread in real-time. The principle was first described in Gustafson and Stille (2005) and was further developed in Kobayashi and Stille (2007) and in Kobayashi et al. (2008).

The chapter summarise the information presented in Stille 2009.

4.2 Grouting control using RTGC

Grouting equipment has a computerised logging tool that continuously records different grouting parameters, such as grouting time, grouting pressure, grout flow and grouted volume. By following the grouting minute by minute, it is possible to predict the course of the grout flow and the penetration and also analyse the risk of uplift and jacking.

The procedures for the real-time grouting control method system are shown in Figure 4-1.

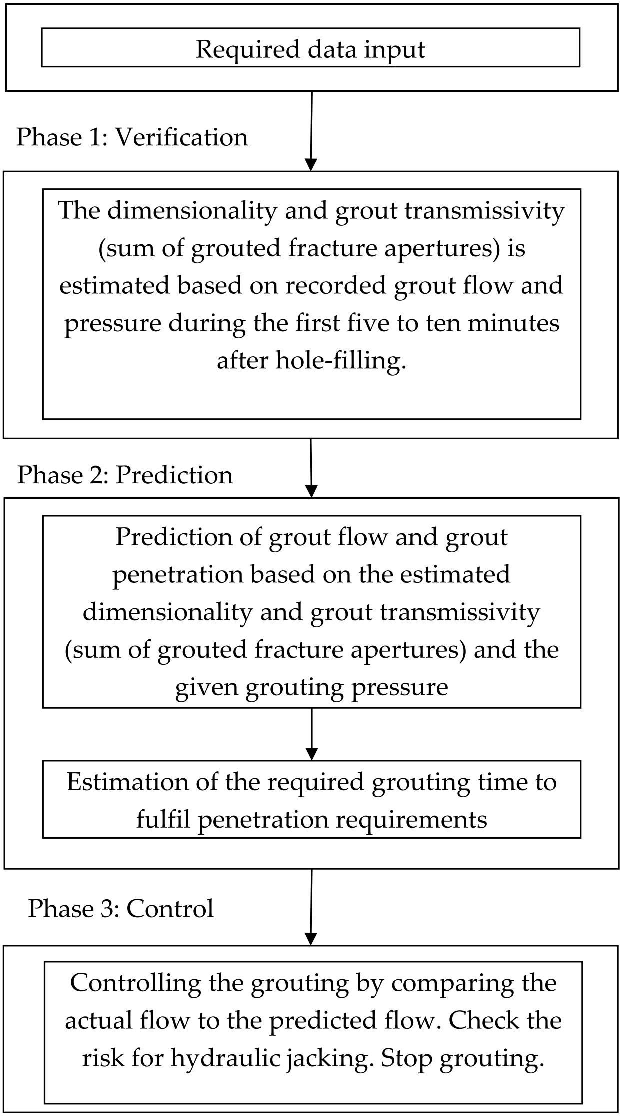

Figure 4-1. Grouting control procedures using the real-time grouting control method (from Stille 2016).

In the real-time grouting control method, the grout penetration is used as a stop criterion. The minimum penetration length and/or maximum penetration length will therefore be required as input data. Since the penetration length is proportional to the fracture aperture, the smallest aperture that needs to be sealed will be required as well as the aperture of the largest fracture.

In a hydrogeological description of the rock mass, the hydraulic fracture width is often used and can be calculated with the 'cubic law', see Snow (1965). However, experience shows that the physical or geometric fracture aperture (which can be calculated from the grout volume, grout flow and time) and the corresponding hydraulic fracture width often differ by about two times (bphysical≈2bhyd), as shown by Tsuji et al. (2012).

Estimation of grout transmissivity (sum of grouted fracture apertures, $\sum wb^2$ or Σb^3)

The theoretical grout volume can be calculated for both the 1D and 2D cases. In both cases it must be borne in mind that grout may enter several fractures, shown by the term $\sum wb^2$ for the 1D case and $\sum b^3$ for the 2D case. Both terms are a function of the fracture aperture, b, and the channel width, w, for the 1D case. Taking the cubic root of $\sum b^3$ may therefore give a reasonable approximate value of the largest 'possible' aperture for the 2D case. Figure 4-2 (Äspö Hard Rock Laboratory Data, see for example Hernqvist et al 2008) shows a comparison of calculated and measured injected volumes after the borehole was filled with grout. The parameter, $\Sigma b^3 = 5{,}5 \cdot 10^{-13}$ m^3, was determined by minimising the sum of squared differences between them. The assumption that the whole transmissivity corresponds to one fracture gives b = 82 µm. For the 1D case, the width of the channel must also be estimated.

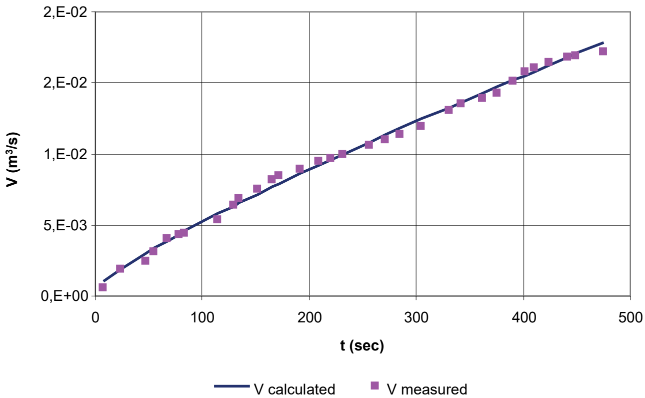

Figure 4-2. Comparison of calculated and measured grout volumes, $\sum b^3 = 5{,}5 \cdot 10^{-13}$ **m^3(from Stille 2016).**

Calculation of the risk of hydraulic jacking and uplift

Grouting will induce stresses in the rock mass, which may cause block movements, hydraulic jacking or uplift. The risk of uncontrolled deformations depends on the pressure and volume and must be avoided (Lombardi and Deere, 1993). Studies carried out by Brantberger et al. (2000) showed that the risk of hydraulic uplift was analysed better by introducing grout penetration instead of volume. Since the penetration length will be calculated during the grouting process in the real-time grouting control method, it may be possible to check the risk of uplift in real-time. Comparison of predicted and measured grout flow, or estimation of grout transmissivity (fracture aperture), will also offer a direct opportunity to discover hydraulic jacking.

The term uplift in this context corresponds to the ultimate bearing capacity of rock mass. Hydraulic jacking may occur at a lower level (Gothäll and Stille, 2009). It is important to point out that there are also other cases, such as leakage of grout into the face or jacking of the face, that also need to be considered. Such risks can be controlled by reviewing the flow-pressure data. The largest risk of uplift is connected to the longest penetration. The penetration length for the largest fracture aperture should therefore be used in the calculations. In Brantberger et al. (2000), the permissible lifting force is calculated using an assumption of a circular open fracture. The uplift or risk of hydraulic jacking has been further developed by Rafi (2014) and the hydromechanical behaviour with regards to the fracture geometry was studied by Thörn (2015).

4.3 Experience from different case histories

The theory of grout penetration and grout flow has been investigated in four projects in order to demonstrate the applicability of the theory. The projects, case histories, are located in different parts of Sweden, in different geologies and at different depths. For each case, grout data have been recorded and the theory of dimensionality, the estimation of fracture apertures and the theoretical grout flow have been calculated. Some relevant samples are shown.

The case histories are from Äspö Hard Rock Laboratory at the 450 m level, the Northern Link road projects in Stockholm and the Bothnia Line rail project in central Sweden.

Short summaries of the project data are shown in Table 4-1.

Prediction of grout flow

After the flow dimensionality of the hole has been calculated, the sum of the fracture apertures and the theoretical flow can be calculated. The calculated flow in Figures 4-3 to 4-5 is related to the analysed dimensionalities. The calculation of the theoretical flow follows the pressure curve and the resulting theoretical flow curve can be compared to the actual recorded flow curve. In the figures, the nominal Q predicted is used as the theoretical flow.

Table 4-1. Case history summary (from Stille 2016).

Case history	Geology	Depth (data from)	Inleakage (after grouting)	Comments
Äspö HRL	Äspö diorite, very competent	450 m	<4.5 l/min.100 m	Swedish nuclear repository research centre. Very good quality data. Kobayashi et al. (2008).
Northern Link NL101	Granite, gneiss, competent	10-20 m	~2 l/min.100 m	Road tunnel system, Swedish Road Administration. Long (20 m) planar and smooth fractures. Bedding planes. Bohlin and Urtel (2008).
Northern Link NL34	Sedimentary gneiss, fractured	20 m	~4 l/min.100 m	Road tunnel system, Swedish Road Administration. Fractured rock. Zones. Bruno, (2009).
Bothnia Line (E3541) Namntall South	Metagreywacke, partly very fractured	80 m	~20 l/min.100 m	Rail-road tunnel system, Swedish Rail/Road Administrations. Very fractured in part. High frequency of zones. Stille and Andersson (2008).

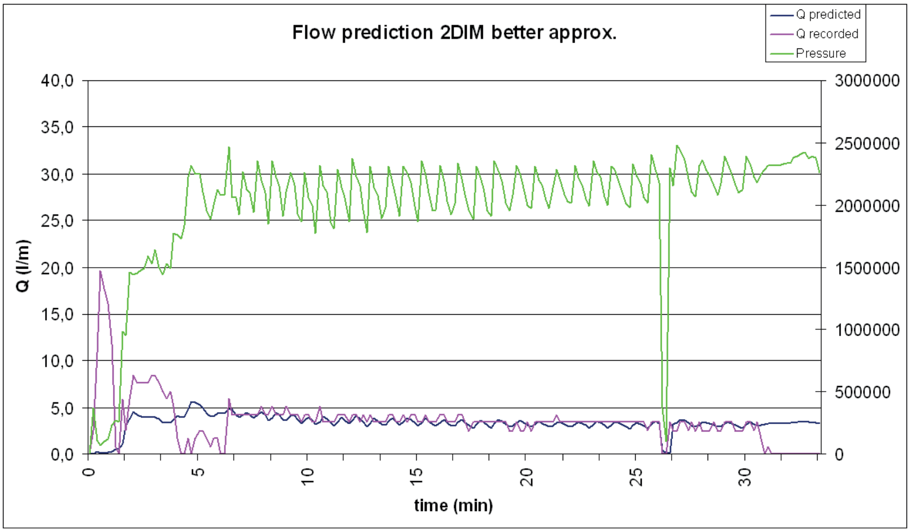

Figure 4-3. Comparison between the measured flow and the predicted flow for hole 31, fan 6, 2D flow. The primary Y-axis show grout flow [l/min] and the secondary Y-axis show pressure [Pa] (from Stille 2016).

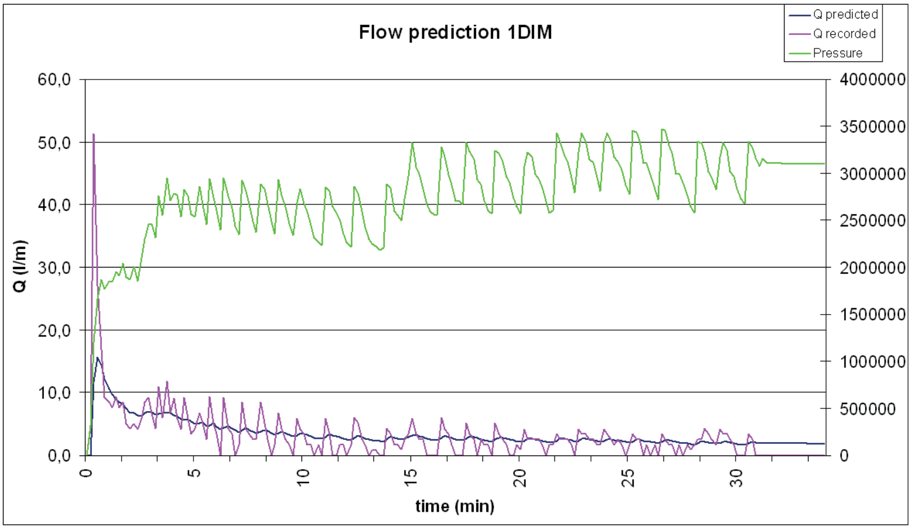

Figure 4-4. Comparison between the measured flow and the predicted flow for hole 1, fan 7, 1D flow. The primary Y-axis show grout flow [l/min] and the secondary Y-axis show pressure [Pa] (from Stille 2016).

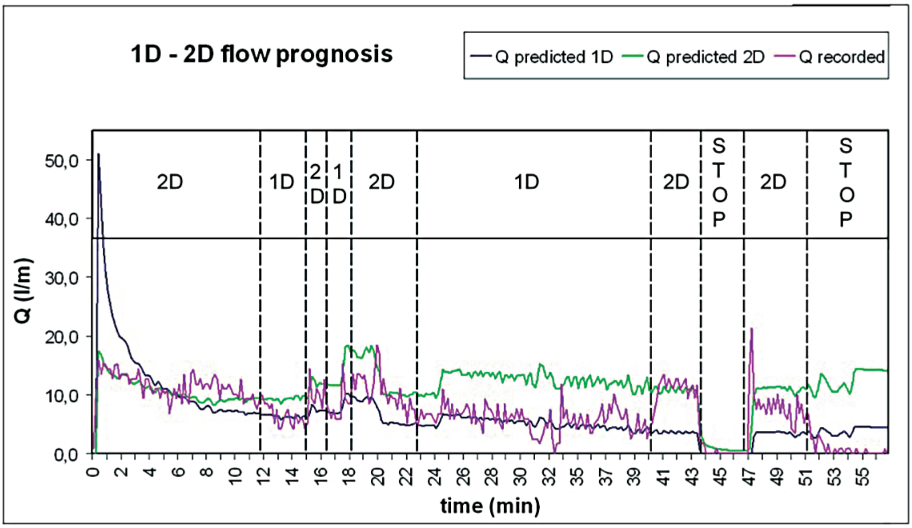

Figure 4-5. Comparison between the measured flow and the predicted flow for hole 22, tunnel 301 section 1547, varying flow. The primary Y-axis show grout flow [l/min] (from Stille 2016).

As can be seen in Figure 4-5, which shows both 1D-predicted flow and 2D-predicted flow, the recorded grout flow follows the respective theoretical curves according to the varying dimensionality.

Jacking

Sometimes other anomalies were recorded during grouting. Some of these include uplift or jacking of the ground for shallow grouting operations, see Figure 4-6. Others seem to influence smaller areas of the rock mass, see Figure 4-7. A case from NL33-34 is shown in Figure 4-6, where after 26 minutes of grouting the pressure is increased, resulting in an immediate increase in flow. The flow then decreases and is followed by a continuous increase in flow until the grouting is aborted after about 40 minutes despite the fact that the pressure was reduced. In Figure 4-7, the grouting pressure increased slightly, producing a non-proportional increase in grout flow. As can be seen, after this initial increase the grout flow diminishes, indicating that the action in the rock mass is more local, including possible rock movements in the fracture zone opening one fracture and closing another.

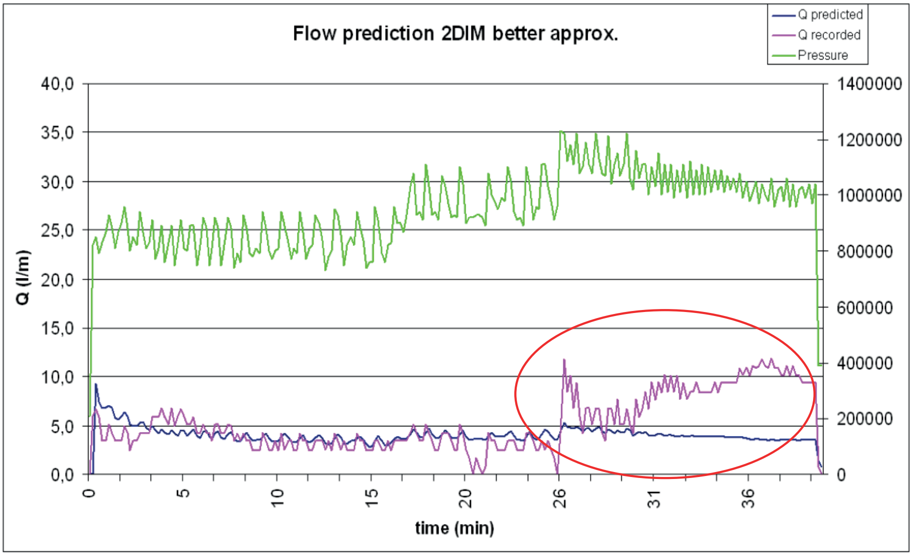

Figure 4-6. Possible uplift or jacking of the rock mass, hole 16, fan, 2D flow. The primary Y-axis show grout flow [l/min] and the secondary Y-axis show pressure [Pa] (from Stille 2016).

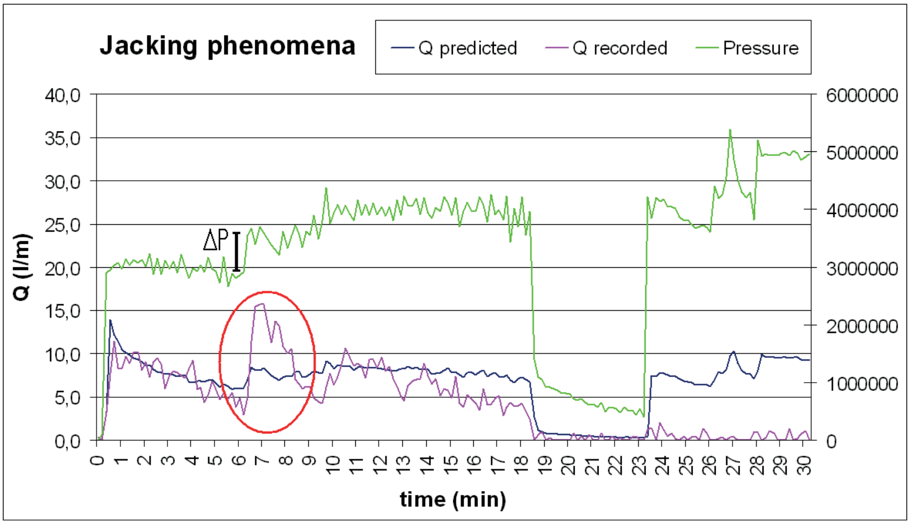

Figure 4-7. Possible jacking influencing a local area of the rock mass, 2D flow. The primary Y-axis show grout flow [l/min] and the secondary Y-axis show pressure [Pa] (from Stille 2016).

4.4 Summary

The concept behind of a real-time grouting control method is to control the grouting in real-time by applying the grout spread theories. It is possible, by following the grouting minute by minute, to predict the course of the grout flow and analyse the risk of uplift and jacking.

This could be of particular interest for shallow surface grouting or grouting close to other subsurface areas where the possibility of detecting jacking or uplift of the rock mass could be of critical interest. The most significant action from the examples presented would have been the ability to abort grouting or lower the grouting pressures. These issues herein are further discussed in Stille 2009.

Verification of the real-time grouting control method, with field data from four tunnel projects in Sweden, is presented herein. The calculated flow dimensionality, the calculated fracture apertures and the calculated grout flows were quite close to those measured. This indicates that the real-time grouting control method is applicable to real grouting design and control.

5. Distribution of rock mass hydraulic conductivity and its application to rock engineering problems

5.1 Introduction

The most common sources of geohydrological information for Scandinavian tunnelling projects with regard to hydraulic properties come from water pressure tests in 3 to 9 m lengths. These are sometimes performed as double packer tests (the tested part of the borehole is closed off using two packers) but often as single packer tests (where one packer is used, and the results are subtracted from the previous measurement to produce the result for a specific part of the borehole). During construction, and as part of the grouting work, water pressure tests are sometimes performed, as was the case with the Namntall Tunnel described by Stille and Gustafson (2010). The accuracy of such 'during construction' methods varies depending on the equipment and in the Namntall project it was concluded that results below 0.2 and over 4 Lugeons (litres/min,m,MPa) were unreliable (corresponding to a transmissivity of T = $7{,}4 \cdot 10^{-7}$ and $1{,}5 \cdot 10^{-5}\, m^2/s$, respectively).

The main focus here is the statistical distribution of the water pressure tests and the transmissivity calculated from those tests, the theory and method for scaling the distribution to the appropriate level, and the analysis of the probability that the rock mass has a mean hydraulic conductivity lower than a set value. Three important issues need to be considered.

1. The spatial variability of hydraulic conductivity in the rock mass (distribution of a property).
2. The scale of the conducted measurements (scale dependency).
3. Water seepage flow regimes and appropriate values for the seepage calculations.

5.2 Effective hydraulic conductivity and mean values

The transmissivities or hydraulic conductivities in a rock mass are neither uniform nor isotropic but vary considerably from rock volume to rock volume. The measurements show that the rock mass transmissivities can be described statistically and it is therefore of interest to study the distribution and the influence of the studied rock volume with regard to size and mean values, see also Holmén (1997). From an engineering point of view, the calculation of water inflow to a tunnel or under a dam should be done as accurately as possible to define the requirements for the grouting work but also with regard to the environmental court rulings. This in turn requires either the use of relevant mean values or the entire statistically described distribution.

Gustafson (2009) indicates in the division of different scales that the statistical models for estimating the distribution of the hydraulic properties are different. For the 3-30 m scale, several fractures intersect the test section or the tunnel and fracture independence cannot be assumed. Instead, the interval transmissivities are a better description of the hydraulic properties of this scale. De Marsily (1986), for example, described the interval transmissivities as lognormally distributed.

Holmén (1997) defined the effective hydraulic conductivity of a rock volume as a representative mean value depending on the flow regime and the statistical distribution of the hydraulic conductivity. The flow regime in turn depends on the heterogeneity of the rock mass. The different flow regimes are shown in the attachment. The different mean values could also be used to estimate the upper and lower boundaries of the hydraulic conductivity where, for example, the arithmetic mean could be considered to be the upper boundary and the geometric mean the lower boundary. The influence of heterogeneous conditions (such as fracture zones) could influence and change the relevant model and thus the relevant mean for the engineering problem. It is clear that the better the description of the distribution, the more reliable the prognosis. The identification of a relevant statistical distribution for section transmissivities was performed by, for example, De Marsily (1986), identifying the lognormal distribution as relevant for most data.

5.3 Evaluation of lognormal distribution parameters of section transmissivity in different cases

The evaluation of lognormal distribution parameters has been performed for four cases (two cases are shown here, Figures 5-1 and 5-2). For these cases, the water pressure test results from the tunnelling operation are analysed and the lognormal distribution is fitted to the measured data. In all geotechnical investigations there are a number of uncertainties, such as measuring accuracy, measuring span and handling errors etc. related to the measured result. It is important to recognise that there are limits to the reliability of the measurements when it comes to interpreting the data.

The water pressure tests (WPT) presented in this analysis are performed in the grout holes over the whole length of the borehole for a limited period of time, 3-5 min, with a pressure of 0.5 MPa above the groundwater pressure. The transmissivities are calculated from the WPT tests and are presented as CDF's 'Grout hole test results' in the figures.

The transmissivity values from the water pressure tests are fitted to a lognormal distribution. The lognormal distribution has the property that the log values of the stochastic variate (X) are normally distributed with $E(\ln X)=\lambda$ and variance$=\xi^2$. The case records are presented using the statistical values of the normal distribution of ln(X) and the mean transmissivity value, μ_T, and the standard deviation σ_T (Figure 5-1 and Figure 5-2). The application of the lognormal distribution and the identification of the mean value and the standard deviation should be performed for the range of reliable data. The easiest way to perform such a curve fitting is to manually change the λ and the ξ values until a good fit is achieved.

The transmissivity results (over a section length of 20 m) and the transmissivity data are presented in the figures. The largest discrepancies are for both small and large values, as expected due to larger measurement errors.

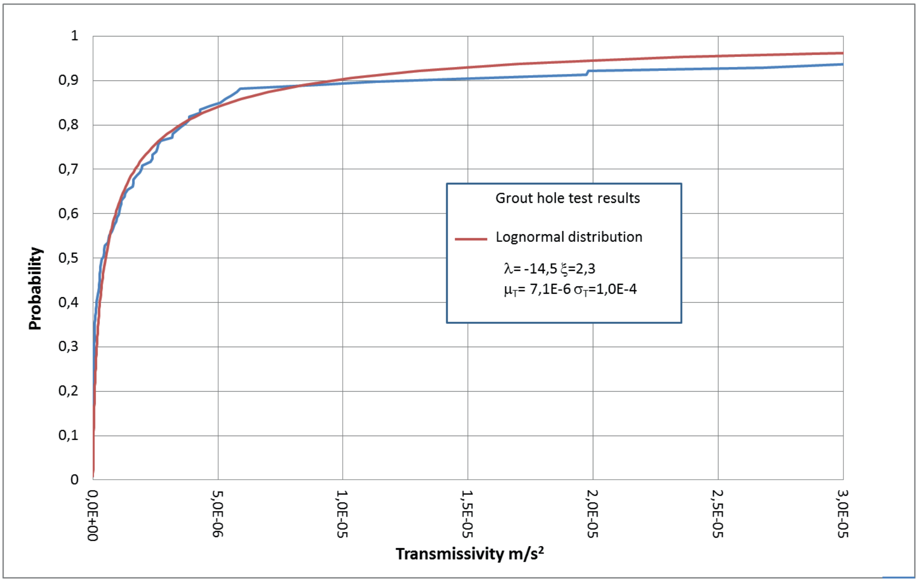

Figure 5-1. Transmissivity distribution for the Odenplan Station on the City Line.

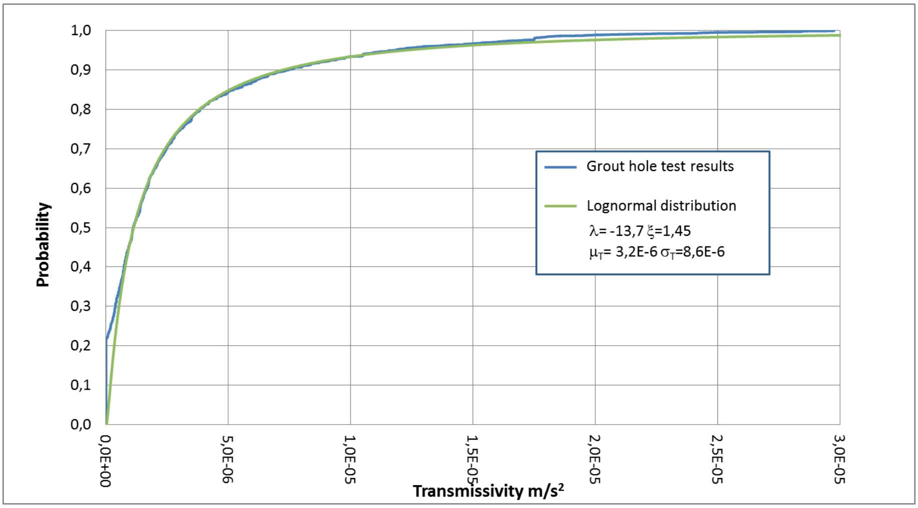

Figure 5-2. Transmissivity distribution for the southern part of the Namntall Tunnel.

The fit is very good, which confirms the use of the lognormal distribution to describe the variation in section transmissivity.

5.4 Influence of scale

One of the properties of a lognormal distribution is that the mean values can be added for two lognormal distributions. This means that it is possible to scale the data to a size that is relevant for the engineering problem. If the transmissivity data is measured over a length, L_{base}, and the transmissivity is investigated for a length, L, then the mean values can be calculated from:

$$n = {}^{L}/_{L_{base}}$$

The arithmetic mean value and the standard deviation will then become:

$$T_a = \mu_{T,L} = n \cdot \mu_{L_{base}}$$

$$\sigma_{T,L} = \sqrt{n} \cdot \sigma_{L_{base}}$$

The geometric mean value of the transmissivity distribution can be calculated for different lengths, L, as:

$$T_g = \frac{\mu_{T,L}}{\sqrt{1 + \left(\frac{\sigma_{T,L}}{\mu_{T,L}}\right)^2}}$$

In this case the hydraulic conductivity is a useful value since it normalises the values of T and makes them comparable for different scales. The hydraulic conductivity can be described as K = T/L and the mean value is calculated as:

$$\mu_K = \frac{1}{L} \cdot \frac{L}{L_{base}} \mu_{T,L_{base}} = \frac{\mu_{T,L_{base}}}{L_{base}}$$

The arithmetic mean is evidently independent of an increase in scale. However, the standard deviation is dependent on the length, L, according to:

$$\sigma_K = \frac{1}{L} \cdot \sqrt{\frac{L}{L_{base}}} \cdot \sigma_{T,L_{base}} = \frac{\sigma_{T,L_{base}}}{\sqrt{L \cdot L_{base}}}$$

The geometric mean of the hydraulic conductivity for a length L can therefore be calculated using:

$$K_g = \frac{\mu_{K,L}}{\sqrt{1 + \left(\frac{\sigma_{K,L}}{\mu_{K,L}}\right)^2}}$$

According to the definition for a lognormal variate X, the variance of the normalised function is $\xi^2 = Var(lnX)$ and can be expressed for the length, L, as:

$$\xi^2 = ln\left(1 + \frac{{\sigma_{T,L_{base}}}^2}{{\mu_{T,L_{base}}}^2} \frac{L_{base}}{L}\right)$$

Using 'Matheron's conjecture', the relative hydraulic conductivity, K_D, can be described, depending on the flow dimension, as:

$$K_D = K_g \cdot e^{\left[\xi^2\left(\frac{1}{2}-\frac{1}{D}\right)\right]} \; where \; D = flow \; dimension$$

The statistics are based on the prerequisite that the data are statistically independent. The correlation distance for section transmissivity probably depends on the actual rock mass characteristics. For fractures in a hard crystalline host rock, the correlation distance has been estimated by Butron (2012) at 2-8 m. The scale of fluctuation indicates that the measured data can be expected to be more or less independent.

For most problems in tunnelling situations, the length of a grouting fan is on an engineering applicable scale, although an extrapolation for the whole tunnel may be required to calculate tunnel seepage. The relationship between the statistical parameters and the hydraulic conductivity for different section lengths can be calculated by adding the transmissivities and dividing μ_T/L, which will produce the arithmetic mean, K_a. The arithmetic mean, K_a, is independent of scale while the standard deviation will decrease with scale. Since the geometric mean, K_g, depends on the standard deviation, the implication is that the geometric mean and K_{3D} will increase with scale and approach the arithmetic mean for very large scales.

The calculation presented in Figure 5-3 is based on data from the Odenplan Station on the City Line, Table 5-1. The measured section intervals range from 3 m to around 20 m for these tunnels (water pressure tests before grouting). It can be shown that the arithmetic mean value for the 20 m sections is about equal to the 3 m sections. However, the standard deviation decreased from $9.7 \cdot 10^{-6}$ to $5 \cdot 10^{-6}$, which confirms the general finding that the geometric mean should increase with scale.

Table 5-1. Statistical data for the Odenplan Station on the City Line based on measured data for 3 and 20 m intervals.

Odenplan Station	3 m	20 m
σ [m/s]	$9.7 \cdot 10^{-6}$	$5 \cdot 10^{-6}$
ξ [-]	2.6	2.3
K_a [m/s]	$3.3 \cdot 10^{-7}$	$3.55 \cdot 10^{-7}$
K_g [m/s]	$1.1 \cdot 10^{-8}$	$2.5 \cdot 10^{-8}$

At the Odenplan Station on the City Line, the water pressure tests were carried out for both 3 m and 20 m sections in the same boreholes.

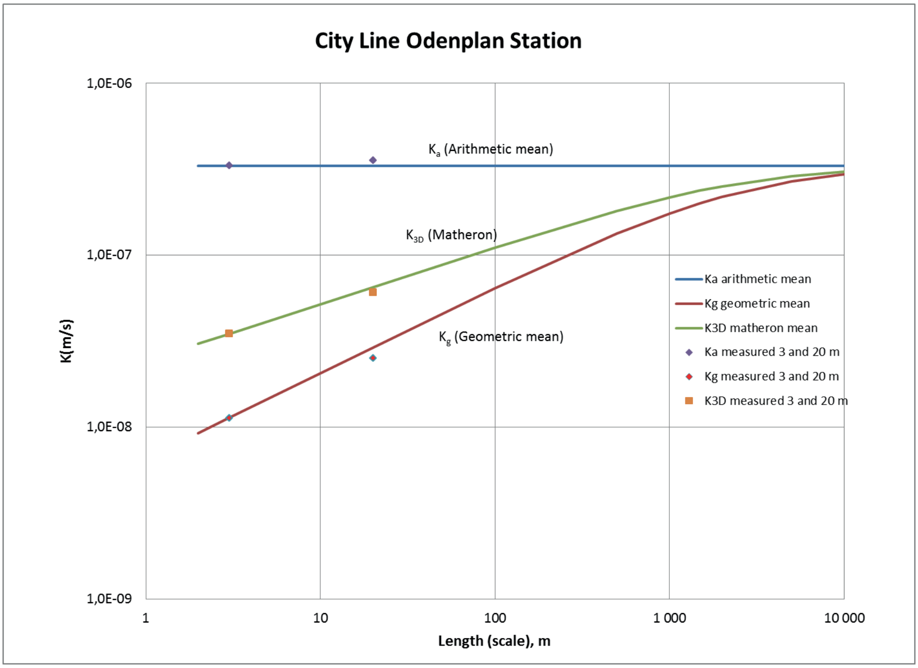

Figure 5-3. Mean values for the Odenplan station on the City Line.

By studying the density function for the different scales, some light is cast on the properties of the hydraulic conductivity distribution. The density function can be plotted to illustrate the variation in the likelihood of encountering a specific value, Figure 5-4. In the figure, the calculated density functions are presented for the intervals 20 m, 100 m, 500 m, 1,000 m and 5,000 m. These represent the scales from a grout hole section (20 m) to the whole tunnel.

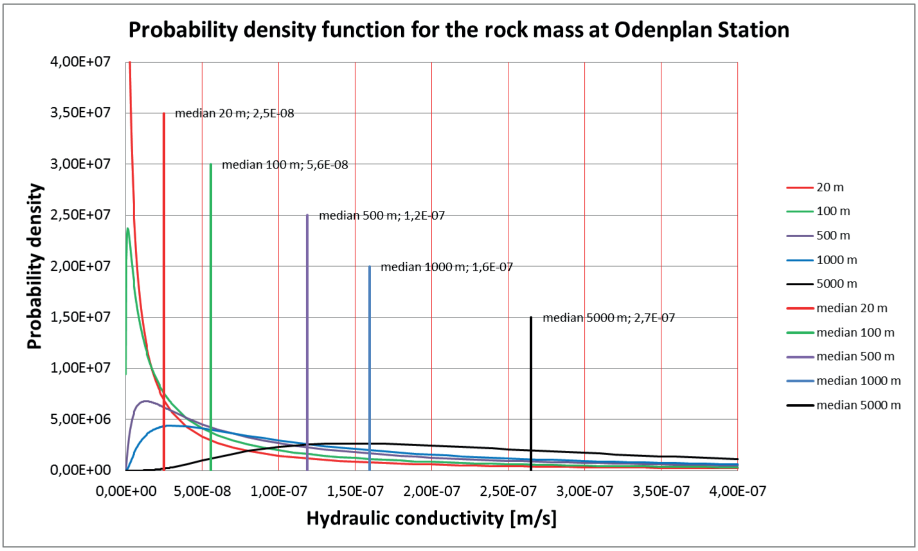

Figure 5-4. Probability density function for different scales (intervals) of hydraulic conductivity at the Odenplan Station.

The results are quite interesting in that they illustrate the variation and high probability that a very low value is encountered for the 20 m interval (median) whereas for the larger intervals the probability of achieving a certain value is more evenly distributed across the measured spectrum. The median represents the probability that for a stochastic tunnel or part of a tunnel, there is a 50% chance that the hydraulic conductivity is lower than or equal to the median.

It can be shown that for a lognormal distribution the probability that the variable X is smaller than b

$$P(X < b) = \Phi\left(\frac{lnb - \lambda}{\xi}\right)$$

The probability that the hydraulic conductivity will be larger than a certain figure is a relevant question for the seepage analysis. The statistical data can be used to calculate this probability for the different scales. As an example, the probability that the hydraulic conductivity is smaller than the mean value and twice the mean value is calculated, Table 5-2.

Table 5-2. Data from the Odenplan Station and calculation of the probability that K< K_{limit} (□K; □□K□□□for different lengths.

Length	μ_K	σ_K	CV (σ/μ)	Median $\mu/\sqrt{1+(CV)^2}$	λ	ξ	Φ P(K< μ_K)	Φ P(K< $2\mu_K$)
3	$3.55 \cdot 10^{-7}$	$1.3 \cdot 10^{-5}$	36.4	$9.8 \cdot 10^{-9}$	-18.45	2.68	0.91	0.95
20 m	$3.55 \cdot 10^{-7}$	$5.0 \cdot 10^{-6}$	14.1	$2.5 \cdot 10^{-8}$	-17.50	2.3	0.88	0.93
100 m	$3.55 \cdot 10^{-7}$	$2.2 \cdot 10^{-6}$	6.3	$5.6 \cdot 10^{-8}$	-16.70	1.92	0.83	0.91
500 m	$3.55 \cdot 10^{-7}$	$1.0 \cdot 10^{-6}$	2.8	$1.2 \cdot 10^{-7}$	-15.95	1.48	0.77	0.89
1000 m	$3.55 \cdot 10^{-7}$	$7.1 \cdot 10^{-7}$	2.0	$1.6 \cdot 10^{-7}$	-15.65	1.27	0.74	0.88
5000 m	$3.55 \cdot 10^{-7}$	$3.2 \cdot 10^{-7}$	0.9	$2.7 \cdot 10^{-7}$	-15.14	0.76	0.65	0.9

5.5 Summary

For an ungrouted tunnel it is suggested that the effective hydraulic conductivity is represented by a mean value ranging from the 3D mean (model d) to the arithmetic mean (model b) with a scale (block size) that depends on the depth of the tunnel but not less than 20 m. In the case of a grouted tunnel, the grouting will significantly change the hydraulic conductivity close to the tunnel and the pressure gradient will act across the grouted zone. The water flow for the grouted tunnel will be parallel (model b) and the effective hydraulic conductivity would be represented by the arithmetic mean (model b) after grouting. It is recommended that the scale is equal to the grout hole length, i.e. around 20 m.

The mean values can be analysed and scaled according to the presented theory. The lognormal distribution seems to fit the transmissivity data. It is important to take the accuracy of the measuring system into consideration when evaluating the data. In this study there seems to be an interval of $7.4 \cdot 10^{-7}$ to $1.5 \cdot 10^{-5}$ m^2/s, which is more reliable. When analysing the scale effects, an increase in scale reduces the variance and the geometric mean increases. For tunnels over 500 m in length, the difference between the mean and arithmetic mean values seems to be relatively small. Considering the general trend in the statistical data, it would seem reasonable that arithmetic mean values should be used for calculating seepage into most tunnels longer than 100 to 1,000 m. However, the requirement is that the data is from the same hydrogeological domain. The variation in data should be considered, especially for cases with a limited number of tests. By applying the statistical data, the variation can be used to describe a likely interval of mean hydraulic conductivity that can be used as an upper and lower boundary for calculating seepage.

The geometric mean values represent the 50% value of the rock mass hydraulic conductivities. This means that there is a 50% chance that the actual mean value is higher than the geometric mean. In the case of the arithmetic mean value, the chance that the actual mean is higher than the arithmetic mean would be about 10%. There seems to be little difference between the probability that the mean value is higher than the arithmetic mean or twice this value. This is a function of the relative low probability to encounter the higher values in the distribution.

The probability that the hydraulic conductivity is predicted accurately is relatively low – 50-90% depending on the mean values that are used. This calls for a design approach that can be adjusted for actual conditions equivalent to an observational method where the inflow and the grouting work are followed up. It is recommended that the initial part of the tunnel is followed more closely since any changes to the design or grouting operation have a greater impact at an early stage.

6. Analysis of water seepage after grouting

6.1 Introduction

The accuracy of predicting seepage is of considerable interest since seepage is part of environmental court rulings, the seepage requirements and the procurement process. Consequently, the seepage calculation influences not only the requirements but also the grouting process and the total project cost.

The conceptual models for calculating seepage into a grouted tunnel are illustrated in Figure 6-1. In principle, these are either a homogeneous model where the mean hydraulic conductivity, K_0, of the rock mass is assumed to be valid over the tunnel or actual domain, or a fracture model where each fracture contributes individually to the seepage.

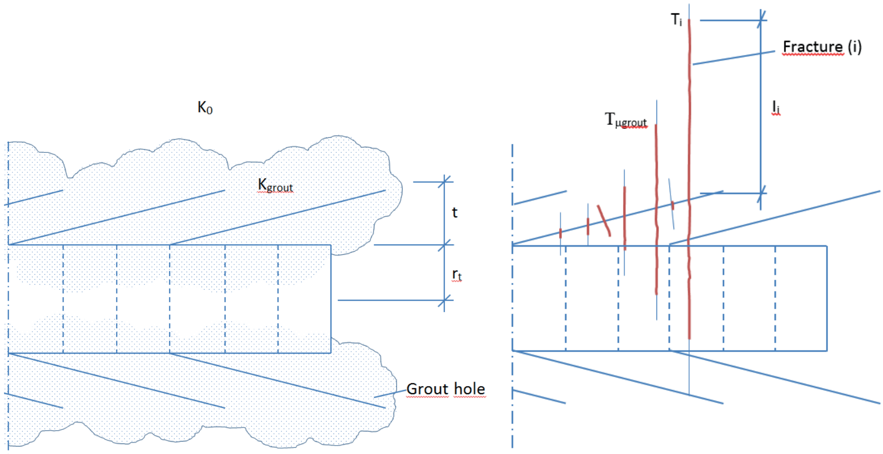

Figure 6-1. Illustration of two conceptual models (homogenous and fracture) for estimating tunnel seepage.

In hard crystalline rocks the rock mass is naturally neither homogeneous nor porous and water seeps through fractures in the rock. The fractures are not independent and may be connected especially for fracture zones. During grouting, the grout penetrates some of the fractures. However, the penetration length for the smaller fracture apertures will be very short or even almost zero and for larger fracture apertures the penetration length will be very long, Figure 6-2. This variation in penetration length and grouting result indicates that the models for analysing seepage may be discussed and modified to accommodate a better understanding of the fracture/grout penetration relationship.

6.2 Grout penetration and grouting results

Several previous back calculations have been performed to analyse the achieved grouting result, K_{grout}, based on the homogeneous model, see e.g. Hernqvist (2011) or Tsuji et al (2012). However, some practical issues related to the grouting work need to be considered when evaluating the grouting result. During grouting, the grout penetrates the fractures down to a certain fracture aperture depending largely on the grout mix and the grain size distribution, Draganovic (2009). Furthermore, the grout penetrates a certain distance into the rock mass depending on the fracture apertures, grout properties, grouting pressure etc., Gustafson and Stille (2005).

The grout filtration tests (EN 14497) of the cement grout from the cases mostly show a bmin of about 50 μm and a bcrit of about 80 μm, which indicate that no grout penetrates the 50 μm physical fracture and that filtration limits the penetration up to an 80 μm physical fracture. See Eriksson and Stille (2005) for a discussion of the filtration test.

It is shown that for Namntall North (Stille and Gustafson, 2010), the Bangård Tunnel (Tsuji et al 2012) and the Odenplan Station (Stille et al 2014), the grout take for about 50 - 70% of the holes is about equal to the hole volume (< 1 l/m). The grout flow stop criteria in most of these cases indicate a grouting time of 1-5 min after the hole has been filled.

For a grouting pressure of 20 bar, the theoretical penetration length, according to Gustafson and Stille (2005), can be calculated as 0.6 m and 1.3 m for a 50 μm fracture and for the 80 μm fracture 1.0 m and 2.2 m with a grouting time of 1-5 min, Figure 6-2. The figure shows that the penetration length is proportional to the fracture aperture, although no filtration is considered for the small fractures, which means that the actual penetration length is probably shorter. The calculation is performed with grout properties typical for most of the grout used in the presented cases with the yield limit and viscosity values shown in the figure.

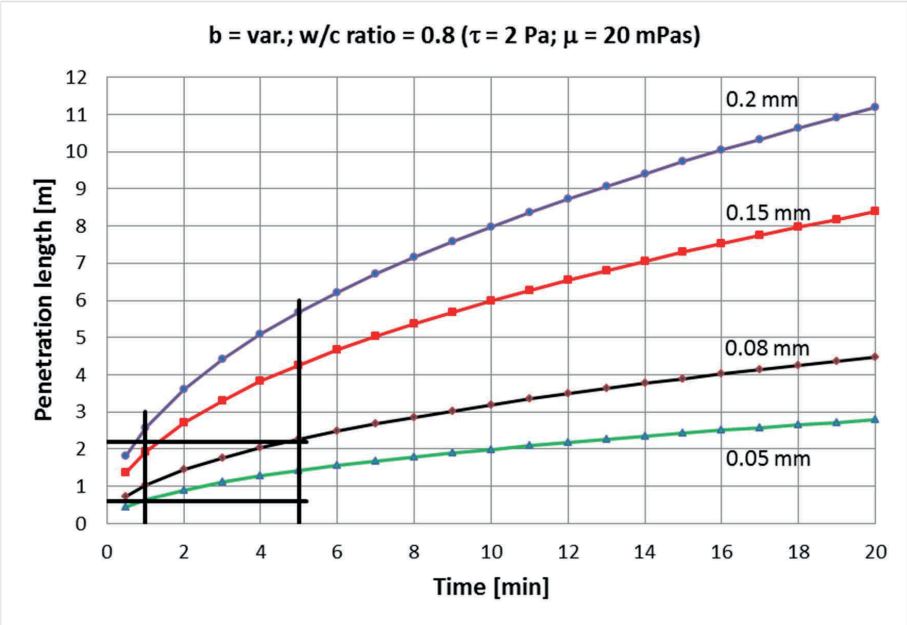

Figure 6-2. Calculated penetration length, according to Gustafson and Stille (2005) for short grouting times. Each curve represent the penetration length for a specific fracture aperture. The straight lines indicate 1 and 5 min grouting time and the penetration length for 0.05 mm and 0.08 mm fracture aperture. The grout pressure is 20 bars.

At Namntall North and South the end point distance of the grout holes was equal to or larger than 4 m and for the City Link Bangård Tunnel and Station Oden the end point distance of the grout holes were about 2.5 m. It seems clear that for fracture apertures smaller than about 80 µm, the penetration length is too small to consider the fractures as fully grouted if stopped within 5 min of grouting time as is the case for many of the grout holes with small grout takes.

The grout flow can be calculated, using the same theory, for each fracture, Figure 6-3. It is shown that the grout flows for the smaller fractures are very small as is exemplified for the 0.08 mm fracture. The calculated grout flow for this fracture is about 0.3 l/min. This is especially interesting when discussing the grout results in relation to grout take, the number of fractures and the grout flow stop criteria, exemplified for Station Odenplan as 5 l/5min or 1 l/min. For a grout take of <1 l/m and a grout time of 5 min this would indicate a grout flow of <0.2 l/min,m and consequently a fracture aperture <0.08 mm/m. If only one fracture in the hole contributed to the grout take the size of this fracture would theoretically be 125 µm (grout flow = 1 l/min).

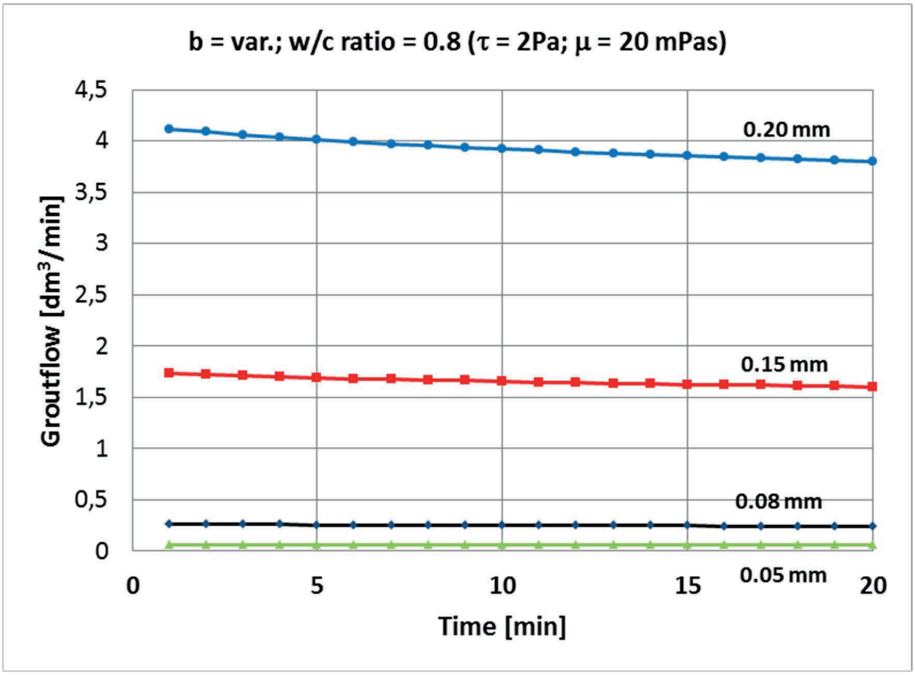

Figure 6-3. Calculated grout flow, according to Gustafson and Stille (2005). Each curve represent the grout flow for a specific fracture aperture. The grout pressure is 20 bars.

Considering the number of fractures that each grout hole crosses it seems likely that for most holes the fracture aperture is quite small. However, it should also be noted that the grout flow meter has quite poor accuracy. The total grouted volume is commonly seen as a more accurate value.

The theory and the cases in this thesis lead to the following line of reasoning:

- The larger fractures may for the Namntall North, the City Link Bangård Tunnel and the City Link Station Odenplan be considered sealed.
- Fractures with small apertures have theoretically small grout take.
- Grout holes with small grout take have short grout time due to the grout flow stop criteria.
- Grout holes with short grouting time (and consequently small apertures) have theoretically short penetration lengths.
- About 50 - 70% of the grout holes have little grout take.
- The general spacing of the fractures show that there generally may about 1 to 5 fractures/m intersecting the grout hole. Each fracture contributes to the total grout take.

By combining the case histories, the grout flow stop criteria, the number of fractures and the theoretical grout flow calculation it seem likely that for the majority of the grout holes the largest fracture taking grout is smaller than about 80 µm.

After considering the execution of the grouting, it is reasonable to conclude that for these cases about 50 - 70% of the rock mass can be considered more or less ungrouted which is important for the conceptual model of calculating water seepage. The remaining 30 - 50% of the holes shows a variety of grouting times and varying grouting pressures. Considering that the rock mass is made up of a number of different-sized fractures it follows that the transmissivity of a section of rock mass that has been grouted must be lower than a certain value indicated by the penetrability of the grout. It is also reasonable to assume that the grouting results (expressed as transmissivity of a section of the rock mass after grouting) cannot be equal for a 'poorer' rock mass with more fractures, as is the case for Namntall South compared to a 'good' rock mass as for Namntall North, Stille and Andersson (2007).

6.3 Transmissivity after grouting

Depending on the scale and flow mode, the arithmetic mean hydraulic conductivity may be representative for water seepage into a tunnel whereas the 3D mean hydraulic conductivity may be representative for the rock mass in general if a distribution is considered for representative block sizes. Each grout hole penetrates a relatively short and inhomogeneous part of the rock mass. The results after grouting should be considered for a length (scale) equal to the grout holes. This is in accordance with the general decision criteria during the execution of the works and the evaluation method in the design.

For a fracture model, the interval transmissivity, T_i, in each hole is a sum of the individual fracture transmissivities, T_f.

$$T_i = \sum_{1}^{n} T_f$$

If the grout penetrates and seals the fractures down to a certain fracture aperture, then the interval transmissivity of the grouted section of rock mass, T_{igr}, can be described as

$$T_{igr} = \sum_{1}^{n} T_f - \sum_{x}^{n} T_{grouted\ fracture}$$

Where n is the number of fractures in the interval and x represent the number of the smallest grouted fracture (in order) for the interval and $T_{groutedfracture}$ represent the transmissivity of the grouted fracture. This method for estimating the residual transmissivity after grouting is similar to the one described by Gustafson et al. (2004) for the fractures in the rock mass. As described, it is the borehole interval transmissivities that are measured and described statistically not individual fractures. It follows that a reduction in transmissivity for each interval is a more approachable variable as a function of the grouting results compared to an analyses based on each fracture transmissivity, even though the rationale for such a reduction can be described according to the equations above. Such a reduction in interval transmissivity after grouting is illustrated for the lognormal distribution in Figure 6-4. In this case, a lower grouting limit or cut off is stipulated as $8.2*10^{-7}$ m^2/s, corresponding to the transmissivity of a single fracture of 0.1 mm (which is an engineering estimate of the transmissivity sum of all ungrouted, seeping fractures in the interval) via the 'cubic law', Snow (1965).

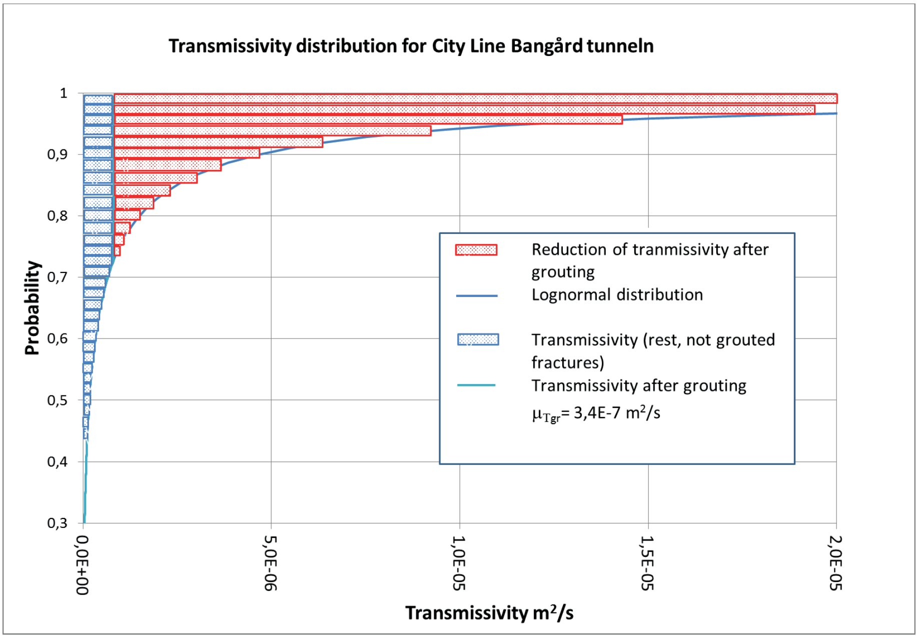

Figure 6-4. Interval transmissivity distribution and reduction in transmissivities after grouting. The mean transmissivity after grouting is denoted, □$_{Tgr}$. The cut-off is $8.2*10^{-7}$ m^2/s.

The lower grout limit or boundary representing the transmissivity after grouting will depend on a number of factors, such as the number of fractures, the fracture transmissivity distribution, the grout penetrability and the grouting process. The grout results may not give such a low mean transmissivity after grouting if the rock mass is highly fractured and conductive. For an investigation into fracture/borehole transmissivity, see the Hernqvist et al (2014) study, which shows, especially for measurements over larger intervals, that there are several fractures that significantly contribute to the transmissivity of the measured interval. Factors such as for example connected holes, limits to the grout mix capacity, grout leaking face or unexpected low pressures (due to jacking or other technical reasons) will influence the results. For Namntall South the end results was often influenced by such factors. It is therefore considered that the limit (or cut-off) would be higher than for the other cases. An engineering estimate could be that this limit would be 3 times to 8 times the cut-off transmissivity of the relatively tight rock mass in the other cases corresponding to a single fracture aperture of 0.15 to 0.2 mm evaluated according to the 'cubic law', Snow (1965). The grout limit (cut-off) depends on the transmissivity sum of the remaining ungrouted fractures, which is probably higher than the grout penetrability property, b_{crit}.

6.4 Analysis of water seepage

The presented approach to analysing limits to transmissivity and seepage through the grouted tunnel is a conceptually different approach to the more standard way of estimating the effect of grouting in the general equation. We can see that the factor K_{grout}/K_{eff} originates from the assumption that water flows through a homogeneous grouted zone, K_{grout}, outside which the rock mass is represented by the original mean hydraulic conductivity, K_{eff} or K_0. For this assumption the seepage can be calculated with the common equation:

$$q_{inj} = \frac{2\pi K_{grout} \cdot H}{\left(1 - \frac{K_{grout}}{K_{eff}}\right) ln\left(\frac{D+2t}{D}\right) + \frac{K_{grout}}{K_{eff}}\left[ln\left(\frac{4H}{D}\right) + \xi\right]}$$

By applying the assumption that a grouted fracture is sealed and that the remaining fractures in the interval constitute the remaining or residual transmissivity, as ungrouted fractures, a new mean hydraulic conductivity can be calculated, $K_{grout} = \mu T_{gr}/L$. Observe that the mean transmissivity must be divided with the length, L, for which the interval transmissivity was evaluated. Conceptually, the water flows through the ungrouted smaller fractures. This also means that there is a normal water pressure gradient driving the water flow compared to the general idea that the water pressure acts over (across) the grouted zone. For this assumption the grouted fractures have been removed from the distribution affecting both the mean transmissivity and the variation. In effect the resulting hydraulic conductivity of the rock mass, K_{eff} or K_0, becomes K_{grout}. The assumption that the fractures are ungrouted also implies that the expression can be simplified according to the equation for an ungrouted rock mass albeit with a different hydraulic conductivity:

$$q_{inj} = \frac{2\pi \mu_{Tgr}/L \cdot H}{\ln\left(\frac{2H}{r_t}\right) + \xi}$$

For both equations the H represent a constant groundwater pressure height which is a boundary condition for the solution see for example Gustafson and Alberts (1983).

6.5 Case comparison

The seepage has been calculated for the cases presented above. For Namntall North, the Bangård Tunnel and the Odenplan Station in predominantly constructed in rock of good quality, it was assumed that the level of achieved tightness after grouting can be represented by an interval transmissivity equal to the transmissivity of a hydraulic fracture width of 0.1 mm. This assumes that the actual grout penetrability is better than 0.1 mm and that the sum of the remaining fractures is not higher than the corresponding transmissivity value. The ground conditions for the Namntall South Tunnel with regard, for example, to zone and fracture density, were described by Stille and Gustafson (2010). It was shown that these conditions differed significantly from the northern part of the Namntall North Tunnel. Furthermore, it was shown that the grouting work encountered significant difficulties due to the adverse ground conditions but also due to the capacity of the grout mixer. It is reasonable to expect that the resulting transmissivity after grouting would differ from the grout result in rock types similar to the Namntall North and City Line tunnels. For this case the cut-off transmissivity was set to $2.8E^{-6}$ m^2/s. The rock mass with a transmissivity beneath these levels will be unaffected by grouting. The transmissivity after grouting, the 'residual transmissivity', indicates the effectiveness of grouting. In Table 6-1 the calculated residual transmissivity, K_{grout}, is presented together with the distance to the phreatic surface, H, K_0, and the calculated seepage using the normal and suggested modified method using the arithmetic mean. The seepage is calculated for the tunnel radius, r_t = 5 m, the thickness of the grouted zone, t = 5 m, and the assumed 'skin' factor, ξ = 5.

Table 6-1 Seepage calculations compared to measured data.

Case	H [m]	K_0 [m/s]	K_{grout} [m/s]	Calculated seepage standard eq. [l/min.100m]	Calculated seepage modified eq. [l/min.100m]	Measured seepage [l/min.100m]
Namntall South	50	$1.6*10^{-7}$	$7.0*10^{-8}$	34.0	16.4	10-25*
Namntall North	135	$5.0*10^{-8}$	$1.5*10^{-8}$	24.3	8.5	5-7*
City Line, Bangård Tunnel	27	$1.4*10^{-7}$	$1.7*10^{-8}$	10.7	2.2	1-3**
City Line, Odenplan Station	20	$2.3*10^{-7}$	$2.4*10^{-8}$	13.0	2.5	2***

*Stille and Gustafson 2010 **Tsuji et al. 2012 ***City Line, Odenplan Station project 2013

The results of the seepage calculation (modified equation) seem to correspond better with the measured seepage for the Namntall North and South Tunnels, the City Line Bangård Tunnel and the Odenplan Station.

6.6 Summary

The seepage calculation could be performed using a concept of adapting the interval transmissivity distribution of the rock mass in relation to the grouting result. Conceptually, the grouting result is based on the idea that grouted fractures are sealed and removed from the distribution. Partly grouted fractures are not considered to be grouted and water may flow through these fractures. The grouted transmissivity distribution can be calculated and since this part of the rock mass is considered ungrouted, the grouted arithmetic mean value can be used for the rock mass in the same way as for an ungrouted tunnel. The calculations in section 5.5 show better conformity with measured seepage values using this method. In hard crystalline rock of generally good quality, the lower grouting limit seems to be equivalent to $8.2*10^{-7}$ m2/s for a 20 m grout hole. It should be noted that this is a limit for the part of the holes that reveal higher transmissivity. The mean for the whole grouted distribution is lower when the whole population is considered. For poorer rock, the grouting result can be difficult to estimate and for these conditions it is appropriate to follow up the initial tunnelling operation with regard to water seepage into the tunnel (observational method).

7. Discussion and conclusion

7.1 Mean values and their use

The mean value of the rock mass hydraulic conductivity is used for early hydrogeological predictions of water seepage but also, for example, to estimate the influence areas of groundwater drawdown, possible settlements due to changes in the pore pressure distribution in clay layers and so forth. Individual major fracture/weakness zones are often modelled using a higher mean hydraulic conductivity to study the influence of more systematic variations in the rock mass. However, the spatial variations within such an individual zone are not described using the data presented herein even though water pressure tests performed for individual grout fans may represent the zone passing the tunnel alignment. If zones are modelled to identify the extent of the influence area and possible drawdown, great care should be taken when reducing the distribution data of the rock mass to identify new mean values for the rock mass in general. It should always be borne in mind that the rock mass is not its mean value. The mean value may be representative from one or more perspectives but there is considerable variation.

The mean values can be analysed and scaled according to the presented theory. The lognormal distribution fits the transmissivity data. It is important to take the accuracy of the measuring system into consideration when evaluating the data. When analysing the scale effects, an increase in scale reduces the variance and the geometric mean increases. For tunnels over 500 m in length, the difference between the geometric mean values and the arithmetic mean values seems to be relatively small. Considering the general trend in the statistical data, it seems reasonable that arithmetic mean values should be used for calculating seepage into most tunnels longer than 100-1,000 m. However, the requirement is that the data is from the same hydrogeological domain. The variation in data should be taken into account, especially for cases with a limited number of tests. By applying the statistical data, the variation can be used to describe a likely interval of mean hydraulic conductivity that can be used as an upper and lower boundary for calculating seepage.

Statistically, the probability that the rock mass effective hydraulic conductivity is smaller than the arithmetic mean value would be about 90% for a scale of 20 m (the approximate length of a grout hole) for a tunnel in the City Line Odenplan Station geological domain. The probability for an effective hydraulic conductivity value smaller than 2 times the arithmetic mean value is also about 90% and seems independent of the scale. This is a function of low probability to encounter the larger values in the distribution.

As can be seen, the probability that the hydraulic conductivity is accurately predicted is relatively low, 50-90% depending on which mean values are used. The geometric mean or the median value represents the 50% probability that a stochastic value for the rock mass is lower than the geometric mean. However, this is only true for the actual scale from which the geometric mean was taken. When looking at the 1,000 m scale from the City Line Odenplan Station data, the geometric mean for this scale will represent the 50%

probability of having this value for any 1,000 m tunnel in similar geological conditions. It should be noted that this value is five times as large as the geometric mean for a 20 m section. The probability of having such a low value for a longer tunnel is evidently small.

It seems reasonable from a design perspective that the boundaries of the mean hydraulic conductivity values are the arithmetic mean (upper boundary) and the geometric mean of the tunnel (lower boundary). In other words, it is probable that the rock mass hydraulic conductivity is smaller than the arithmetic mean value and from a design point of view it does not seem suitable to use a value lower than the geometric mean. It should be noted that the scale in this case is the tunnel or geological domain length.

For the grouting results, the expected mean hydraulic conductivity after grouting should be the arithmetic mean although the expected grouting result should be related to the anticipated geological conditions. The experience from projects in similar geologies will be very valuable in this respect when evaluating probable grouting results.

The general conclusions with regard to rock mass transmissivity are:

- The lognormal distribution is suitable for transmissivity data.
- The accuracy of the measuring system must be considered when evaluating the transmissivity data
- An increase in scale reduces variation and increases the geometric mean of the transmissivity
- The presented theory for the lognormal distribution can be used to calculate values as different scales.

7.2 The observational method as a design approach

The rock engineering problems described herein are related to the water flow into a tunnel or under a dam. The variation and the heterogeneity of the rock mass and the hydrogeological properties have been clearly shown, as well as the measuring and model uncertainties that are connected to the design. It is therefore not surprising to recommend the observational method for the design. In general, the observational method consists of three steps: prediction, observation and adaptation. The method requires a stringent approach and certain rules should be followed if it is used as a design method. The principle of the observational method was presented by Peck (1969) and was further discussed by Holmberg and Stille (2007). Sometimes the observational method is confused with a 'design as you go' attitude, but the method requires a much more structured and transparent way of working. It is necessary that the decision points are clear and that the observed behaviour is relevant for the interpretation/verification of the design prerequisites.

The basis for the method is to work with a design that’s defining probable behaviour and likely limits, including prepared mitigations, thus avoiding an overly conservative design. By structuring the problem, the relevant steps can be highlighted and the results can be presented transparently, as is the case in steps 0-7 below. Equally important is to observe the resulting behaviour and verify the assumptions in the design during the construction phase. The water pressure tests before (and sometimes after) grouting and the water seepage for a length of tunnel after grouting are two such measureable parameters that should be followed very closely during the initial construction phase since any changes to the design or grouting have a greater impact at an early stage. Although observations should be carried out throughout the project, the frequency can usually be reduced over time.

The general design procedure, preconstruction, is described sequentially to reach the objective of predicting water seepage and required sealing effect (sufficient level of grouting). As is usual for engineering problems, some updates may be required after going through an initial estimate before finalising the approach.

1. Definition of the engineering problem
2. Tests for rock mass interval transmissivity, T_i, over the tested length, L_i.
3. Calculation of statistical parameters for the transmissivity distribution, mean value, μ, and standard deviation, σ.
4. Define the objective scale length, L, of the analysis "20 – ≈1000 m": the lower limit corresponds to the length of the grout fan, whereas the upper limit of the scale length is the size of the geological domain or the tunnel length.
5. Rescale data from test length $T(L_i)$, to $T(L)$ and subsequently $K(L_i)$, to $K(L)$.
6. A. Estimate of mean hydraulic conductivity; upper and lower boundaries.
   B. Estimate of hydraulic conductivity after grouting.
7. A. Calculation of water seepage
   B. Calculation of water seepage after grouting
8. Decision of design approach taking into account the uncertainties in the tests, mean values and the grouting results, with regard to the objective scales (L), grout fan or tunnel (domain).
9. Follow up during construction, possible grout procedure adaptation with predefined measures or actions.

As can be seen, there are several choices that need to be made. These should be motivated and described in the design. In the spirit of the observational method, these choices should be shown to be representative of the geological domain and represent a probable outcome or property. An adjustment to the water pressure test results due to an increase in measuring/analysis error can be discussed, where the hypothesis is that the $\sum T_i$ (3 m) is larger than the $\sum T_i$ (20 m) and that the error is reduced with increased L.

7.3 The grouting process

The grouting process shows several interesting conditions that influence the idea of grouting result and performance. In hard crystalline rock the rock mass is often relatively tight and about 50 - 70% of the holes are grouted with a grout take (volume) of about the hole filling (grout hole volume). The stop criteria used for these holes is usually the 'grout flow criterion'. Theoretically, this indicates that about 50 - 70% of the rock mass around a tunnel is more or less considered to be ungrouted. On the other hand, since the porosity or the fracture void of the rock mass, again for hard crystalline rock, is very small the grout spread will for a relatively small amount of grout be quite large. For 100 l of grout in a physical aperture of 0.2 mm, the grouted area would be about 500 m2. The relationship between the physical aperture and the hydraulic aperture is also a very interesting topic that requires further study. Considering the experience from the City Line for example, where the physical aperture was shown to be about twice the hydraulic aperture. Since the water seepage is related to the hydraulic aperture it seems reasonable that the grout result is related to the hydraulic aperture. It also seems reasonable that the penetrability of the grout and therefore the grouting result should be related to the physical aperture. For the individual fractures the hydraulic aperture would be smaller than the physical aperture. This indicates that the expected size of the smallest grouted fracture could be about half its corresponding hydraulic fracture aperture.

Andersson and Stille (2007) and Bruno (2009) showed that one of the more detrimental issues for the grouting result is the connected holes. It was clearly shown by Bruno that the sequential grouting procedure with long grouting times produced smaller and smaller grout takes. It was concluded that this was related to changes in grout properties where non-flowing grout 'settles' within 2 h. It is clear that grouting of holes should be completed within about 15 minutes and that connected holes, if possible, should be grouted within 45 minutes for this type of grout. The connected holes aspect indicates that the smaller fractures (and possibly some larger ones) may not be considered to be grouted. This is especially true for a more fractured rock mass where there may be a need for a second grouting round.

Each grout hole crosses a number of fractures, some water-bearing and some not. The grout can only penetrate fractures of a certain size and there will always be fractures that remain more or less ungrouted. The fractures that are not grouted will be part of the 'residual' transmissivity after grouting. The sum of the residual transmissivity will be a function of the smallest grouted fracture.

Examples from the Namntall Tunnel and the Northern Link show that elastic jacking frequently occurs and that this may, in some cases, influence the final seepage or grouting result. It might be possible, by following the grouting minute by minute, to predict the course of the grout flow and also analyse the risk of uplift and jacking. This could be of particular interest for shallow surface grouting or grouting close to other subsurface areas where the possibility of detecting jacking or uplift of the rock mass could be of critical interest. The most significant action from the presented examples would have been the ability to abort grouting or lower the grouting pressure.

Another condition that influences the grouting performance is the grout mixing capacity, which was observed during the excavation of the Namntall Tunnel in 2004-2006. It was observed that in reality the grout mix capacity was about 20 l/min for each of the two pumps and that the design grouting pressures often could not be reached for this reason.

Lower grouting pressures naturally result in shorter grout penetration lengths or longer grouting times, which is especially poor for conditions with many connected holes.

Grouting is to a large degree a special skill and experience is required to handle the rig decisions rationally. In the report 'Field tests with multi-hole grouting ' by Stille et al 2014 it was clearly shown that this was the case. Recurring issues, such as poor planning and poor communication, were often the case when inexperienced personnel handled the equipment compared to the relatively efficient grouting procedure with experienced personnel.

The case studies show that it is generally easier to achieve a better grouting result in good quality rock mass compared to poor rock. Furthermore, there are practical aspects with regard to the grouting process that need to be considered, such as equipment types and capacities, temperature, organisational competencies and contractual arrangements, which may influence the grouting result.

The general conclusions with regards to the grouting process are

- The grouted fracture aperture will be about twice the hydraulic aperture.
- For the presented cases in generally good quality crystalline hard rock, about 50 - 70% of the rock mass can be considered ungrouted from a water seepage calculation perspective.
- Connected holes influence the grouting performance, resulting in areas that are not grouted or only partly grouted. After the third or fourth connected hole that is sequentially grouted, the holes are almost lost (small grout take, short grouting time and a resulting short fracture penetration).
- Mixing capacities may limit the grouting performance, especially in areas with higher seepage and with many connected holes.
- Elastic (and ultimate) jacking should be avoided.
- Two (or more) grout fans may be required to seal more seeping areas.
- For most grouting fans, one person at the face can only handle two hoses effectively. For more seeping areas, up to four hoses can be handled with grouting times of around 15-20 minutes per hole.
- An efficient grouting procedure requires experienced personnel and may limit the problems related to connected holes and grouting capacity problems.

8. References

Alberts C. Gustafson G., 1983: Undermarksbyggande i svagt berg: *4 Vattenproblem och tätningsåtgärder*. BeFo 106. Stiftelsen Bergteknisk Forskning.

Axelsson M. Eriksson M. Wilén P., 2007: *Dimensionering av typinjektering – koncept 1*. design document 9563-13-025-001 for Citybanan in Swedish.

Barton N., 2002. *Some new Q-value correlation to assist in site characterisation and tunnel design.* International Journal of Rock mechanics & Mining Sciences, Vol. 39, pp 185-216.

Bieniawski Z.T., 1989: *Engineering rock mass classifications*. Wiley, New York.

Bohlin M. Urtel K., 2008: *Utvärdering av injekteringsutförande – Fallstudie av NL101 och Hornsberg*. Stockholm, Master Thesis, Division of Soil and Rock Mechanics, Royal Institute of Technology.

Botniabanan AB, 2003. Adapted from the tender documents rev.E, project E3541 Offersjön-Bjällstaån.

Brantberger M. Stille H. Eriksson M., 2000: Controlling the grout spread in tunnel grouting – Analyses and developments of the GIN-method. Tunneling and Underground Space Technology, Vol 15, No. 4, pp 343-352.

Brantberger M., 2000: Metodik vid förinjektering i uppsprucket hårt berg. Licentiate Thesis, Division of Soil and Rock Mechanics, Royal Institute of Technology Stockholm.

Bruno A., 2009: Grouting operation monitoring and analysis of the "Real-Time Grouting Control" method. Master Thesis at KTH and Politechnico di Torino.

Caine J.S. Evans J.P. Forster C.B., 1996: Fault zone architecture and permeability structure. Geology, No. 11, pp 1025-1028.

De Marsily G., 1986: Quantitative Hydrogeology: Ground Water Hydrology for Engineers. Academic Press. Inc. San Diego, USA.

Dalmalm T., 2004: Choice of grouting method for jointed hard rock based on sealing time predictions. Ph.D. thesis, Royal Institute of Technology, Stockholm.

Draganovic A., 2009: Bleeding and filtration of cement-based grout. PhD Thesis, Division of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm.

Eriksson M. Stille H. Andersson J., 2000: Numerical calculations for prediction of grout spread with account for filtration and varying aperture. Tunnelling and Underground Space Technology, Vol 15, No. 4, pp 353-364.

Eriksson M. Stille H., 2005: Cementinjektering i hårt berg (Grouting with cement based grout in hard jointed rock). SveBeFo, Stockholm, Sweden.

Fransson Å., 2001: Characterisation of fractured rock for grouting using hydrogeological methods. PhD thesis, Chalmers University of Technology, Department of Geology, Gothenburg.

Funehag J. Gustafson G., 2004: Injekteringsförsök med Cembinder U22 i Hallandsås. Publication 2004:1, Division of GeoEngineering, Chalmers University of Technology, Gothenburg.

Funehag J., 2007: Grouting of fractured rock with silica sol — Grouting design based on penetration length. Ph.D. thesis, Chalmers University of Technology, Division of GeoEngineering. Gothenburg.

Gothäll R. Stille H., 2009. Fracture dilation during grouting. Tunnelling and Underground Space Technology, Vol. 24, pp 126-135.

Gustafson G. Stille H., 1996. Prediction of groutability from grout properties and hydro geological data. Tunnelling and Underground Space Technology 11(3), pp 325-332.

Gustafson G. Franson Å. Funehag J. Axelsson M., 2004: Ett nytt angreppssätt for berg-beskrivning och analysprocess för injektering. (in Swedish). Väg- och Vattenbyggaren 4, 2004.

Gustafson G. Claesson J. Fransson Å., 2013: Steering Parameters for Rock Grouting. Journal of Applied Mathematics, vol. 2013 pp. article ID 269594.

Gustafson G. Stille H., 2005: Stop criteria for cement grouting. Felsbau 23(3), pp 62-68.

Gustafson G., 2009: Hydrogeologi för bergbyggare. in Swedish, Formas, ISBN 978-91-540-6029-0.

Hakami E., 1995: Aperture distribution of rock fractures. Doctoral Thesis, Division of Engineering Geology Department of Civil and Environmental Engineering, Royal Institute of Technology.

Hernqvist L. Fransson Å. Gustafson G. Emmelin A. Eriksson M. Stille H., 2008. Analyses of the grouting results for a section of the APSE tunnel at Äspö Hard Rock Laboratory. Journal of Rock Mechanics and Mining Sciences.

Hernqvist L., 2011: Tunnel Grouting: Engineering Methods for Characterization of Fracture Systems in Hard Rock and Implications for Tunnel Inflow. Institutionen för bygg- och miljöteknik, Chalmers University of Technology, Göteborg.

Hernqvist L. Einarsson V. Höglund A., 2014: Does one fracture dominate the borehole transmissivity? Conference paper, Swedish Rock Engineering Research foundation, BeFo.

Holmberg M. Stille H., 2007: Observationsmetodens grunder och dess tillämpning på design av konstruktioner i berg. (in Swedish). SveBeFo Report 80.

Holmén J., 1997: On the flow of groundwater in closed tunnels – generic hydrogeological modelling of nuclear waste repository, SFL 3-5. Doctoral thesis, Uppsala University.

Houlsby A.C., 1990: Construction and design of cement grouting. ISBN 0-471-51629-5, John Wiley & Sons, Inc., USA.

Håkansson U., 1993: Rheology of fresh cement-based grout. Ph.D. thesis, Royal Institute of Technology (KTH), Division of Soil and Rock Mechanics, Stockholm.

Hässler L. Stille H. Håkansson U., 1988: Simulation of grouting in jointed rock. Proc. 6th International Congress on Rock Mechanics, Vol. 2, pp 943-946, Montreal.

Hässler L., 1991: Grouting of rock –Simulation and classification. PhD Thesis, Division of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm.

Hässler L., 2007: Personal communication, Golder Associates AB.

Kobayashi S. Stille H., 2007. Design for rock grouting based on analysis of grout penetration. Verification using Äspö HRL data and parameter analysis. R-07-13, Swedish Nuclear Fuel and Waste Management Company, Stockholm.

Kobayashi S. Stille H. Gustafson G. Stille B., 2008: Real- Time grouting Control Method, Development and application using Äspö HRL data. R-08-133, Swedish Nuclear Fuel and Waste Management Company, Stockholm.

Lombardi G. Deere D., 1993: Grouting design and control using the GIN principle. Water Power and Dam Construction, June 1993, pp 15-22.

Lindström L., 2007: Site geologist, Personal communication. Vattenfall Power Consultants AB.

Lundqvist T. Gee D. Kumpalainen R. Karis L. Kresten P., 1990: Beskrivning till berggrundskartan över Västernorrlands län. pp 18 – 44, pp. 322 – 327.

Morfeldt C-O., 1979: Är injektering i berg en svartkonst? Bergmekanikdagen 1979, BeFo, Stockholm.

Moye D.G., 1967: Diamond Drilling for Foundation Exploration. Civil Engineering Transactions, April, pp 95-100.

Munier R. Stenberg L. Stanfors R. Milnes A.G. Hermanson J. Triumf C-A., 2003: Geological Site Descriptive Model. A strategy for the model development during site investigations. R-03-07. Swedish Nuclear Fuel and Waste Management Company, Stockholm.

Olsson R., 1998: Mechanical and Hydromechanical Behaviour of Hard Rock joints, A laboratory study. Ph. D. Thesis, Chalmers University of Technology. Göteborg.

Peck R.B., 1969: Advantages and limitations of the observational method in applied soil mechanics: 9th Rankine lecture. Géotechnique, 19 (2), pp. 171-187.

Rafi J.Y., 2014: Study of pumping pressure and stop criteria in grouting of rock fractures. PhD Thesis, KTH Royal Institute of Technology, Stockholm.

Rahman M., 2015: Rheology of cement grout: Ultrasound based in-line measurement technique and grouting design parameters. PhD Thesis, KTH Royal Institute of Technology, Stockholm.

Rhén I. Bäckblom G. Gustafson G. Stanfors R. Wikberg P., 1997: Äspö HRL - Geoscientific evaluation 1997/2. Results from pre-investigations and detailed site characterization. Summary report. SKB TR-97-03, Svensk Kärnbränslehantering AB.

Shamu J., 2021: Rock grouting design: Rheological aspects and radial flow visualizations with ultrasound. Ph.D Thesis, KTH Royal Institute of Technology, Stockholm.

Snow U. T., 1965: A Parallel Plate Model of Fractured Permeable Media. Ph.D. thesis, University of California, Berkeley.

Stille B, Anderson F., 2008: Injektering – tillämpning av injekteringsforskning i fält. Pre-grouting – application of grouting research in the field. SveBeFo rapport 79, Stockholm.

Stille B. Stille H. Gustafson G. Kobayashi S., 2009: Experience with the real-time grouting control method. Geomechanics and Tunneling. Vol 2, pp.447-459.

Stille B. Gustafson G., 2010: A review of the Namntall Tunnel project with regard to grouting performance. Tunnelling and Underground Space Technology. Vol 25, pp. 346-356.

Stille B. Batres R. Forslund C., 2014: Fältförsök med multihålsinjektering, Field tests with multi-hole grouting, in Swedish. BeFo rapport 140, Stockholm.

Stille B., 2016: Grouting theory and grouting practice. Lic. Thesis, Chalmers University of Technology. Göteborg.

Stille H., 2015. Rock Grouting – Theories and Applications. BeFo, Stockholm.

Tsuji M. Holmberg M. Stille B. Rafi J Y. Stille H., 2012: Optimization of the grouting procedure with RTGC method. Data from a trial grouting at city line project in Stockholm. SKB R-12-16, Svensk Kärnbränslehantering AB.

Thörn J., 2015: The Impact of Fracture Geometry on the Hydromechanical Behaviour of Crystalline Rock. Ph. D. Thesis, Chalmers University of Technology. Göteborg.

Zetterlund M., 2014: Value of Information Analysis in Rock Engineering Investigations. Institutionen för bygg- och miljöteknik, Chalmers University of Technology, Göteborg ISBN: 978-91-7597-023-3.

Zhang S., 2021: Design of grout curtains under dams founded on rock.
Lic.Thesis, KTH Royal Institute of Technology, Stockholm.

BeFo

Box 5501
SE-114 85 Stockholm

info@befoonline.org • www.befoonline.org
Visiting address: Storgatan 19, Stockholm

ISSN 1104-1773