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Choice of Grouting Method for Jointed Hard Rock based on Sealing Time Predictions

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Stockholm, May 2004
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Sammanfattning

Föreliggande avhandling behandlar frågeställningar relaterade till val av injekteringsmetod i uppsprucket hårt berg. Genom att beräkna den totala tätningstiden för att uppnå kravnivån, kan injekteringsmetoderna jämföras och den mest fördelaktiga metoden väljas.

En metodik har utarbetats för att beräkna den totala tätningstiden i olika bergmassor och med olika injekteringsmetoder. Fem huvudaktiviteter ingår i metodiken; borrning, injektering, ställtid, undersökningar och ominjektering som i olika grad bidrar till den totala tätningstiden. Därtill finns en extra aktivitet, efterinjektering, som inom detta arbete behandlas skilt från övriga aktiviteter, då tiden för efterinjektering vanligtvis inte skall räknas med i den totala tätningstiden. I de olika aktiviteterna har ämnesområden som bergmassans egenskaper, undersökningar av bergmassan, injekteringsteknik, injekteringsbruk och olika ställ- och väntetider studerats.

Att tillämpa metodiken innebär att för en injekteringsskärm få fram tidsåtgångens medelvärde och varians för varje aktivitet. Tidsåtgången för de olika aktiviteterna summeras för att få fram den totala tätningstiden för hela injekteringsskärmen. Därefter summeras tiden för injekteringsskärmarna för att få fram den totala tätningstiden för hela tunneln.

För t ex injekteringsaktiviteten har sprickor tillhörande en injekteringsskärm slumpats ur en sprickviddsfördelning för den aktuella bergmassan. För varje spricka i denna skärm beräknas sedan fördelningar av parametrar för inflöden, tider och injekteringsvolymer t ex tätningstid och uppnådd tätningseffekt. Därefter utförs en Monte Carlo simulering baserad på dessa fördelningar och för varje injekteringsskärm beräknas medelvärde samt varians.

För att testa metodiken har numeriska beräkningar i tre olika bergmassor och för tre olika injekteringsmetoder utförts. Den mest fördelaktiga injekteringsmetoden definieras som den metod som kan uppfylla kravnivån på kortast totala tätningstid.

Genom analys av de numeriska beräkningarna har det konstaterats att det många gånger inte är injekteringstiden som är avgörande för den totala tätningstiden, utan att även andra aktiviteter har stor inverkan på den totala tätningstiden. Att välja mellan förinjektering och efterinjektering är ett beslutsproblem, som är starkt kopplat till övriga aktiviteter för tunneldrivningen. Möjligheten att täta har visat sig variera starkt med vald injekteringsmetod och valt injekteringsbruk. En lång injekteringstid behöver inte betyda att injekteringsresultatet är bra, det motsatta har genom beräkningar även visat sig vara möjligt.

Vidare har beräkningar visat att avgörande för injekteringsresultatet är en kombination av hålavstånd mellan injekteringshålen, inträngningslängd för bruket och rätt pumptid. Dessutom har det demonstrerats hur denna kombination skulle kunna väljas för att optimera injekteringen. Ett beslut avseende lämplig injekteringsmetod måste dock alltid fattas med hänsyn till övriga aktiviteter för tunneldrivningen.

Summary

This thesis concerns subjects related to the choice of a grouting method in a jointed hard rock mass. By calculating the total sealing time to reach the requested sealing level, different grouting methods could be compared and the most favourable chosen.

A methodology, to calculate the sealing time for different grouting methods in different rock masses has been developed. Five main activities are included in the methodology; drilling, grouting, waiting, probing and re-grouting, which to differing degrees contribute to the total sealing time. In addition, an extra activity, post-grouting, is regarded separately, as the post-grouting time normally is not included in the total sealing time. Within the different activities, subjects such as rock mass properties, examination of rock mass, grouting technique, grout mix and the different waiting times have been studied.

In practice, the application of the methodology requires that the average time and variance for each activity on a fan level is expressed. The times for each activity are then added together to achieve the total time for a fan. After which, the time for each fan is added over the entire tunnel length to calculate the total sealing time for a tunnel.

E.g. for the grouting activity, joints belonging to a grouting fan are randomly selected from a joint aperture distribution for the appropriate rock mass. For each joint in a fan, distributions are calculated for parameters such as inflow, time and grouting volumes as e.g. sealing time and sealing effect. Then a Monte Carlo simulation based on these distributions and for each grouting fan is carried out and the average grouting time and variance calculated.

To test the methodology, numerical calculations for three grouting methods and in three rock masses have been carried out. The most favourable grouting method is defined as the method which fulfils the requirement in the shortest sealing time.

By analysing the result of the numerical calculations, it has been shown that other activities besides grouting often have a large impact on the total sealing time. To choose between pre-grouting and post-grouting is regarded as a decision problem, which is strongly related to other activities of the tunnel production cycle. A strong relationship was shown between the possibility to seal and the chosen grouting method and grout mix. A long grouting time is not always equivalent to a good sealing result and the opposite has shown to be possible.

Further, calculations have shown that a correct combination of hole spacing, grout penetration length and appropriate pumping time is essential for a good sealing result. In addition, this thesis has demonstrated a method as to how this correct combination should be chosen in order to achieve an optimal grouting solution. However, a decision regarding an appropriate grouting method, always needs to regard other activities of the tunnel production cycle.

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At the end the reader should notice that the section of “Glossary and Symbols” have been excluded from this thesis as the symbols are locally explained for each equation presented throughout the thesis.

Chapter One

1 Introduction

1.1 Background

The requirements for water sealing of underground facilities have constantly been increased during recent years, due to more sensitive location and increased knowledge. An underground facility with too high water inflows can result in severe consequences for the surrounding environment and constructions, while appropriate inflows can be the result of extensive- or well optimized sealing methods.

As surface space become more and more occupied, other locations will be considered more often as an alternative. The number of activities suitable for underground location has, as a result of more occupied surface space and improved sealing methods, increased during recent years.

An underground facility can be sealed against water leakage through, for example, a water tight lining or through the host formation, that the facility is built in, being condensed so that the hydraulic conductivity is reduced.

To prevent water leakage and to fulfil the inflow requirements, cement based grouting has, for Swedish conditions, shown to be both practical and economical. Grouting has therefore become an integrated part of the tunnel excavation cycle in order to handle the increased sealing requirements.

In Sweden, grouting is often used as a permanent sealing solution, which differs from many other countries, where casted lining systems are more often used for permanent support and consequently for sealing. Grouting is more commonly used as a temporary solution during the excavation process. For sealing purposes, the reason for this could be because the Nordic countries have quite good quality rock mass and casted concrete as support measures are seldom required for stable conditions.

The inflow requirement is often expressed as a maximum allowable amount of water ingress to the tunnel. In Sweden today a typical value is between 0.5 and 10 litres/min/100 meters of tunnel. The increased sealing requirements result in the grouting process demands a larger part of the total production cycle. As a result, the cost for grouting has increased substantially during recent years. One dilemma is that the cost for grouting is not always proportional to the grouting result. Another dilemma is that the grouting result is often difficult to measure.

The high demands of low water ingress are based on the geo-hydrological pre-investigations, which commonly have been carried out during the design phase. Based on these data, limits for maximum allowable inflow of water ingress to the facility are calculated. The correlation between these limits and the required grouting efforts are very uncertain and highlights that more research is required. In the light of today’s high requirements of low water ingress, and experiences from some difficult grouting projects, the need to clarify different aspects of the sealing effect has increased.

The knowledge how to reach the water ingress requirements for underground facilities in an effective way is still in a developmental phase. If different grouting methods could be evaluated regarding the efforts and the result for different rock mass situations, it would be possible to compare them, which would enable a more effective use.

The cost saving could be considerable with a more effective and controlled grouting process. The grouting process needs to be developed into a process, which continuously and on short notice can be adjusted to current rock mass conditions. To organise registered grout data, geo-hydrological pre-investigations and geological surveying into a model could be a base for a grouting prediction system, which forecast the required grouting effort for different situations.

The theories for grouting can be divided into two major topics; first the knowledge of the rock mass condition, its hydraulic properties and secondly the behaviour of the grout. Earlier studies of grouting theories describe some basic principles for example Cambefort (1964), Wallner (1976), Cambefort (1977), Lombardi (1985). The studied relationship between grout-take and hydraulic properties of the rock mass was considered weak by, for example, Ewert (1992). By using the GIN-principle for grouting performance, the rock mass properties may be studied indirectly during grouting hence giving a possible way of controlling the grouting process (Lombardi & Deere, 1993). The pressure drop velocity was studied as a break-off criterion to stop grouting by Zettler et al (1997). The grout flow has also been described by numerical calculations made, for example, by Wallner (1976).

Research at the division of Soil and Rock Mechanics at the Royal Institute of Technology in Stockholm, has so far resulted in four PhD theses. Hässler (1991), studied the flow of grout and how to simulate the grouting process using numerical calculations. Håkansson (1993), focused on the behaviour of fresh cement paste and how to measure the rheological behaviour. Janson (1998), proposed models for grout take based on grouting technique, properties of the grout and geological characteristics. Eriksson (2002) simulated grout spread and sealing effect in a numerical model with different grout- and joint properties. Eriksson (2002) also studied the filtration and separation effect of the grout and their influence on spreading and sealing of different geometry's. Seven licentiate theses have also been presented; one of them was Brantberger (2000), who studied different demands and conditions that influence the choice of grouting method and methods for controlling the grouting process. In 2001, Dalmalm presented models to adjust the calculated grout volume due to different restricting situations. Dalmalm (2001) also suggested that the grouting time could be used as a parameter, which is followed up in this thesis. In 2003 Eklund presented a licentiate thesis concerning the properties of cement suspensions, which will be followed up by a PhD thesis on the same subject.

Still several issues concerning grouting technique and theory are unexplained and further research is ongoing developing the calculation models of grouting and the measuring technique of grout mix properties.

At Chalmers University of Technology research has resulted in three PhD theses. Andersson (1998) studied grouting with polyurethane foams, Fransson (2001) studied the characterisation of rock for grouting purposes and Swedenborg (2001) studied the change of rock mass properties due to grouting. Further research is also continued at Chalmers, studying the use of silica soil grouts and the grout resistance and the erosion of the grouts. At the Cement and Concrete Institute (CBI) studies concerning the properties of the grout mix have been studied for many years and are being continued.

1.2 Objectives and disposition of work

The objective of this study is to optimize the grouting process by develop, describe and propose a system for grouting method evaluation, which predicts the sealing time for fulfilled requirements.

Through the proposed system the grouting process could be optimised based on various information and adjusted for changing rock mass conditions, which makes the system a part of an active design concept for sealing of tunnels.

This thesis is disposed in such way that general aspects on grouting of tunnels and requirements, together with a number of grouting methods are briefly described and compared in chapter 2. Chapter 2 is then summarized in a sealing time component chart, which then is the base for a literature review presented in chapter 3. Chapter 3 covers general issues related to the rock mass, inflow calculations, grouting technique, grout mix and prediction of grouting, with focus on issues related to sealing time.

In chapter 4 a system for evaluating different grouting methods and to calculate the sealing time is described. The grouting method evaluation system presented here is more general than that presented previously in literature. Some of the theories have been presented before, but they have not been combined into a system as in chapter 4.

Three different hypothetical rock masses and three different grouting methods are then evaluated and presented in chapter 5. The conclusions from the evaluation are presented in chapter 6 and the sealing time for the different methods are calculated and presented in chapter 7. In chapter 8, some concluding remarks are made and some suggestions for further research are presented.

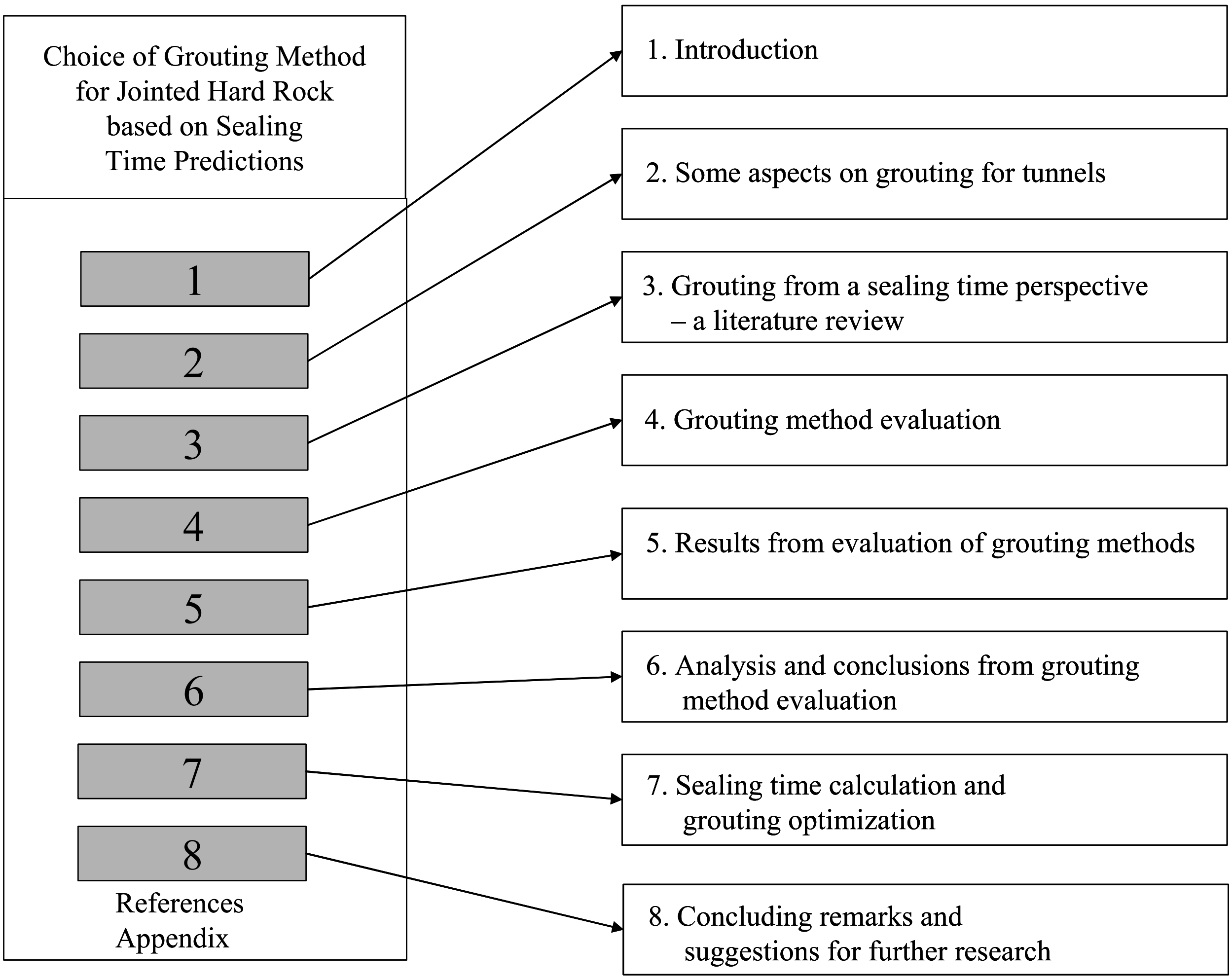


Figure 1-1 Disposition of thesis.

1.3 Limitations

Only grouting with cement based grouts in jointed hard rock masses are considered. The comparisons made are based on limited material; other results may therefore be possible if the base of comparisons should be extended.

The models proposed are not yet fully verified with practical grouting data.

The required inflow reduction to a tunnel or a facility can be achieved by grouting, lining or drainage, whereof only grouting is regarded here.

Chapter Two

2 Some aspects on grouting for tunnels

2.1 General

Grouting for water control, could in general be described as a process of injecting fluids that set into cracks or voids of a rock mass. Grouting has long since been considered as the most cost effective system for reducing the ingress of water to tunnels in jointed hard rock. Grouting in jointed hard rock is further recognized as filling out the joint system, while the solid rock mass is considered impenetrable. The joint system may, dependent of an external inflow requirement, need to be sealed to different degrees.

Throughout this thesis the term *hard rock* is used to described a rock mass, which has intact compressive rock strength of above approximately 100 MPa and corresponds to the term Very strong or Extremely strong (Hoek et. al., 1996). The specimen requires many blows of a geological hammer to get fractured. Some examples are Basalt, Diabase, Gneiss, Granite, Quartzite, Amphibolite, Gabbro, Granodiorite, Limestone, or Marble. In addition the rock mass contains a number of joint systems and other weakness zones. The joints and weakness zones heritages from litho static, tectonic, thermal forces and high water pressures (NRC, 1996).

When grouting a tunnel the grout is usually distributed to the joint system through a number of holes drilled ahead of the excavation with a fan layout. The most common fluid used for rock grouting is a mix of cement and water, which is pressed, at high pressure, into the joint system by a pump. The grout could, for some situations, be a suitable chemical solution.

Grouting could be performed as pre- or post grouting, were pre-grouting is carried out ahead of the tunnel excavation and post-grouting after excavation. Normally grouting is performed as pre-grouting, with, if necessary, complementary post-grouting and/or drainage. The pre-grouting is to be preferred from a sealing point of view, due to the possibility to use higher grout pressures, with extended grout spread and penetration as a result. The post-grouting should normally, if possible, be limited due to higher cost and poorer results compared to the pre-grouting.

If the purpose is to *optimise* the grouting process, it is necessary to be able to predict the outcome of different grouting methods. In a practical grouting perspective, to optimise could be to evaluate and select a cost effective system for sealing. The cost is mainly dependent of the sealing time. The grouting prediction system will therefore, preferably, be set up to predict the sealing time. To *predict* is then referred to as a process that in advance of an occurrence, with limited information and with a certain degree of accuracy, among a predefined number of methods, can suggest the grouting method which gives the shortest sealing time and fulfils the requirements.

The term *Grouting Prediction System* is in this thesis used to describe a system that evaluates and separates different grouting methods based on sealing times.

A *system* or grouting method can, based on the general definition of the system, be defined as: "System: Composite entity, at any level of complexity, of personnel, producers, materials, tools, equipment, facilities and software. The activities of this composite entity are used in the intended operational or support environment to perform a given task or achieve a specific objective (IEC 300-3-9)".

The term *Grouting Philosophy* is, in this thesis defined, as a background idea which is used to form a Grouting Method.

The term *Grouting Method* is, in this thesis, used for to describe the transformation of the grouting philosophy to a practical application.

The term *Grouting Process* is a synonym to Grouting Method, but is more general.

The degree of difficulty regarding grouting works is dependent on, among other things, the initial hydraulic conductivity of the rock mass and the inflow requirements. Therefore high requirements do not necessarily require extensive sealing if, for example, the initial rock mass conductivity is low. The relationship between initial inflow, sealing requirements and the difficulty of grouting is rarely discussed in literature. One example showing the relationship was discussed by Bergman & Nord (1982), and is shown in Figure 2-1. “Large inflow prior to grouting” in combination with “heavy demands on tightness” gives a “difficult degree of difficulty in grouting”. The “degree of difficulty in grouting” is related to the time to seal a tunnel (sealing time), which will be discussed further in this thesis.

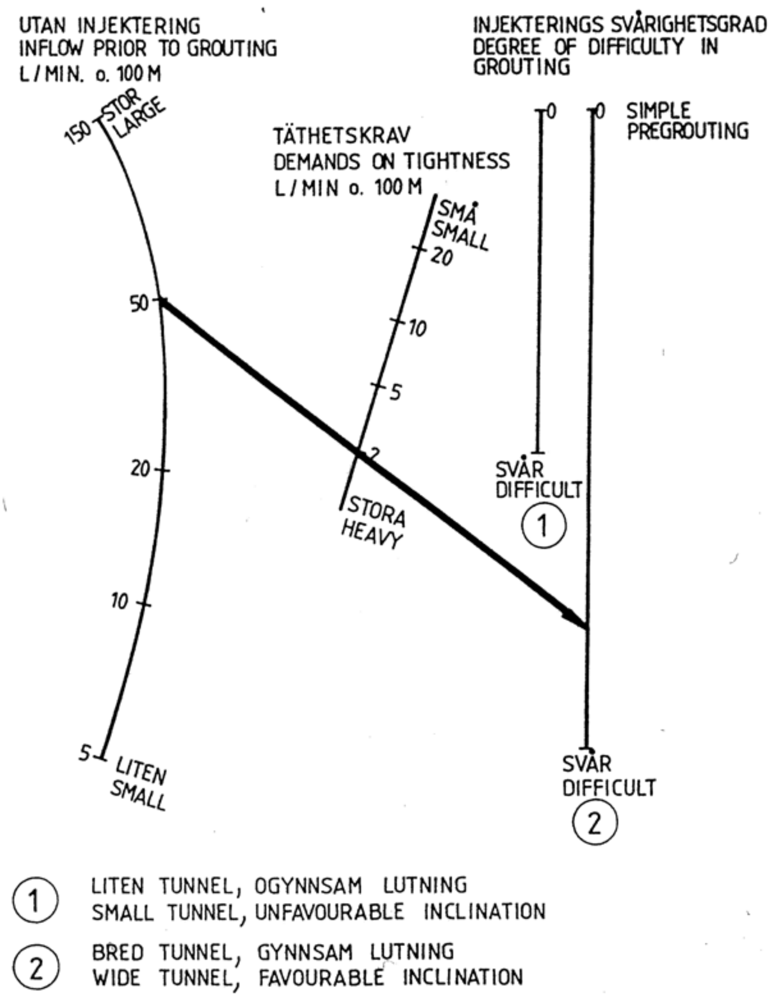


Figure 2-1 Principal relationship between inflow prior to grouting, sealing requirements and sealing efforts (Bergman & Nord, 1982).

2.2 Requirements

For road- and rail tunnels as well as for other underground facilities, normally a requirement for maximum permitted inflow of water is given. The inflow requirement is set up to prevent lowering of the ground water table, to prevent damage on installations and to guarantee a high health and safety level.

The inflow requirement for a tunnel is based on three main types of requirements as shown by Lindblom (1999) in Figure 2-2.

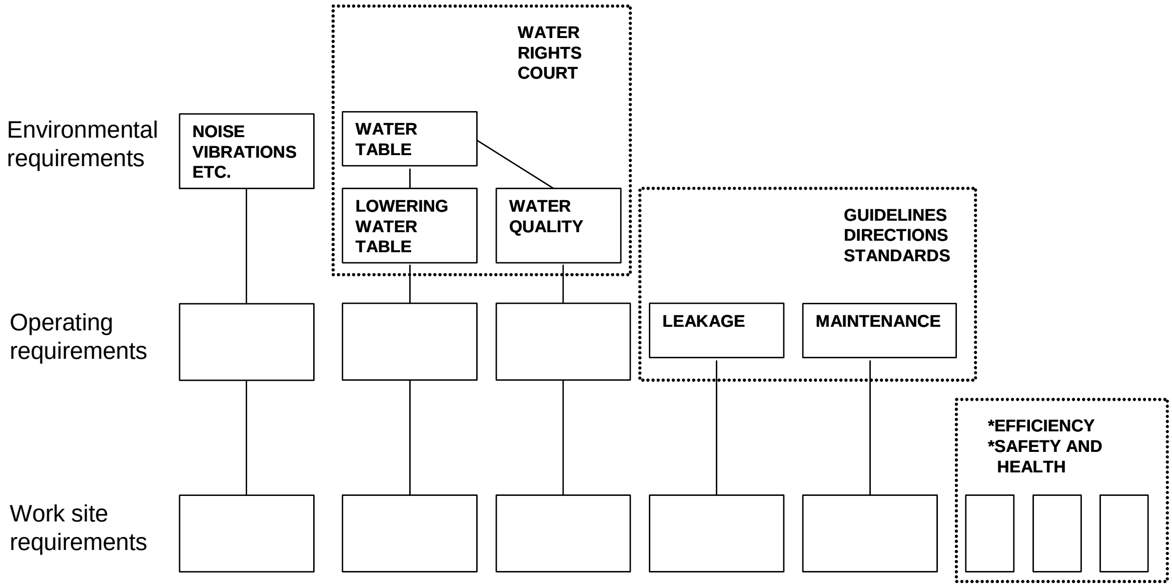


Figure 2-2 Structure of requirements for tunnel grouting (Lindblom, 1999).

The three main types of requirement regarding grouting works for underground facilities has, for example, been summarized by Andersson & Sellner, (2000) and Albertsson & Sjöholm (1999) as:

The community regulates <u>environmental requirements</u> on sealing- and grouting work, with the purpose to secure the surrounding environment from unacceptable lowering of the ground water table and unwanted spreading of contamination to the environment.

The client sets up <u>operating requirement</u> of inflow for the facility. These requirements are set in order to meet the security demands for users and vehicles, to ensure necessary functioning of the fittings and installations and also in order to give the ability for a rational maintenance.

The contractor, who will construct the facility, sets up <u>work site requirements</u>, which are related to staff working environment and staff safety. In addition the contractor normally has requests on the grouting work process as it is related to the cost of the project.

The community's requirements are normally based on investigations of the geo-hydrological conditions. The requirements are then often expressed as a maximum allowed amount of water in-leakage for a facility in litres per second or for a tunnel in litre per minute and 100 meter of tunnel. Normal in-leakage values for tunnels are today between 0.5 and 10 litres/minute and 100 meters.

The requirement can also be translated to a requirement on hydraulic conductivity of the rock mass K_g and an extension of the grouted zone I_g, see further chapter 3.

The Swedish Road Administration and the Swedish National Rail Administration are two of the main clients for tunnels in Sweden. They have, therefore, compiled guidelines for grouting work in tunnels.

The Swedish Road Administrations requirements on water tightness, grout mix and grouting works can be summarized in accordance to Tunnel 99 (Freiholtz, 1999):

"The investigation shall verify or by other means show that it is probable that the chosen concept for sealing / drainage fulfils the requirement of allowed water in-leakage to the tunnel. Harmful influence on the surroundings from a tunnel shall primarily be handled by grouting and secondly by infiltration or by a water tight construction. The aimed environment in the tunnel shall primarily be reached by grouting and secondly by drainage. Drainage of water and infiltration are methods, which increase the operation cost and should therefore be preceded by analysis of life cycle costs. Grouting of a rock mass should firstly be by cement grouting. For some occasions chemical grouting may be necessary. The grout mix shall for each grouting be adapted for the current situation and thereby have proper fluid properties, proper filtration stability and be chemically and physically long-term resistant. A sulphur resistant grout mix shall be used if there is risk for contact to sulphates in the surrounding environment."

The allowed inflow to a tunnel is dependent on a number of parameters as for example: precipitation, size of water basin, in- and outflows from basin.

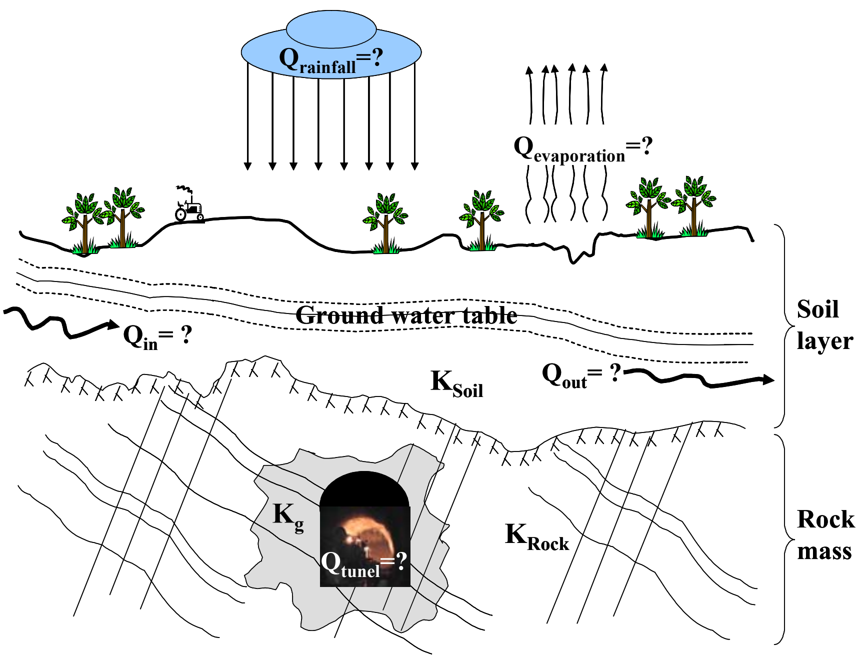


Figure 2-3 In- and outflows, which affects the allowed inflow to the tunnel.

The different parameters are both location and time dependent, since for example the rainfall varies over the year, during the winter the rain may be stored as snow and the evaporation is higher during the summer.

The in- and outflows in Figure 2-3 results in a balance for the soil layer. If a tunnel is then excavated and an inflow to the tunnel occurs, the inflow may, depending whether there is excess water in the soil layer or not, change the initial water table. If for example the precipitation generates excess water, which was earlier transported out from the basin and is now transported through the tunnel, the ground water table may be unchanged after excavation of the tunnel.

The ground water moves in a soil layer due to pressure head differences. The speed is also dependent of the hydraulic conductivity, K_{soil}, of the soil layer and could be expressed according to Darcy's law. For further information of calculating sub soil water transport, see for example the HBV-96 model by Bergström (1995) or lowering of the water table due to tunnelling by Ahlberg & Lundgren (1977).

The inflow to a tunnel is, as mentioned, dependent on a number of parameters. A typical requirement could, for example, be that the tunnel is not allowed to damage surrounding constructions or environment. Damage to surrounding constructions can occur due to a total- or differential settling of the soil layer, as a consequence of increasing the effective stress when the groundwater table is lowered. The change of the ground water table depends on the capacity of the ground water depository and the magnitude of the inflow to the tunnel. The magnitude of the inflow only indirectly affects the surroundings and may therefore be replaced as a requirement by some other suitable quantity. Instead it is the water table level, which affects the surroundings and therefore should be expressed as a requirement. By exchanging the requirements, the uncertainty related to the models, which for example express the relationship between the inflow and the water table can be decreased (Lindblom, 1999).

In order to calculate an corresponding inflow, the demand is expressed in a number of terms such as; acceptable load without damaging the building, soil settlement corresponding to acceptable load, ground water table corresponding to the settlement and in- and outflows corresponding to the change of the ground water table. The total variance for the calculated inflow may be high, which needs to be regarded when a decision concerning the inflow requirement is decided.

At an early stage when a tunnel project is proposed, only general information such as bedrock maps or information received from earlier adjacent projects are available. Thereafter some kind of survey; geological mapping and probe holes are commonly carried out to improve the information. Based on this information it is possible to decide an effective grouting method, which will be shown in this thesis.

The available information concerning the rock mass and the groutability of the rock mass is then widely improved when the project is started and as grouting work advances. By the end there will be a complete knowledge data base, which is of little use for the current project, but can be of great use for the next project carried out in a surrounding area or even at large distance but with similar rock mass properties.

It is possible to among a number of grouting methods with limited information decide an effective grouting method, but it is not possible to decide an optimal grouting method. By using an active design philosophy the grouting method can, during the progress of the project, be step by step improved, which enables an optimisation of the grouting method that fulfils the requirements.

2.3 Some grouting methods of today

Grouting can be performed based on different grouting philosophies. The required grouting work to seal a rock mass could for different projects be more or less complicated, dependent of the sealing requirements and the rock mass complexity. Some may pledge for probe holes to be able to adopt the most appropriate grouting concept, other may pledge for more time to be spent on grouting instead of investigating the rock mass. There are a great number of (more or less) different grouting philosophies. In this chapter, section 2.3.1 - 2.3.6, philosophies of some recently performed grouting methods are studied. The following projects were studied, with project data according to Table 2-1:

1. The access tunnels for the South Link-tunnels (road), Stockholm, Sweden
2. South Link tunnels, SL01 & SL02 (road), Stockholm, Sweden
3. Arlandabanan (rail), Arlanda, Sweden
4. Bergshamra tunnel (power), Solna, Sweden
5. Nordlänken (rail), Trollhättan, Sweden
6. Lunner tunnel (road), Grualia, Norway

Table 2-1 Summarized grouting project data for the studied projects.

Property/Project	Access SL	SL [01,02]	Arlandabanan	Bergshamra	Nordlänken	Lunner
Tunnel size [m^2]	55	75	100	20	110	61
Tunnel length [~m]	90	9000	8600	850	5500	1555
Average tunnel dept [m]	5	20-30	10	10-20	20-30	130
Requirement [litres/minute 100 m.]	2	0.5-2	2 - 5	5	2.5-3.5	10-20
~Initial conductivity [m/s]	(*) 1.9*10^{-6} – 5.1*10^{-8}	1.9*10^{-6} – 5.1*10^{-8}	1.7*10^{-6} – 1.7*10^{-8}	-	9.4*10^{-7} – 3.6*10^{-8}	-
Total number of grout fans [-]	-	608+160(Re)	~580	60	366	104
Procedure**	Thickening	Thickening	2 grouts	2 grouts	3 grouts	Thickening
Re-groutings (p > one grouting)	-	26%	~5 %	20%	45%	6%
Number of grout holes (first grouting)	22	32	22	4	31	24
Hole length [m]	18	20	20	20	20	24
Grouting overlap between fans [m]	-	5	5	5 (min.3)	5	4-9
Hole intensity [holes/m^2]	0.40	0.42	0.22	0.20	0.28	0.39
Hole inclination [degrees]	12	12	17.5	11.3	11.3	15
Hole diameter [mm]	64	64	64	64	64	64
Grouting Min - Max Time [hours]	21 - 29	24 – 97	16 - 20	8 - 16	21 - 39	20 – 29
Grout stop pressure (excess pressure)	2.5 Mpa	2.5 Mpa	2-4 Mpa	1.5 Mpa	2 Mpa	6-7 Mpa
Stop flow [l/min]	2	2	0.2	0.2	no	2 - 4
Packers (sleeves)	Disp./Multi.	Disposable	Disposable	Disposable	Multiuse	Disposable
Main Grout type	Inj30	Inj30	UF16 Rheocem800	Rheocem650 Rheocem800	UF16	UF12 Ind.(125)
Main grout type W/C-ratio	(2.0)-1.0-0.8	(2.0)-1.0-0.8	0.8-0.6	0.8-0.6	1.0-0.8-0.6	2.0-0.55
Waiting time before drilling [h]	5	5	4	4	5	5

(*) = assumed similar to the rock mass of the main tunnels. (**) = thickening is referred to an initial grout mix with higher W/C ratio and a successively lowering of the W/C ratio. If the used number of grout mixes for thickening is lower, it is marked as two or three grout.

The different methods and their components are at the end of this chapter summarized in a generalized **sealing time component chart**. The sealing time component chart is then used as a base for the literature review presented in chapter 3 and for the **sealing time equation** presented in chapter 4.

2.3.1 The access tunnels for the South Link tunnels

The South Link is a road traffic system in the south of Stockholm. The South Link access-tunnels were excavated during 1998, with the purpose of creating an access for the following South Link road tunnels. The access tunnel cross section area was approximately 55 m^2.

The rock mass within the area of the South Link is dominated by sedimentary gneiss. Further description on geology is given in next section (South Link road tunnels).

The number of grout holes for the primary grouting varied between 22 and 29 for the different access tunnels.

The grouting method consisted of 10 different activities, as shown in Figure 2-4. Based on experience the time for each activity of the grouting method was estimated for the project, which gave a total time for the method between 21 and 29 hours. During this project, the time for the grouting method was also measured, which resulted in a time of between 22 and 25 hours (Janson, 1998b).

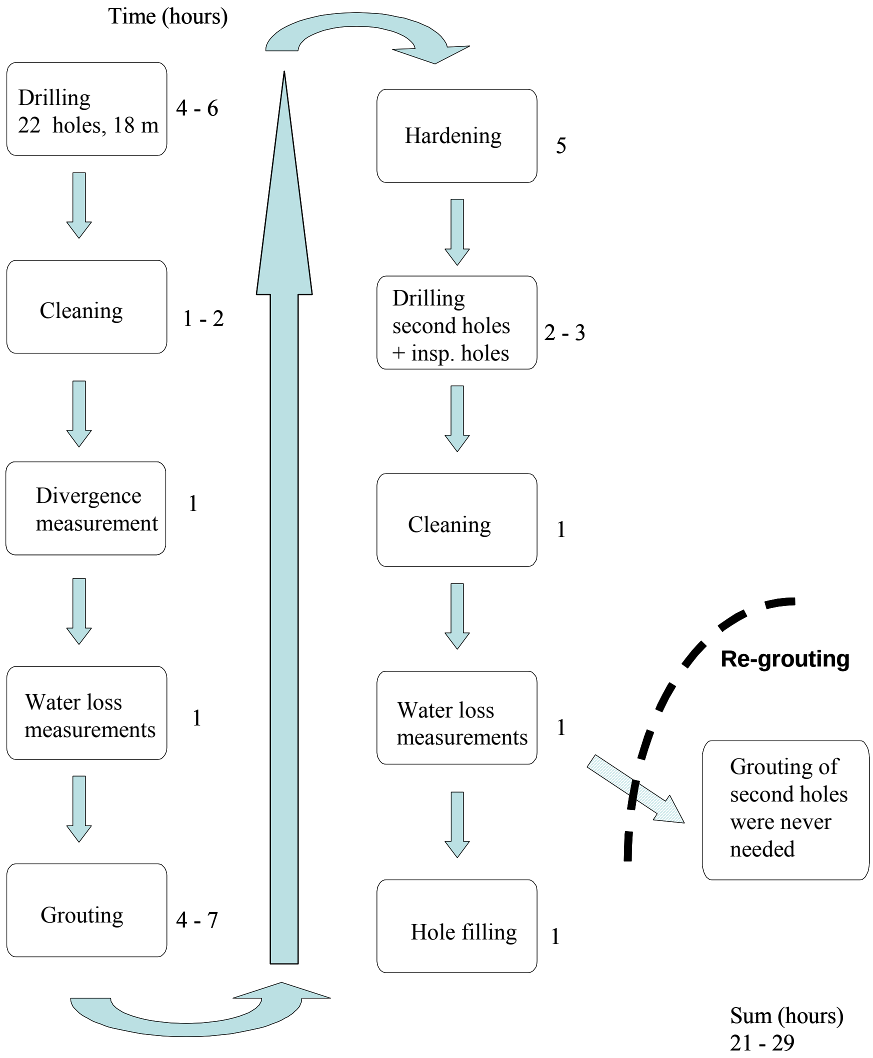


Figure 2-4 A generalisation of the grouting method carried out at the South Link access-tunnels.

2.3.2 South Link road tunnels

The South Link is a road traffic system that links some main traffic routes in the south of Stockholm. It has a total length of 6 kilometres, of which 4.5 kilometres runs in tunnels. The South Link tunnels were excavated between 1999 and 2002. For the tunnel parts studied within this work the tunnel cross-sectional area was 75 m^2.

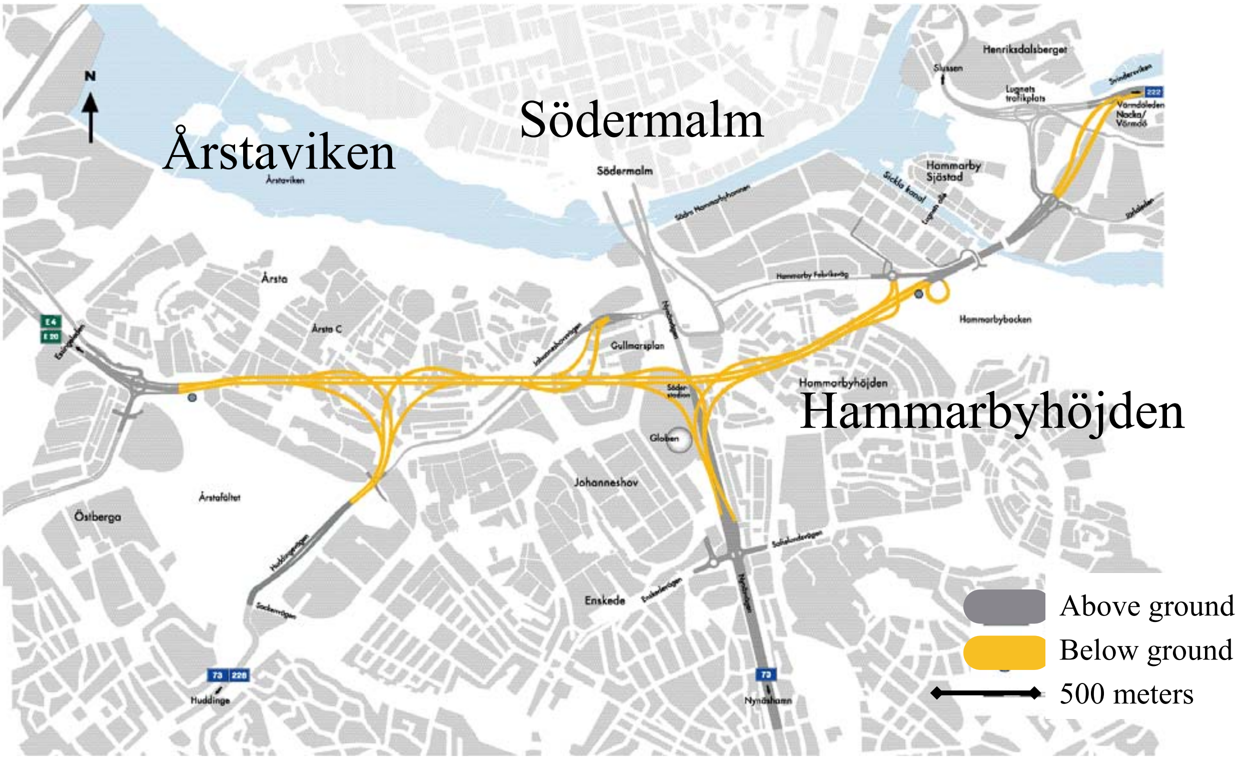


Figure 2-5 Layout of the South Link tunnels.

The rock mass within the area of the South Link is dominated by sedimentary gneiss and grey-red gneissic-granite, about 2000 millions years old. The dominating minerals are quarts, feldspar and mica. The jointing of the Gneiss changes rapidly between different sections and the block size is irregular. The joints are partly filled with calcite or clay. The joint opening is homogenous and narrow, with a physical aperture ranging between ~35 to ~160 µm, with an average of ~90 µm (Barton, 2002). The RQD value is in general ranging between 80 and 100.

The main tunnels are elongated along a schistose zone. In addition there are some vertical joint zones, which cross the main tunnels (Palmqvist, 1999). A large number of probe holes were measured for water loss during the project, which resulted in an average hydraulic conductivity of between $1.9*10^{-6}$ and $5.1*10^{-8}$ [m/s].

The rock mass at the South Link could further be described as a typical Stockholm rock mass, which have previously been excavated and grouted to great extent. Based on previous grouting activities in the Stockholm area, general descriptions of a typical Stockholm area rock mass from a grouting point of view are shown in Figure 2-6 (Andersson & Sellner, 2000).

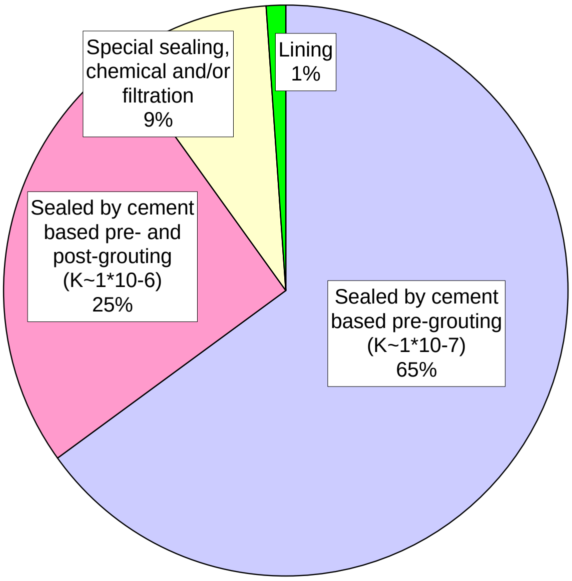


Figure 2-6 Typical Stockholm area rock masses regarding grouting (Andersson & Sellner, 2000).

The grouting method for sealing the South Link tunnels was changed during the progress of the project to improve the sealing result and the grouting efficiency. Probe holes were initially included in the method, but they were later excluded and further on included again. Initially grouting started with a W/C 2.0 grout mix, later grouting started with a W/C 1.0 grout mix. In general the grouting method could be described according to Figure 2-7. Grouting at the South Link has predominantly been performed as cement based pre-grouting. The chosen concept was by continuously grouting each 15 meters with 20 meters long grout holes. There were no inspection holes prior to the grouting in order to decide the required extent of the grouting.

The number of grout holes varied dependent of the size of the tunnel. For a 75 m^2 section there was 32 grout holes, angled 12 degrees out from the tunnel centre. The grout was predominantly based a conventional grout cement (d_{95}=30µm), with W/C ratio between 2.0 and 0.8. Only a small amount of micro cement was used. The grouting pressure was set to 2.5 MPa above ground water table and the stop flow was set to 2 litres / minute.

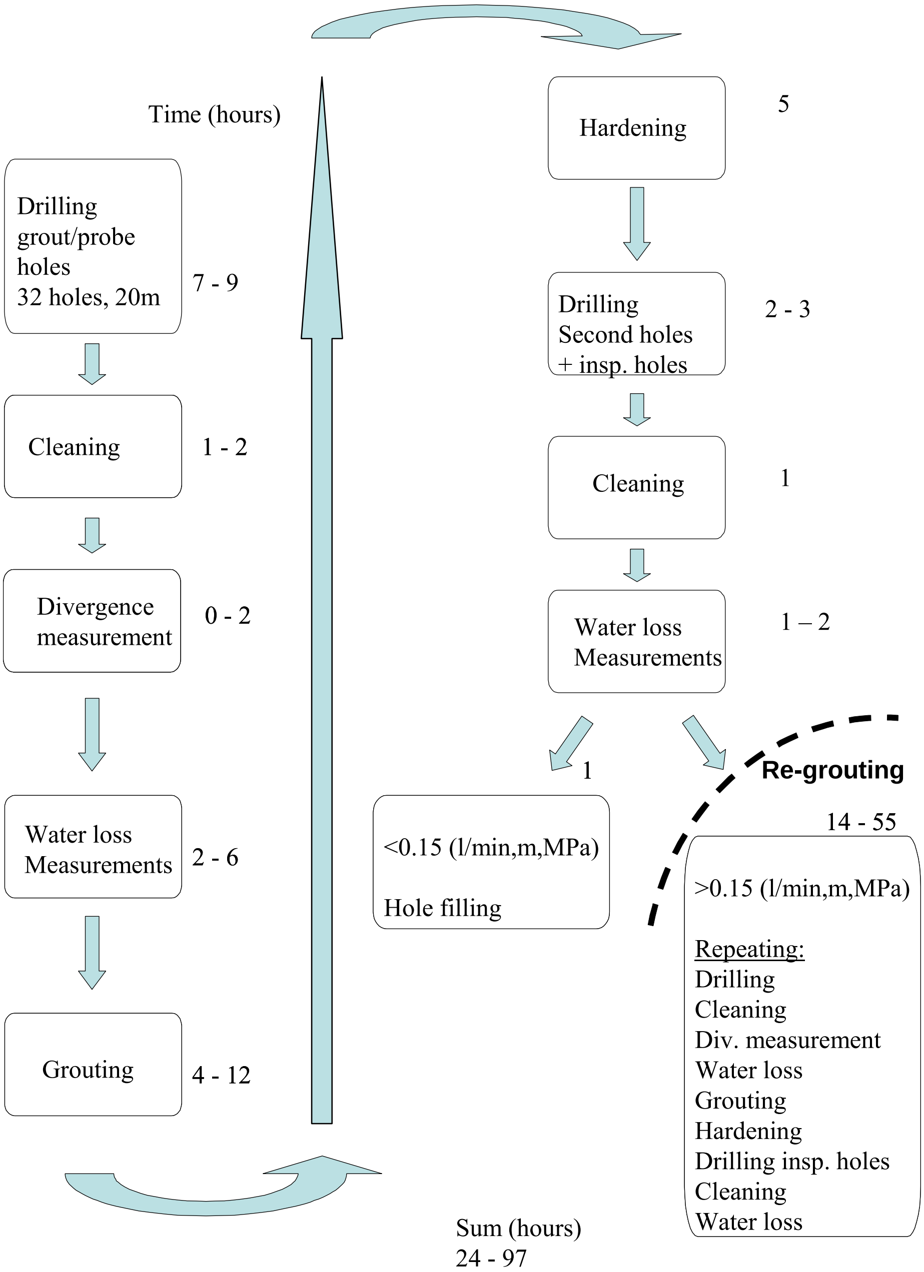


Figure 2-7 Generalisation of the grouting method carried out at the South Link tunnels.

The grouting method consisted of ten different activities, as shown in Figure 2-7.

Based on experience from the project, the time for each activity of the grouting method was estimated, which gave a total time for the method of between 24 and 97 hours.

2.3.3 Arlandabanan

Arlandabanan is a fast track rail link between Arlanda airport and central Stockholm with 8.6 km of tunnel in the area of Arlanda. The tunnels were constructed between 1996 and 1997 and had an cross-sectional area of ~100 m^2.

The rock mass at Arlanda is dominated by three main geological regions as shown in Figure 2-8. In the central area there is a combination of mica schist and mica gneiss. North and South of the airport there is a massive Granodiorite.

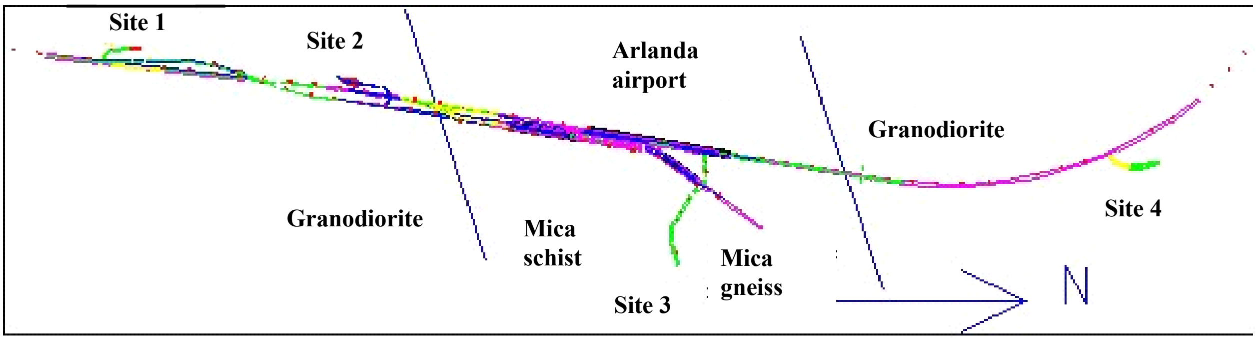


Figure 2-8 Layout of the Arlandabanan tunnels with three main geological regions.

The granodiorite is a strong and massive rock type, but in the Arlanda area is influenced by horizontal fracturing. A possible reason for the horizontal fracturing could be the large horizontal stress, which in the area is measured to between 2-10 MPa at shallow depths.

The mica schist is strongly fractured along a vertical stratification with foliation planes (N40ºØ/70ºV) partly affected by clay. However, this rock type has a high compressive strength and in between the foliation planes the rock mass is massive, characterized as gneissic.

The hydraulic properties of the rock mass for site 3 shown in Figure 2-8, were investigated by water loss measurements in core holes. During the investigation 80 % of the rock mass was classified as a mica gneiss/schist and 20 % of the rock mass as a granodiorite. 87 % of the rock mass had a medium hydraulic conductivity ranging between ~$1.7 \cdot 10^{-6}$ and ~$1.7 \cdot 10^{-8}$ (0-10 Lugeon's) and 13 % of the rock mass had a higher conductivity ranging above ~$1.7 \cdot 10^{-6}$ (>10 Lugeon's). The investigation showed that the granodiorite was more conductive than the mica gneiss/schist (Hässler et al, 1998).

Grouting at Arlandabanan has predominantly been performed as pre-grouting with cement based grouts. The chosen method was to continuously grout each 15 meters of the tunnel with grout holes of a length of 20 meters, giving an overlap of about 5 meters. Since grouting was performed even if not required, it was decided to exclude the inspection holes. Consequently it was not possible to estimate the required extent of grouting based on inspection holes drilled ahead of excavation.

The number of grout holes varied depending on the size of tunnel. Between 22, 30 and 34 holes were used, with 22 holes for the 100 m^2 section. For some occasions, extra holes were grouted in the middle of the tunnel face. For the smaller pilot tunnel, 10 grout holes were used. The angle of the grout holes varied between 6 and 20 degrees. For normal conditions the angle was set to 17.5 degrees. The grout mixes were based on micro cements with w/c ratios of 0.6 and 0.8.

According to the initial technical grouting specification a maximum grouting excess pressure for unrestricted areas of up to 4 MPa and for restricted areas an excess pressure between 0.5 and 1.5 MPa were allowed. During the construction, in order to improve the sealing result, it was decided to increase the grouting excess pressure to between 0.5 and 6 MPa, with a stop flow of 0.2 litres / minute.

Uneven grout distribution and approximately 80 % non-groutable holes urged for a re-design of the grouting method. The decision was taken to open the rock joints with an excess pressure of 6 MPa for an initial grouting of 150 litres, which was considered as high since there was only between 4 and 12 meters of rock cover. After an initial joint opening the pressure was decreased to 2 MPa. The maximum grout volume was, at the same time, decreased from 1200 litres to 350 litres.

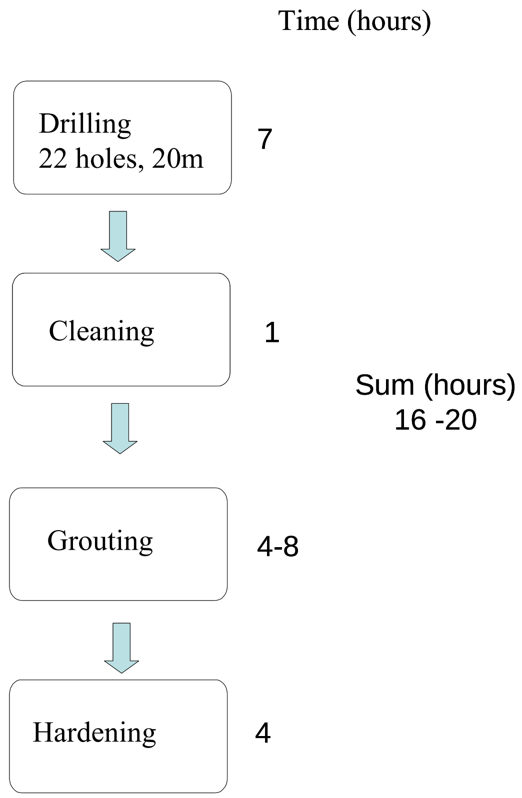


Figure 2-9 Generalisation of the grouting method carried out at Arlandabanan.

The grouting method consisted of four different activities, as shown in Figure 2-9. Based on experience from the project, the time for each activity of the grouting method was estimated, which gave a total time for the method of between 16 and 20 hours.

2.3.4 Bergshamra

The Bergshamra power tunnel was carried out between august 1997 and October 1998 in Solna, north of Stockholm. The tunnel was constructed in order to relocate an existing overhead power transmission line which ran through a residential area.

The excavated tunnel was 850 meters long with a cross-sectional area of around 20 m^2. The grouting concept suggested continuous pre-grouting. Grouting started with 4 probe holes, which were measured for outflow to decide the required extent of the grouting. The probe holes were then grouted. If the stop criteria for grouting were fulfilled, no further grouting was necessary. If not, new grout holes were drilled and grouted next to the leaking holes. The grout fan was then condensed until the stop criteria was reached for all grout holes, commonly between 6-16 grout holes per fan. The stop criterion was to reach 1.5 MPa above ground water pressure at a flow of 0.2 l/minute or grouting 70 kg of cement per meter of grout hole. Grout mix I (w/c 0.8) was used until 40 l/m, had been reached, thereafter, grouting was continued with grout mix II (w/c 0.6) for another 40 l/m and then thickened until stop was obtained. Two types of micro cements were used Rheocem 650 and Rheocem 800. The grouting method, which consisted of six different activities, could, in general, be described as in Figure 2-10.

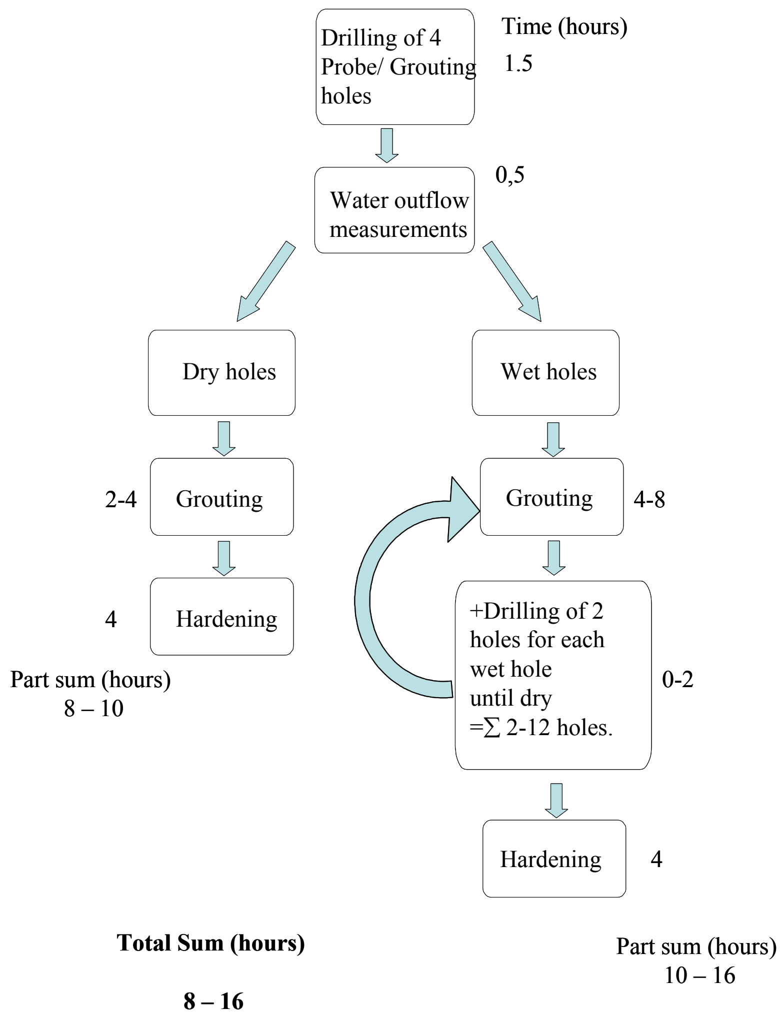


Figure 2-10 A generalisation of the grouting method carried out at the Bergshamra power tunnel.

Based on experience from the project, the time for each activity of the grouting method was estimated, which gave a total time for the method of between 8 and 16 hours.

At Bergshamra the required shear strength of the grout before drilling was set to 1 kPa, which is lower than normal. Therefore the hardening time for this project could be reduced from the norm of 5 hours to just 4 hours. In total 60 grout fans were performed during the construction of the tunnel.

2.3.5 Trollhättetunneln

The Trollhätte tunnel, a part of the Nordlänken rail road, is a 3.5 km single tunnel with a cross-sectional area of 110 m^2, which is currently under construction (between 2003 and 2004) north of central Trollhättan, see Figure 2-11. Parallel to the main tunnel there is also 2.0 km of rescue tunnels excavated. The project is a relocation of an old curved single lane rail track to a double lane straight rail track, which is done in order to improve the capacity of the railway.

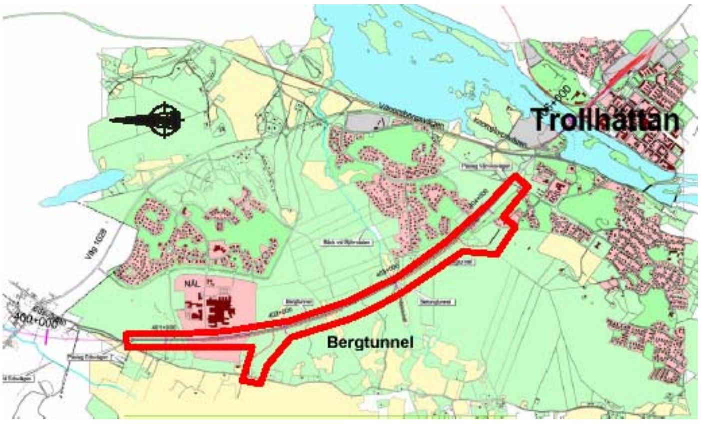


Figure 2-11 Location of the Nordlänken tunnel, north of central Trollhättan.

The rock mass in the area is dominated by granite, gneiss and Granodiorite. The hydraulic properties of the rock mass were investigated by water loss measurements in 12 core holes from the surface. The hydraulic conductivity was evaluated to between $\sim 1.4 * 10^{-7}$ and $\sim 3.6 * 10^{-8}$.

The suggested concept was to pre-grout the whole tunnel with initially 25 grout holes. In order to improve the grouting result and to decrease the number of re-grouting, (initially 45 % of the fans had to be re-grouted), the number of grout holes was increased to 31 (Manell, 2004). To study the grouting result, inspection holes were drilled and measured for water loss. Initially 10 inspection holes were drilled, to optimize, they were later decreased to 6 holes. If water loss in the inspection holes was observed, according to the present sealing class adjacent holes close to the inspection holes were drilled and grouted. This resulted in between 6 to 18 re-grouting holes. On average there were 9 re-grouting holes for each fan. The sealing requirement for the inspection holes was dependent on the allowed inflow to the tunnel and was divided into two sealing classes.

Class 1, (1-3 litre/min. 100m.)	allowed water loss < 0.2 litre/min. m. MPa
Class 2, (3-7 litre/min. 100m)	allowed water loss < 0.5 litre/min. m. MPa

For areas with at least 15 meters of rock cover the stop pressure was set to 2.0 MPa above ground water table. For areas with less than 15 meters of rock cover the allowed pressure was between 0.2 and 1.6 MPa above ground water table.

The grouting method included three grouts (see Table 2-2). Grouting started with grout type I and continued until 20 kg cement per meter had been grouted. Thereafter grout type II was used for another 20 kg cement per meter and after that grout type III for another 50 kg cement per meter. If no stop pressure had been reached before grouting of 50 kg cement per meter with grout type III, grouting stopped and the fan was condensed.

Table 2-2 Grout mixes used at Nordlänken.

Grout type	Specification (in brief)
I	Micro cement applied for sealing of fine fissures < 0.2 µm with a w/c ratio of 0.8.
II	Conventional grout cement applied for sealing of fissures > 0.2 µm with a w/c ratio of 0.6.
III	Accelerated grout cement applied for sealing of open fissures and structures with a w/c ratio of 0.5.

The grouting method for the Nordlänken is in brief shown in Figure 2-12.

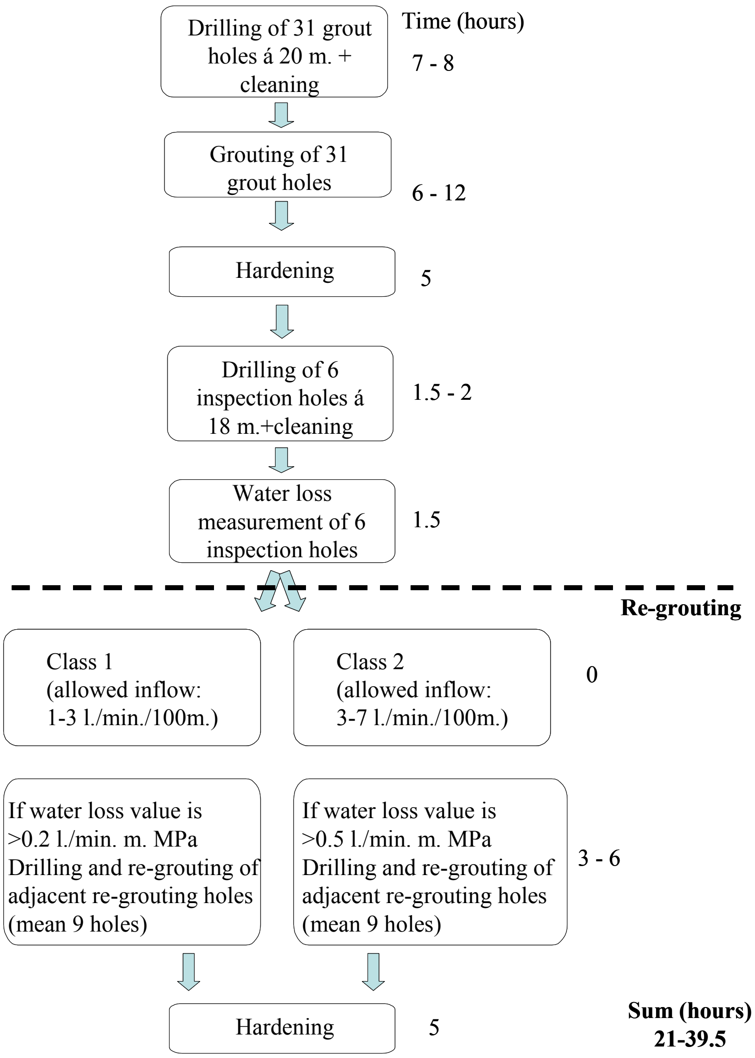


Figure 2-12 A generalisation of the grouting method carried out at Nordlänken.

The grouting method consisted of six different activities, as shown in Figure 2-12. Based on experience from the project, the time for each activity of the grouting method was estimated, which gave a total time for the method of between 21 and 39 hours.

2.3.6 Lunner tunnel

The Lunner tunnel is an east-west road tunnel located about 40 kilometres north of Oslo. The tunnel was excavated between September 2001 and September 2002. The tunnel has a cross-sectional area of 57 m^2 and the total tunnel length is 3.8 kilometres. This study only concerns the east part of the tunnel (1555 m.).

The bedrock at the Lunner tunnel belongs to the Oslo field and is on the edge between older

Cambrian-silur sediment and younger granite or syenitte. From the east, the first 208 meters (chainage 3948-3740) runs through a sandstone/conglomerate rock mass which also consist of a rhombic porphyrite dyke. The next 1110 meters (chainage 3740-2640) runs mainly through volcanic rocks, which are described as dark, very fine grained, feldspar and quartz rich and occasionally almost like rhombic porphyrite. The next 60 meters (chainage 2640-2580) runs through alternating syenite and vulcanic rock masses. The last 185 meters (< 2580) runs through a syenite (Andersson, 2003).

The overburden is between 10 and 220 meters, with an average of 130 meters. The inflow before grouting was estimated to about 80 litres per minute and 100 meter of tunnel. The outflow, measured as the total flow from all grout holes, is shown in Figure 2-13. High inflows occurred in the syenitte, which is also positioned directly below a lake. The distance between the tunnel and the lake is 75 meters.

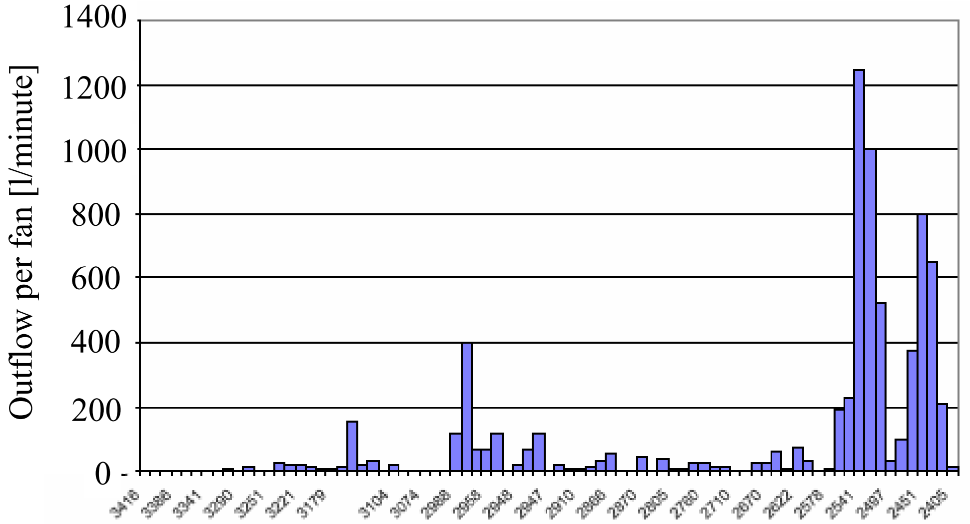


Figure 2-13 Fan outflow measured the eastern part of the Lunner tunnel (Andersson, 2003).

Water loss measurements were, for some locations, carried out in core holes. The water loss was measured to an average of between 0.53 LU to 1.69 LU, with a peak value of 4.25 LU (Holmöy, 2002). This means that the conductivity could be estimated to between 9.0*10^{-8} [m/s] and 2.9*10^{-7} [m/s].

In the Lunner project, there were three different sealing requirements dependent on the location, as shown in Table 2-2.

Table 2-3 Inflow requirement classes for the Lunner tunnel.

Class	Requirement	
0	No inflow requirement	∞
1	Moderate requirements	20 litres/min/100 meters and 10 litres/min/20 meters
2	High requirements	10 litres/min/100 meters and 5 litres/min/20 meters

Grouting at the Lunner tunnel was predominantly carried out as pre-grouting with cement based grouts. The chosen concept was continuous grouting using 24 holes to a length of 24 meters (only a few sections were left without pre-grouting).

The hole inclination was set to 15 degrees and the grout hole diameter was 64 mm. The grouting pressure varied between 4 and 7 MPa above the ground water table and the stop flow was set to between 2 and 4 litres / minute.

Initially the grouting method included between 1 and 4 probe holes in which the outflow was measured as a base to decide the grouting method. The probe holes were rather early excluded from the grouting method.

Instead of probe holes, a full grouting fan was drilled and measured for outflow. If the total outflow was above 20 litres per minute conventional cement (d_{95}=125 μm) was used. If the total outflow was less than 20 litres per minute, micro cement (d_{95}=12 μm) was used.

Grouting was started with a thin grout mix W/C ratio of 2.0 and continued with a gradual thickening of the grout mix to W/C ratio of 0.55.

The grout fan initially consisted of 25 holes with a length of 24, which with 15 meter of rock mass excavated between the groutings gave ~9 meters of overlap.

In order to improve the grouting method the number of holes was decreased to 14 and the overlap decreased to ~4 meters. A distribution for the number of grout holes per fan for the studied part of the tunnel is shown in Figure 2-14. As noticed the dominating number of holes per fan was ~14 and ~24. The drilling time for the ~14 hole fan was on average 5.4 hours and the drilling time for the ~24 hole fan was on average 8.4 hours, as shown in Figure 2-15.

The criterion for the total amount of outflow from the control holes, used for to decide the grout mix, was changed during the project. If the outflow was above 15 (earlier 20) litres per minute, conventional cement was decided. If the outflow was less than 15 (earlier 20) litres per minute, micro cement was decided. Later on, the number of grout holes was increased back to 24 (Måge, 2003).

The grouting time for the ~14 hole fan was on average 8.6 hours and the grouting time for the ~24 hole fan was on average 14.5 hours, as shown in Figure 2-16. In total 104 grout fans were performed during the construction of the studied part of the tunnel, most of them consisted of 24 grout holes.

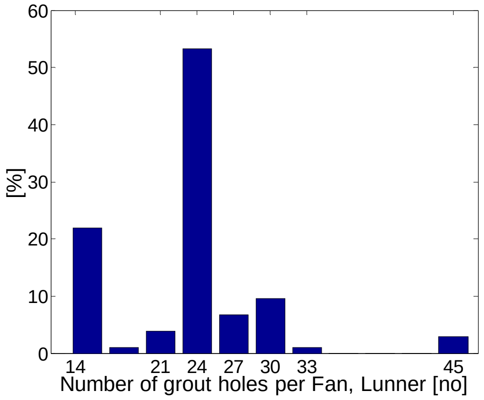


Figure 2-14 Distribution of the number of grout holes per fan at the Lunner tunnel.

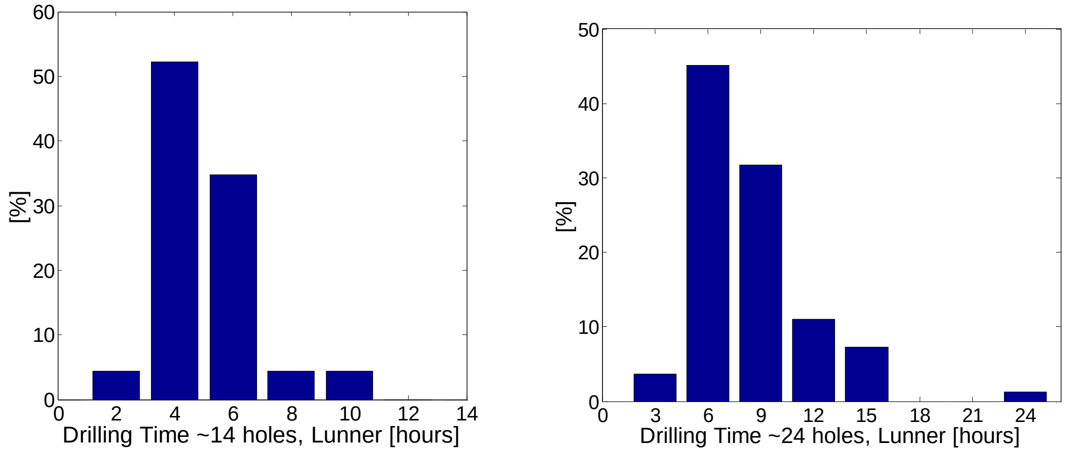


Figure 2-15 Drilling time distribution for ~14 and ~24 holes at the Lunner tunnel.

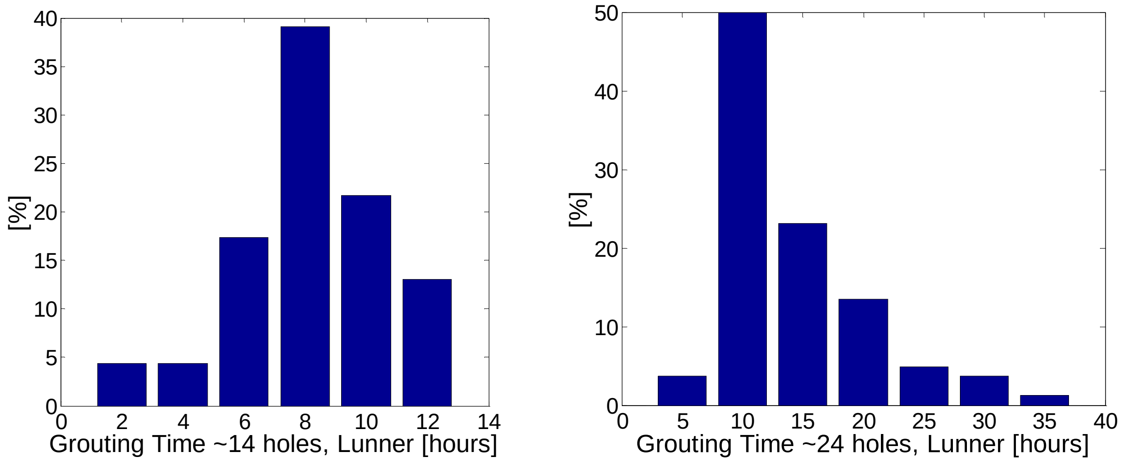


Figure 2-16 Grouting time distribution for ~14 and ~24 holes at the Lunner tunnel.

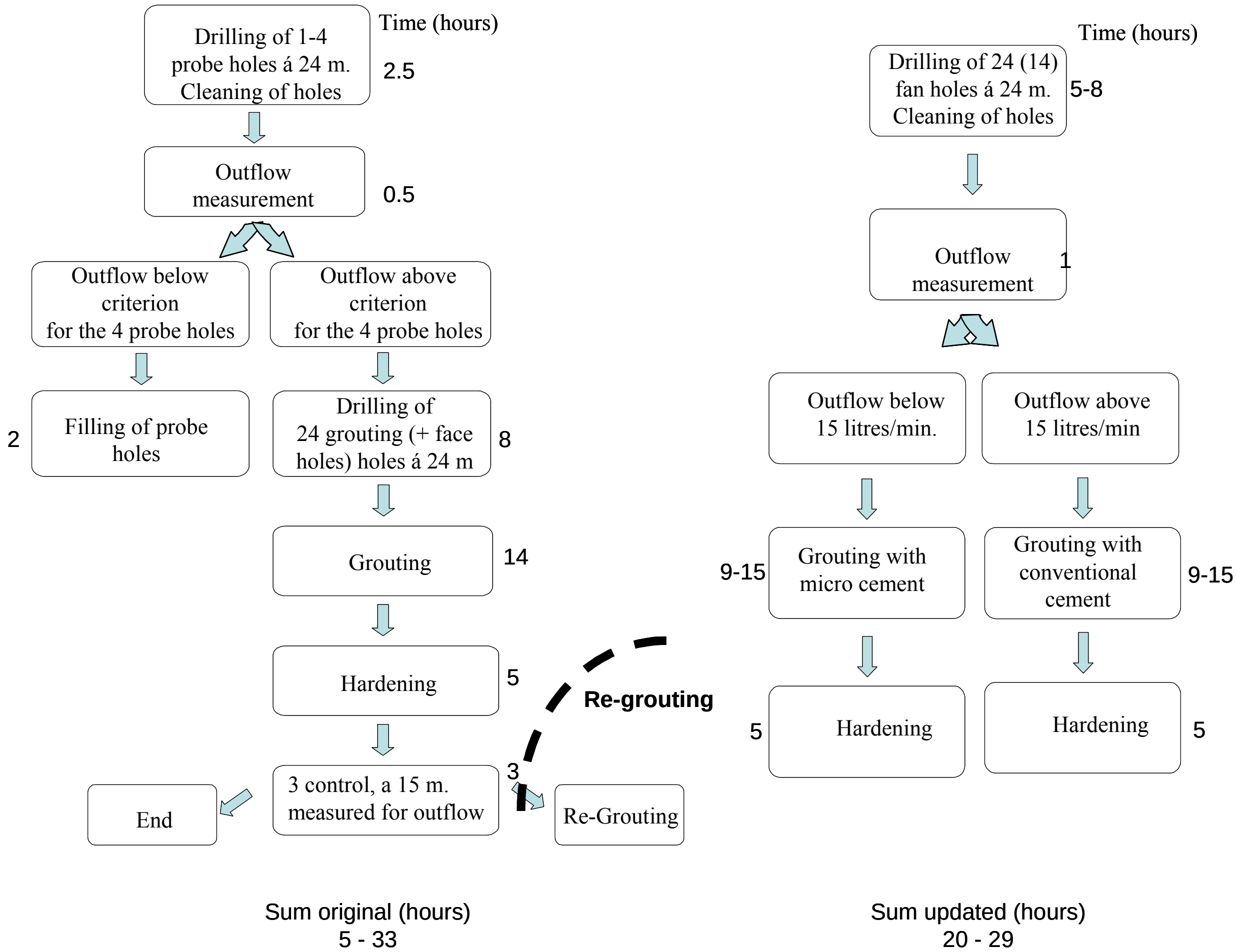


Figure 2-17 A generalisation of the grouting method carried out at the Lunner road tunnel. Original method (left), updated method (right)

Based on experience from the project, the time for each activity of the grouting method was estimated, which gave a total time for the method of between 20 and 29 hours.

2.3.7 Summary of grouting methods

Six different projects have been studied regarding the grouting method to seal a tunnel. All of the grouting methods consisted of the activities: drilling, grouting and waiting. Some of the methods also included the activities: probe / inspection holes and re-grouting. None of the methods included the activity post-grouting as an ordinary method of sealing. In reality post-grouting sometimes will be an activity or by purpose could be regarded as an alternative for grouting or re-grouting. Based on the six projects a grouting method could in general be noted consisting of five main activities:

Drilling

Grouting

Waiting

Probe holes / water loss measurements

Re-grouting

Each one of the five main activities are affected by components such as number of holes, length of holes etc. (given in Figure 2-18) and thereby the sealing time. A generalized sealing time component chart has therefore been noted, as shown in Figure 2-18. The *Sealing Time* is as shown in the figure defined as the top activity, which includes the time for drilling, grouting and post-grouting etc. In order to seal the rock mass to a specified required level. The *grouting time* is defined as the time from start to stop of the grouting pump and depends on the components shown in the figure. The activity post-grouting is very different compared to grouting and re-grouting and therefore treated separately.

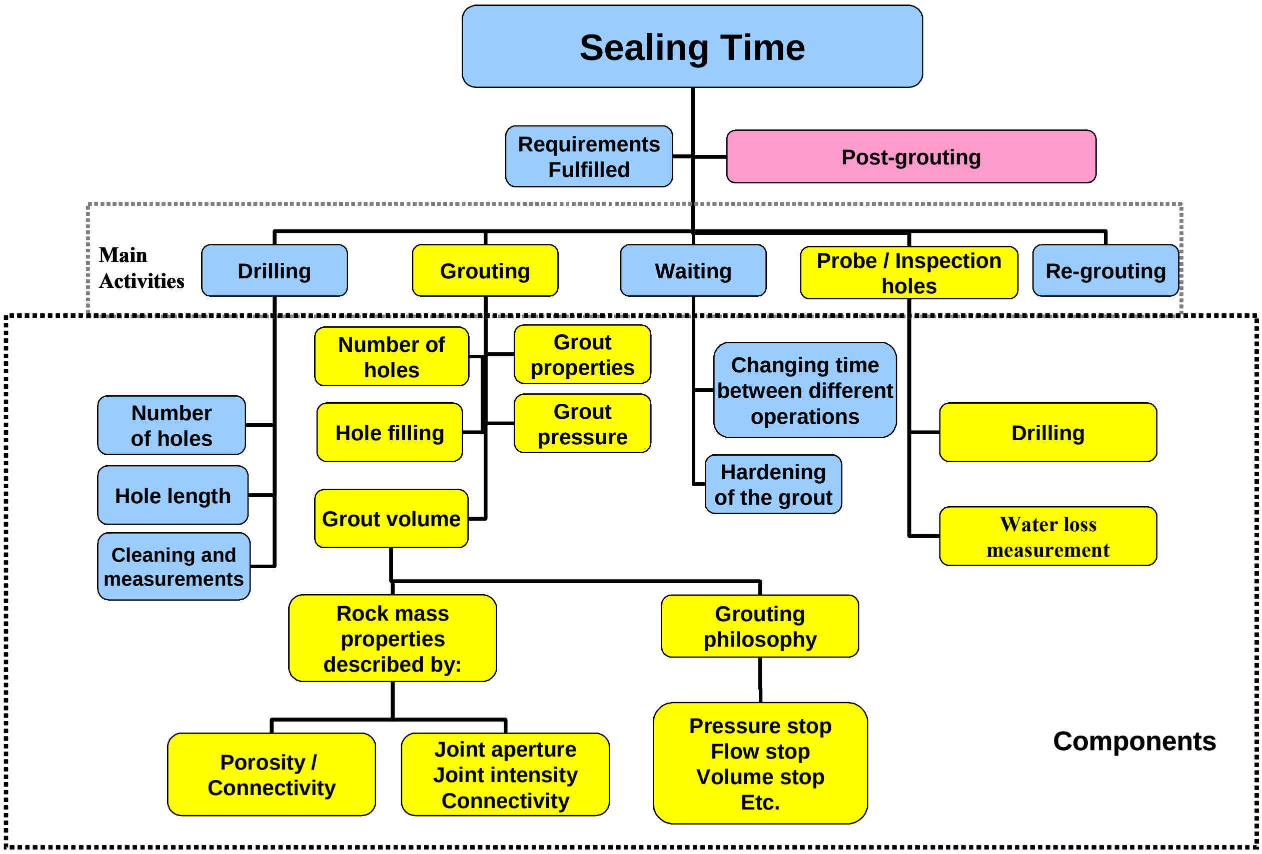


Figure 2-18 Generalized sealing time component chart.

The required time for each activity of the six grouting methods have been presented earlier and are here summarized as defined by the sealing time component chart. The time can then be calculated as a minimum and a maximum time for each main activity and is shown in Table 2-4. To compare the time between the different activities, the time share [%] has also been calculated. The share is then calculated as the average time, taken as the central time of each interval, compared to the total central time for all activities. The time share may therefore be incorrect as the skewness of each distribution is neither known nor regarded.

As noticed the time share is almost equal for *drilling* and *grouting*. The initial degree of optimization may differ between drilling and grouting. Based on the time share there is an equal potential for further optimization of drilling and grouting.

The time share of *waiting* is 17 %. By using other grouting materials or by applying different philosophies for waiting, the time share could be reduced substantially, which gives waiting a high potential of optimization.

The time share of *Probe / Inspection holes* is only 11 %, and it is therefore regarded as having less potential for optimization.

The time share of *re-grouting* is 22 %, which is almost as high as the drilling or grouting. It is difficult to estimate the potential for optimization as re-grouting may depend on a prior decision. For some methods, fewer grout holes in combination with a more extensive re-grouting may be chosen. For normal conditions, it could be regarded as preferable, to minimize the number of re-groutings, as extensive re-grouting increases the number of establishments, which consequently increases the time without production. For a situation with a high requirement on low inflow, it may be necessary with more than one grouting to threat the rock mass with different grout mixes etc. From a production point of view a systematic re-grouting may be preferred, rather than sporadic re-grouting based on water loss measurements, as the total production cycle will be easier to predict.

Table 2-4 The average maximum and average minimum times and the time share of each main activity as a function of the total time for the six projects.

	Average min. time [h]	Average max. time [h]	Time share of total time [%]
Drilling	35.5	49.5	24
Grouting	29	62	26
Waiting (hardening of grout)	28	33	17
Probe / Inspection holes	15.5	24	11
Re-grouting	17	61	22

Based on the six projects, the grouting works are in general analysed by two different concepts or a combination of them:

Probing

Learning the rock mass

The concept *probing* means that the rock mass ahead of the tunnel face is probed and the hydraulic properties are measured by, for example, water loss measurements. The grouting method is then for each fan to be adjusted, based on the measured properties. The concept *learning the rock mass* means that based on the performed grouting and the measured result, the grouting method is updated for the fans ahead. Both of the concepts can be categorized as *active design* philosophies.

Of the six projects, five are carried out in Sweden and one in Norway. In general, higher inflows and grouting pressures are accepted in Norway compared to Sweden. Micro cements have been used more often for the tunnels with the highest inflow requirements. The grouting time for the different grouting methods does not correspond to the inflow requirement. Therefore the conclusion is that the different activities within the grouting methods need to be optimized, so that the sealing time better corresponds to the inflow requirements and the initial rock mass properties.

Based on the sealing time component chart, a first step to optimize the sealing time could be to compare the main activities with experience from previous projects or Table 2-4 and then decide which one of them has the largest potential for optimization. Each of the main activities depends on a number of underlying components, which will be the focus for the optimisation.

The six different grouting projects could further be categorises into two groups:

Small quantity of grouting works

Large quantity of grouting works

The two small projects with small quantity of grouting works have in common that the grouting method has stayed almost unchanged during the progress of the project. The four larger projects have all gone through a varying quantity of change regarding the grouting method. For a large project there is a high potential to improve the grouting method by active design where as for a smaller project the potential is less, which has been shown from practice.

Based on the sealing time component chart, a general equation to calculate the sealing time is expressed in chapter four and used to compare different grouting methods in chapter six.

Chapter Three

3 Grouting from a sealing time perspective

This chapter concerns grouting from a sealing time perspective mainly as a literature review, but also complemented with experiences gained from previous work and laboratory experiments within the project.

3.1 Requirements

3.1.1 Models for water inflow and sealing effect

In order to achieve a sealing of the rock mass around a tunnel and thereby a reduction of the water inflow, it is necessary that the grout penetrates and fills out the joints of the rock mass. The aim is to achieve a dense zone, wide enough, around the tunnel periphery as shown in Figure 3-1 (Brantberger et al, 1998).

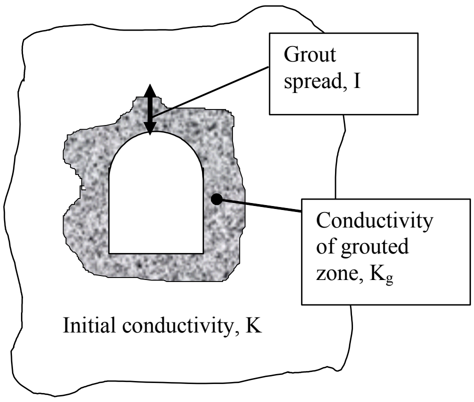


Figure 3-1 Grouted zone around a tunnel, with penetration length I, and hydraulic conductivity, K_g.

The following theory, with a homogenous isotropic rock mass approach, refers to an appendix from the Swedish Road Administration (1993), which is based on works from e.g. Renius (1977), Bergman & Nord (1982) and Alberts & Gustafson (1983).

The expected water inflow to a tunnel is dependent of: geometry, location, water pressure, rock mass geological- and hydrogeological conditions and thickness and hydraulic conditions of the grouted zone. The inflow could for tunnels without a grouted zone be calculated from Equation 3-1 for shallow locations, and from Equation 3-2 for deep locations. A shallow located tunnel is defined as a tunnel, which is located less than approximately 3 to 4 times the tunnel diameter, below the water table.

Shallow located tunnels (ungrouted)

$$q = \frac{2 \cdot \pi \cdot K \cdot (H - R_t)}{\ln\left(\frac{2 \cdot H}{R_t}\right) + \xi}$$ Equation 3-1

Deep located tunnels (ungrouted)

$$q = \frac{2 \cdot \pi \cdot K \cdot H}{\ln\left(\frac{2 \cdot H}{R_t}\right) + \xi}$$ Equation 3-2

Where:

q	flow per meter of tunnel	[m^3/s]
K	initial hydraulic conductivity	[m/s]
H	water pressure (in tunnel centre)	[m]
R_t	radius of tunnel	[m]
ξ	skin factor	[-]

The flow per meter into the tunnel *q* depends on the initial conductivity *K*, the water pressure *H* and the radius of the tunnel R_t. The skin factor ξ takes into account that there is a pressure loss at the tunnel periphery. This skin factor depends for example on: less permeability in the near field due to rock mass deformations and obstructed water inflow to the tunnel due to a partly saturated water zone. The value of the skin factor was at Äspö Hard Rock laboratory determined to between 3 and 7. For other project, different from Äspö, different values of the skin factor could be expected (Swedish Road Administration, 1993).

For tunnels with a grouted zone the inflow is also dependent on the conductivity of the grouted zone K_g and the thickness of the grouted zone *I*. The inflow could then be calculated with Equation 3-3 and Equation 3-4.

Shallow located tunnels (grouted)

$$q = \frac{2 \cdot \pi \cdot K_g \cdot (H - R_t)}{\ln\left(\frac{R_t + I}{R_t}\right) + \frac{K_g}{K}\xi}$$ Equation 3-3

Deep located tunnels (grouted)

$$q = \frac{2 \cdot \pi \cdot K_g \cdot H}{\ln\left(\frac{R_t + I}{R_t}\right) + \frac{K_g}{K} \cdot \ln\left(\frac{2 \cdot H}{R_t + I}\right) + \frac{K_g}{K} \cdot \xi}$$ Equation 3-4

Where:

I	thickness of the grouted zone	[m]
K_g	hydraulic conductivity of the grouted zone	[m/s]

If all of the pressure loss is taken in the grouted zone and if the hydraulic conductivity of the grouted zone is reduced to approximately 1/10 of the initial, Equation 3-3 is also valid for deep located tunnels.

The equations with a grouted zone were first presented by the Swedish Road Administration (1993), since then they have been modified. The skin factor ξ should be multiplied with K_g / K, referring to discussions with Gustafson (1999) as shown in the above equations.

3.1.2 Required sealing effect

The inflow requirements, discussed in chapter 2, measured as water inflow to a tunnel will in combination with the inflow without grouting to the tunnel result in a required sealing effect.

Dependent on the initial hydraulic conductivity of the rock mass, the tunnel geometry and the water pressure, the required sealing effect can commonly be between 90 % and almost 100 %. In Table 3-1 some calculation examples are shown for one shallow and two deep located tunnels, with inflow requirement set to 5 litres per minute and 100 meter of tunnel. The required sealing effect has been calculated from Equation 3-5:

$$Sealing\ \ effect\ [\%] = 100 \cdot \frac{Inflow\ \ without\ \ grouting\ - Inflow\ \ with\ \ grouting}{Inflow\ \ without\ \ grouting} \qquad \text{Equation 3-5}$$

Table 3-1 Sealing effect requirements for shallow and deep located tunnels. (R_t = 5 m, inflow requirement 5 litres / minute and 100 m of tunnel, skin factor = 3).

Water pressure (m)	Rock mass conductivity K (m/s)	Water inflow without grouting (lit/min. 100m)	Required sealing effect (%)
10	$1.7*10^{-7}$	7	29
10	$1.7*10^{-6}$	73	93
10	$8.5*10^{-6}$	365	98.6
50	$1.7*10^{-7}$	53	90
50	$1.7*10^{-6}$	534	99.1
50	$8.5*10^{-6}$	2672	99.8
100	$1.7*10^{-7}$	96	95
100	$1.7*10^{-6}$	958	99.5
100	$8.5*10^{-6}$	4791	99.9

It could be noted that both the rock mass hydraulic conductivity and the water pressure has a large influence on the water inflow to the tunnel, and thereby also on the required sealing effect. The tunnel radius influences, but is less important as shown in Figure 3-2.

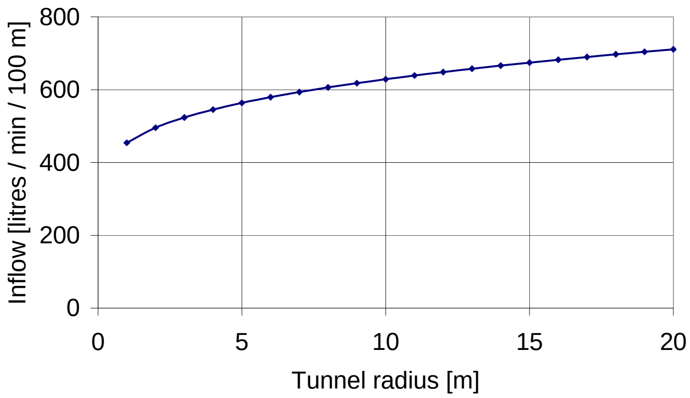


Figure 3-2 The inflow dependency of the tunnel radius, calculated for a rock mass with an initial hydraulic conductivity of $1*10^{-6}$m/s, a water pressure of 100 meters and a skin value of 3.

The sealing effect was studied for a large number of grouting fans at the Hallandsås Rail Link. On the north side the highest sealing effect, 90 %, was reached after the first grouting and the following groutings had less influence on the total sealing effect as shown in Table 3-2. The required sealing effect for the north side was calculated to 99.8 % and for the south side calculated to 98.0 %. On the south side of Hallandsås an average sealing effect of 99.5 % was reached after the first grouting and thereby re-grouting was not necessary. It shall be noted that the geological and hydro-geological conditions as well as the grout mix was different between the north- and south side of Hallandsås (Sturk & Nelson 1996).

Table 3-2 Grouting efficiency, average values, from the north side of the Hallandsås Rail Link, performed between February and august 1996, (Sturk & Nelson, 1996).

Number of groutings	Accumulated sealing effect (%)
1	90
2	94
3	97

According to the theory presented in chapter 3.1.1, there are two characteristics of the grouted zone which in most cases affect the water inflow to the tunnel; the conductivity and the thickness of the grouted zone. The conductivity of the grouted zone is dependent of how well the grout penetrates and fills out the discontinuities, which allow water transportation.

In the following some calculation examples are shown for different thickness of the grouted zone in combination with different hydraulic conductivity. The skin factor was set to 3 and the initial rock mass conductivity to $1*10^{-6}$, which resulted in an inflow to the tunnel and is presented in Figure 3-3 and Figure 3-4.

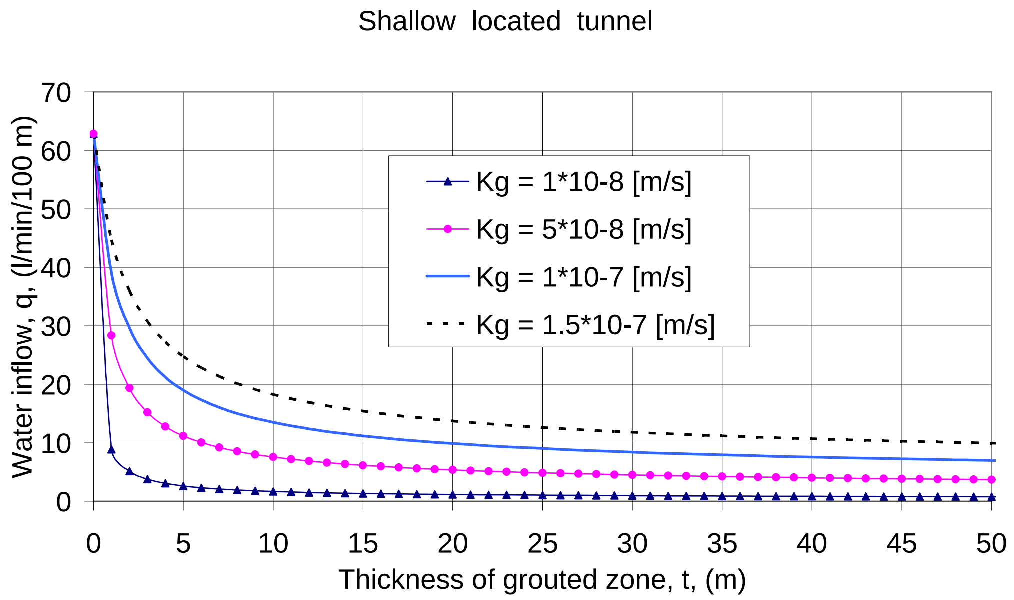


Figure 3-3 Water inflow to a shallow located grouted tunnel as a function of the conductivity and the thickness of the grouted zone, $H = 10$, $R_t = 5$, the inflow after grouting is calculated from Equation 3-3 and an initial conductivity of $1*10^{-6}$ [m/s].

From Figure 3-3 it could be noted that a thickness of 1 meter with a conductivity of $1*10^{-8}$, is equal to a thickness of 50 meters if the conductivity of the grouted zone only is $1.5*10^{-7}$.

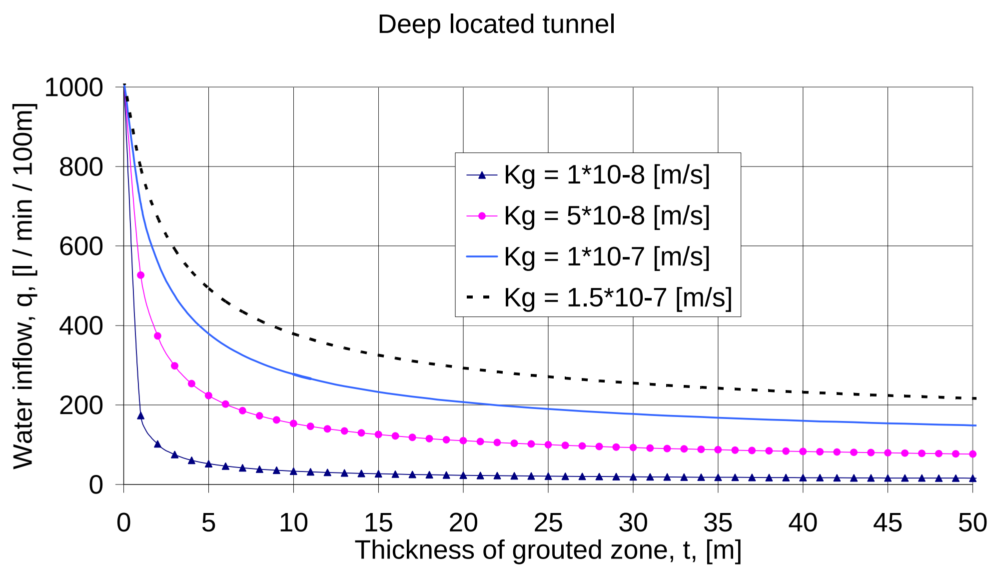


Figure 3-4 Water inflow to a deep located grouted tunnel as a function of the conductivity and the thickness of the grouted zone, $H = 100$, $R_t = 5$, the inflow after grouting is calculated from Equation 3-4 and an initial conductivity of $1*10^{-6}$ [m/s].

From Figure 3-3 and Figure 3-4 it could be noted that the conductivity of the grouted zone is more important than the thickness of the grouted zone. A thickness increase above 10 to 40 meters, dependent on the conductivity, is of very little benefit in reducing the inflow. Further it is noticed that typical inflow requirements will only be possible to achieve with a low conductivity of the grouted zone ($1*10^{-8}$) and that the water pressure has a large influence on the water inflow after grouting. The initial rock mass hydraulic conductivity has a slight influence, but when the hydraulic conductivity of the grouted zone increases the importance of the initial rock mass hydraulic conductivity decreases. Other values of the skin factor will affect the water inflow, but not the fact that the hydraulic conductivity of the grouted zone is the dominating property for reducing the water inflow to a tunnel.

For a tunnel, the thickness requirement of the grouted zone is often based on the rock bolts or the blasting damage zone, the lengths of which are not allowed to puncture the grouted zone. A normal thickness requirement could therefore be a minimum of 3 to 5 meters.

Unfortunately, the true thickness of the grouted zone is difficult to investigate and is not so well examined from practice. However, at the Äspö Hard Rock Laboratory some studies were performed from drilled holes. The spreading of the grout was observed to be between 5 and 30 meters measured from the tunnel periphery (Rhen & Stanfors, 1993).

Lowest achievable conductivity of the grouted zone by cement grouting

According to the Swedish Road Administration (1993), there are experiences from different tunnel projects whereby with normal cement grouting and without additives it is possible to reach a sealing of the grouted zone down to ~ $0{,}5\text{-}1{,}5*10^{-7}$ m/s. The initial rock mass conductivity before grouting was on this occasion not reported. To further seal the rock mass, it was therefore assumed that other grouting techniques using micro cement or chemical grouting products had to be used.

In general, when grouting with cement, a reduction of the initial conductivity by approximately a factor 10 could be expected and the lowest possible expected conductivity could reach $5 *10^{-8}$ - $5*10^{-7}$ m/s according to Kutzner (1996).

From the Stripa project, Pusch et al (1991) demonstrated that it was possible to reduce the hydraulic conductivity from 10^{-8} to 10^{-9} m/s, by using micro cement and superplasticizers in combination with both static and dynamic grouting pressure.

Later studies have pushed the limits further. At the Hallandsås tunnel, grouting with colloidal silica was tested to demonstrate the sealing effect, the hydraulic conductivity could be decrease from initially 10^{-7} to 10^{-9} m/s (Funehag, 2004). At the Äspö Hard Rock Laboratory, grouting trials with cement were performed by Emmelin et al (2004) and the conductivity was decreased from approximately 10^{-7} to 10^{-10} m/s.

According to Kutzner (1996) a rock mass with high conductivity is easy to grout and the sealing effect will be better under these conditions. It should be noted that a rock mass, which is difficult to grout, does not always have to be a rock mass with low conductivity. With equal conductivity, but with different joint frequency, joint distribution or joint connectivity the difference in grouting complexity could be large.

3.2 The rock mass

3.2.1 Introduction

The rock mass is within this project considered as a predefined state of nature parameter with certain characteristics. For certain occasions, when the rock mass is very unfavourable for tunnel excavation or grouting, there is a possibility to move the tunnel to another location with other rock mass characteristics. The option to move the tunnel is beyond the limitations of this project and is therefore not discussed further. In this chapter, the parameters of the rock mass will further be described from a sealing time perspective.

From a rock mechanic point of view the joint aperture, the joint frequency, the joint filling and the direction of the joints are important parameters, which need to be regarded. These parameters are also important for water transport in the rock mass and for the possibility to seal the rock by grouting. The joints can be divided into two groups; the open and the sealed. The open joints can be further divided into two groups, the conductive and the non-conductive joints. A joint is conductive if it is open and connected to an open system, which carries water. The conductive joints can be more or less conductive depending on joint aperture, contact area and different kind of joint filling material. Regards grouting, the effect of these different parameters are more or less complex to describe. For example, the aperture could be measured in a number of points, but the grout aperture pathway will still not be fully known. When grouting a rock mass, the sealing effect could therefore be different even if the rock mass conditions appear to be similar.

From a sealing perspective it is the conductive joints, which are of interest and should be sealed during the pre-grouting process. An excavation or other activities which may cause movements of the rock mass may result in redistribution of stresses in the rock mass. Different joints may then be conductive, which may be one explanation as to why new water transport routes are found some time after excavation.

3.2.2 Rock mass porosity and conductivity

The intact rock is from a grouting perspective not conductive, instead it is the joint system of the rock mass, which is conductive. Frequency studies of conductive joints in relation to the total number of joints were performed in bore holes by Axelsson & Turesson (1994). The studies showed that the frequency of conductive joints was between 5 – 25 %. Another study by Janson (1998), based on (Munier & Hermansson (1994) shown that 8 % of the joints contained water or grout.

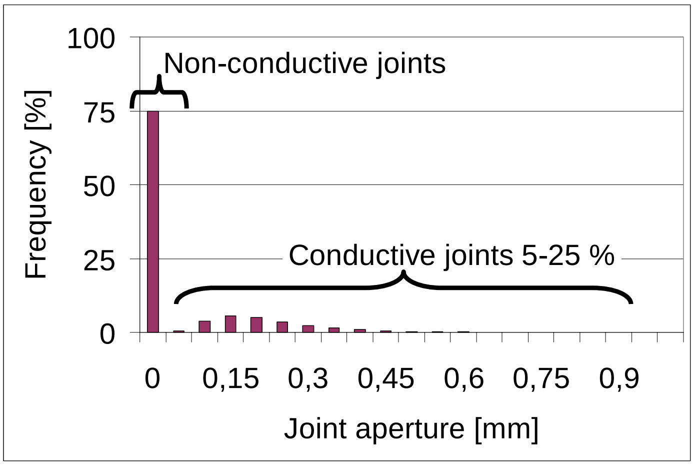


Figure 3-5 Illustration of conductive and non-conductive joints.

The hydraulic conductivity is dependent on the depth and varies normally for Swedish bedrock as shown in Figure 3-6 according to Stille et al (2004).

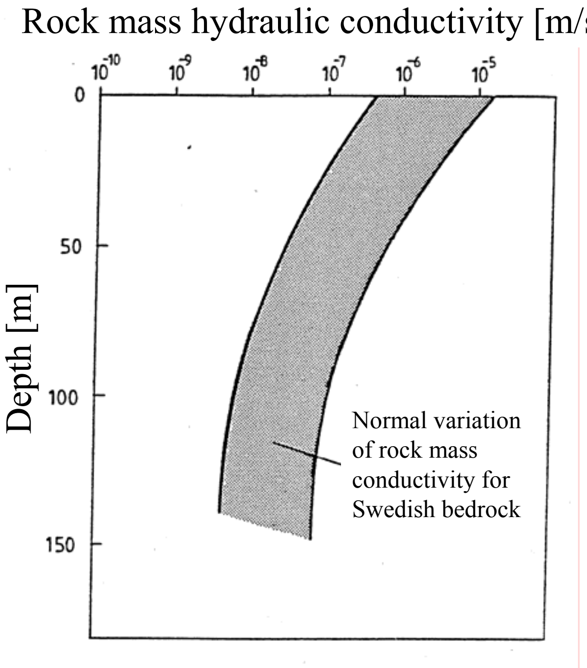


Figure 3-6 Increase of the hydraulic conductivity due to depth (Stille et al, 2004).

If the observed rock mass volume is large, the conductivity of the rock mass can be expressed. If, instead, the rock mass is locally observed the transmissivity is a more appropriate expression. In Gustafson (1986) the number of joints or the representative element volume, REV for hydraulic measurements in a rock mass was calculated. The number is dependent on the sample size and the standard deviation of the transmissivity distribution. Based on a log-normal distribution and a standard deviation of between 1 and 3, the REV was calculated to between 1000 and 10 000 joints. For a more homogenous rock mass, with less standard deviation, the REV will consequently be lower.

In for example Janson (1998), the total transmissivity was calculated as the sum of the transmissivity for the individual joints. If the number is equal to or above the REV, the transmissivity may be expressed as a conductivity according to Equation 3-6:

$$K = \frac{1}{L}\sum \gamma_i T_i$$ Equation 3-6

K = conductivity [m/s]
L = length of borehole [m]
γ_i = number of joints with T_i [-o]
T_i = transmissivity [m^2/s]

In the same way, if a jointed rock mass volume is large enough, the open part of the joint system could be expressed as the porosity of the rock mass. The porosity of the rock mass could also be divided into primary and secondary porosity, were the primary is the "total" porosity and the secondary regards the porosity which is part of the water transport route of the rock mass. The secondary porosity is also called the conductive porosity. For one type of rock mass, a normal value of the secondary porosity could accord to Gustafson (1986), be between 0.001 and 0.1 %.

The correlation between porosity *p* and conductivity *k* has been studied by for example Brotzen (1990). From tracer tests, a correlation was noted as shown in Equation 3-7.

$$\log p = 0.17 \cdot \log k - 1.7 \pm 0.3$$ Equation 3-7

3.2.3 Flow and sealing of one joint plane

In this section, the parameters of a discrete joint plane are discussed, which differs from the previous homogenous isotropic rock mass approach.

The separated joint plane is often described as an opening in the rock. In this plane, there are parts which are more or less in contact with each other. The contact area depends on the rock pressure, to which the joint is subjected (Pyrak-Nolte et al, 1990). A typical value for the area in contact is therefore complicated to estimate. A literature survey by Janson (1998) showed different results of contact area and were found to lie between 1-70 percent. For the possibility to seal a joint with a grout the amount of openness and the distribution of the openness are important. The openness and the direction of the joints should therefore be regarded during design of the grout fan layout. The openness of joint planes expressed as "total potential contact area" has further been studied by Grasselli et al (2002). The "total potential contact area, was found to be dependent of the direction for different types of rock masses.

According to Moreno et al (1988) only 10 % or less of the total joint plane was conductive and according to Abelin et al (1985) between 5-20 % of the joint plane was conductive.

The joint plane geometry is according to NRC (1996) especially dependent on (1) the geologic heritage of the joint; (2) the changes of the tension system, due to natural or human activities; (3) mineral deposits or dispersion due to flow in the joint.

According to Hakami (1995) the joint geometry is characterised by a number of parameters as shown in Figure 3-7. All of them have influence on the fluid flow through the joint, but with varying importance.

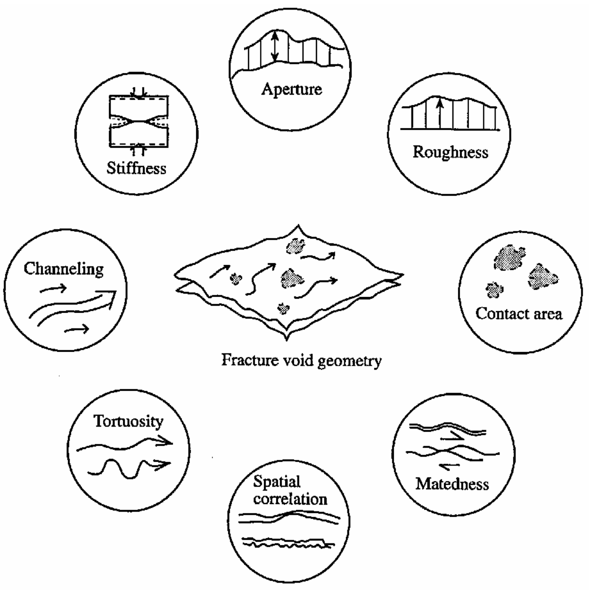


Figure 3-7 Joint properties based on joint geometry (Hakami, 1995).

The joint apertures in a joint plane measured on outcrops by photographing are log-normally distributed (Snow, 1969), (Keller, 1998), (Hakami, 1995).

Fluid flow in a rock mass and in the separated joint plane has been studied in a number of investigations. A popular description is to describe the flow in the joint as canalised, Hakami (1995), Nordquist et al (1992). Large water flows in the rock mass will often occur in the crossing between two joint planes, Hakami (1995), Nordquist (1995).

The water flow in a joint plane is dependent of a number of parameters whereof the joint aperture is the most important. In a study of groutability and hydrogeological data by Gustafson & Stille (1996), it was shown that the standard deviation of the aperture distribution is very important. The total transmissivity may due to a high standard deviation, be totally dominated of a single joint.

The ”Cubic law” which is valid for laminar flow between two parallel plates describes the flow Q as shown in Equation 3-8, where i is the hydraulic gradient.

$$Q = -\frac{\rho_W g}{\mu_W}\frac{Wb^3}{12}i \qquad \text{Equation 3-8}$$

Where:

ρ_w	density of water	[kg/s^3]
g	acceleration due to gravity	[m/s^2]
W	width of the channel	[m]
b	aperture of the joint	[m]
μ_w	viscosity of water	[N/m^{2}s]
i	hydraulic gradient	[-]

Based on Gustafsson (1986) the transmissivity T [m^2/s], for one intersected joint in a borehole could be calculated as:

$$T = \frac{b_{hyd}^{\ 3} \rho_w g}{12 \mu_w f} \qquad \text{Equation 3-9}$$

Where f is the roughness and is set equal to 1, and b_{hyd} the hydraulic joint aperture.

The hydraulic aperture b_{hyd} has by a many authors been found to be smaller than the arithmetic mean aperture b. This implies that the porosity calculated according to b_{hyd} will be less then the actual porosity.

The introduction of the transmissivity will simplify the ”Cubic law” equation, which then could be written as:

$$Q = -WTi \qquad \text{Equation 3-10}$$

The flow in a joint plane is according to Equation 3-10 proportional to the transmissivity.

The transmissivity is in addition strongly correlated to the effective stress. According to Alm (1999), the overall transmissivity of a joint was increased by a factor 20 when the effective stress decreased by 0.8 MPa.

The grout can penetrate and fill out the joint planes in many different ways. Typically the flow in a joint plane has been described as canalised, which means that the grout penetrates the plane in certain channels with apertures larger than in surrounding areas, as examplified in Figure 3-8.

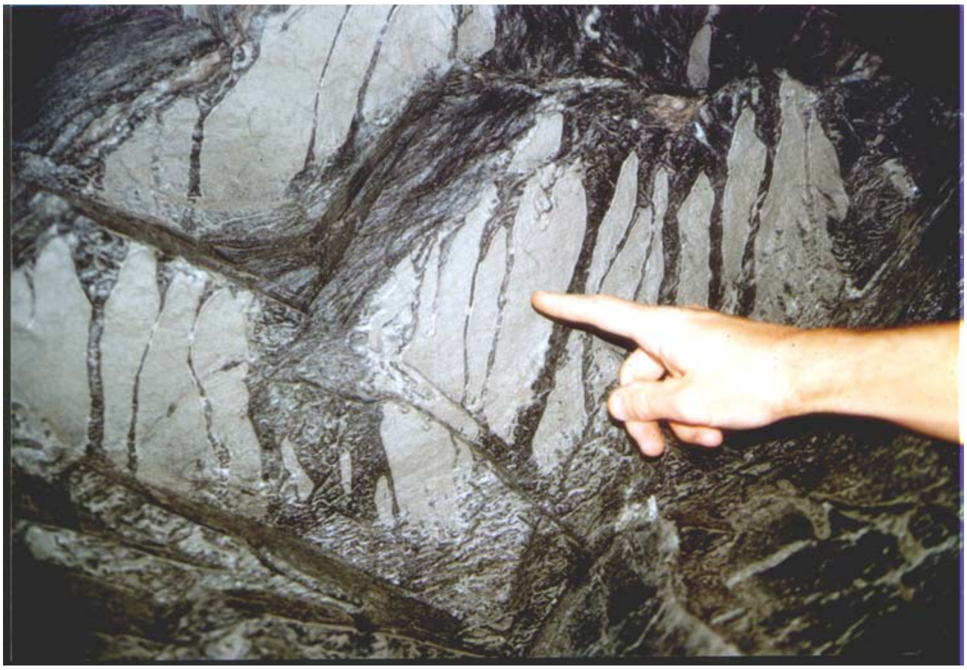


Figure 3-8 A example of two joint planes which have been grouted. The bright areas are grout and the dark areas are rock mass.

The joint channel where grout flow takes place was modelled by Pusch et al (1991) as a generalised rhomboid. In the model the channels were subdivided into wide and narrow channels. The wide channel shown in Figure 3-9 with an aperture of at least 50 μm can in principal be filled out with grout. The narrow channel with an aperture less than 50 μm will only be partly filled. The experiments were performed with Alofix cement, d_{max} 15 μm, and with an addition of 1,4 % Mighty 100 superplasticizer.

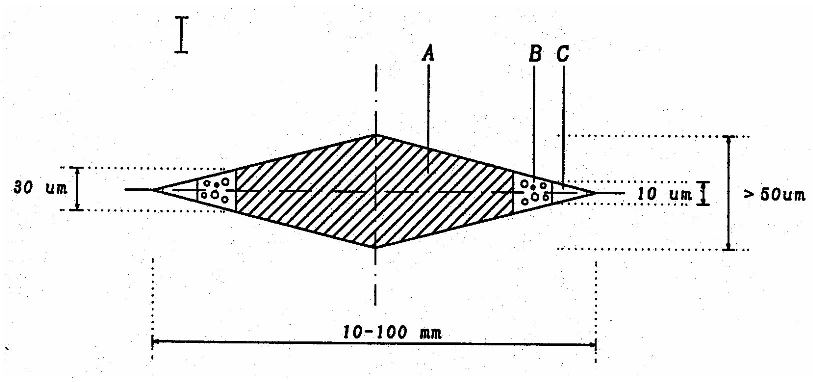


Figure 3-9 A wide channel (>50 μm) filled with grout. A is completely filled, B is partly filled and C is an unfilled area (Pusch et al, 1991).

To seal a joint plane with grout will always leave some parts unsealed even if the aperture is wide enough for penetration. This is because of small aperture, which will restrict the grout from reaching certain areas of each channel as illustrated in Figure 3-9. The result will then be a partial sealing of the joint, where the unsealed parts can be open for water flow even after grouting.

3.2.4 Sealing of a system of conductive joint planes

The water transport in a rock mass is directed to the conductive part of the conductive joint planes. To seal all conductive joints of a rock mass and thereby restrict water inflow to a tunnel can be difficult. The single most important parameter affecting the possibility to seal the joints in a rock mass is the joint aperture, since it restricts the grout propagation in the joint.

The joint aperture distribution for a number of joint planes in a rock mass is not so well studied. A good assumption is that they vary as a separate joint plane, with a lognormal distribution and a part of non-conductive or tight areas, Nordquist et al (1992).

Based on studies from Äspö Hard Rock Laboratory, Munier (1992) concluded that the joint length follow a lognormal distribution with mean, mode and standard deviation estimated to 2.84 m, 1.52 m and 1.43 m, respectively. The joint frequency was also evaluated for different joint sets and varied between 0.32 m $^{-1}$ to 8.45 m $^{-1}$.

If the "cubic law" model as described in equation 3-10 is assumed to be representative for flow of water in a joint plane, the flow to a tunnel can be calculated. The flow is dependent of the joint aperture and can be calculated with Equation 3-11, which is the "cubic law" equation complemented with the number of joints, were γ_i is then the number of joints with the same aperture and summation is done for all apertures.

$$Q = \sum(-WT_i i \cdot \gamma_i) \qquad \text{Equation 3-11}$$

The relation between the flow and the aperture, based on the medium aperture distribution from Figure 3-12 is shown in Figure 3-10. This illustrates that the flow rate reaches a maximum when the aperture is around 2 times the average aperture of 0.2475 mm, which is an effect of aperture and the number of joints.

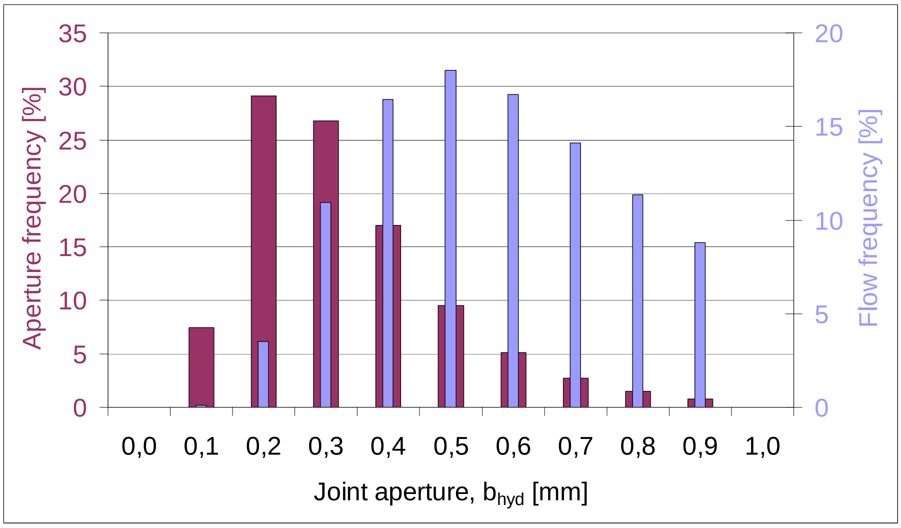


Figure 3-10 Illustration of flow as a function of joint aperture.

The medium aperture distribution shown in Figure 3-12 is assumed to be only partly sealed, due to the channel geometry. The parts of the joint planes with a width over a certain value have been sealed and parts with a width under this certain value are open for flow.

In Figure 3-11 it is shown that joints down to an aperture of approximately 0.2 mm, based on partly sealing of the joints, has to be sealed to reach a flow reduction of more than 91.3 %.

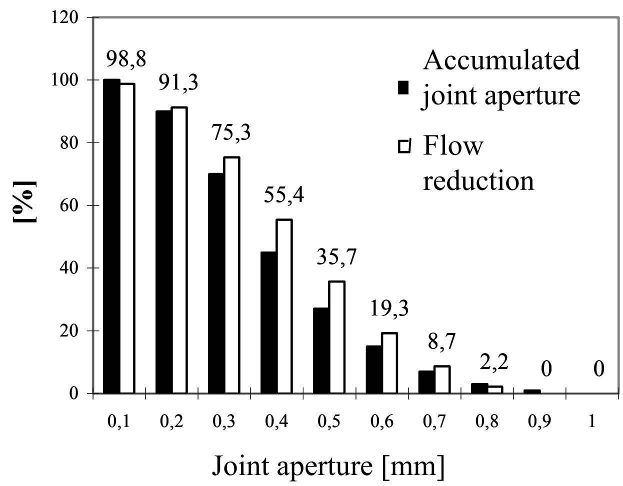


Figure 3-11 Flow reduction by partially sealing of joints down to a certain aperture.

To further discuss this issue regarding the relation of ingress of water and the demands on sealing of certain joints, a calculation example is here presented.

In Table 3-3 the properties of three different rock mass joint distributions are shown regarding the joint frequency, the porosity, the minimum and maximum aperture, which together results in a hydraulic conductivity.

Table 3-3 Three joint distributions.

Distribution	Maximum aperture [mm]	Minimum aperture [mm]	Number of joints/ m [n/m]	Theoretical porosity [%]
Medium	1	0.1	0.1	0.0036
Fine	0.5	0.05	0.8	0.0143
Wide	2	0.2	0.0125	0.0009

The aperture distribution varies between the rock masses. The first one consists of medium apertures, the second of fine apertures and the third of wide apertures. The hydraulic conductivity have been set equal for the three distributions ($5.2*10^{-6}$), which means that the number of joint is greater for the fine distribution than for the wide distribution, as shown in Figure 3-12.

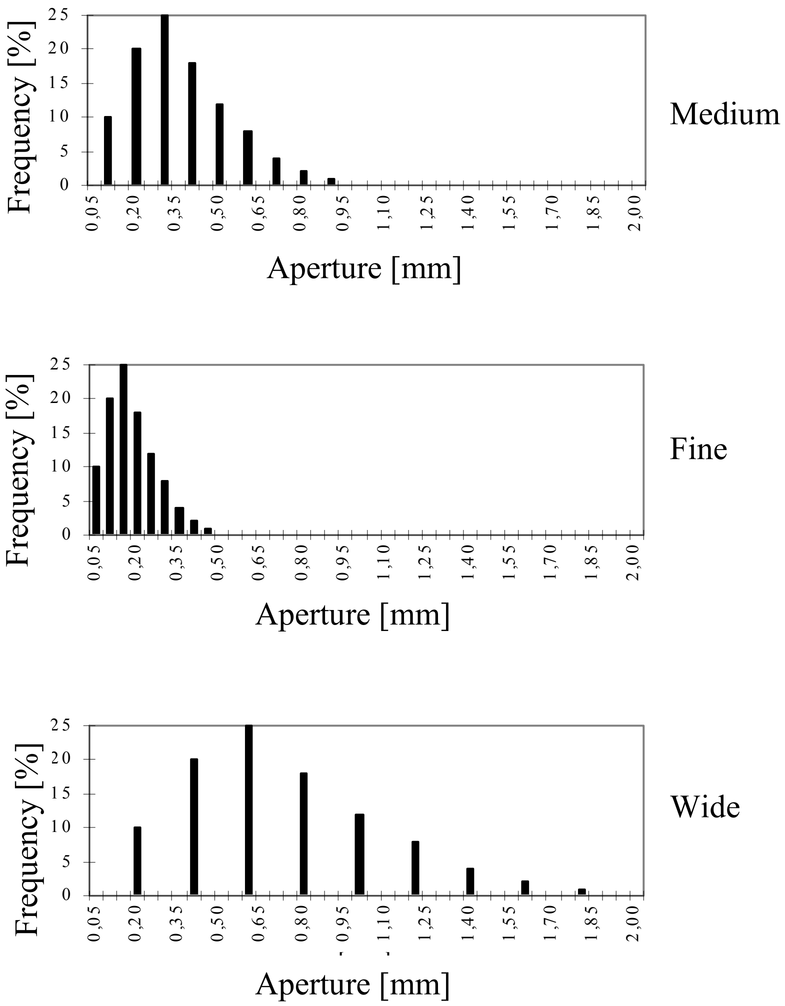


Figure 3-12 Aperture distribution for three different rock masses.

The calculated theoretical porosities could be compared to the secondary porosity, which according to Gustafson (1986) is usually between 0.001 and 0.1 %. From tracer test, Brotzen

(1990) found that the porosity for a measured conductivity was higher than the theoretical porosity.

Rock mass properties are different from site to site and consequently the possibilities to seal them also differ. The joint situation, defined in Figure 3-12, is here illustrated by some calculation examples. In Table 3-4 the different distributions have been sealed down to equal conductivities corresponding to 90 % and 99 % sealing effect. To receive equal conductivity different apertures have to be sealed for the different distributions. Within the calculations, partial sealing of the joints are assumed. Partial sealing has within these calculations been regarded as: if a grout can penetrate joints down to 0.2 mm, the joints with smaller aperture will after sealing be unsealed and the joints with larger aperture will after sealing have a remaining aperture of 0.2 mm.

Table 3-4 Flow reduction for the different joint distributions with partial sealing down to certain apertures.

Joint distribution	K, initial [m/s]	b to reach 90% reduction [mm]	K (90%) [m/s]	b to reach 99% reduction [mm]	K (99%) [m/s]
Medium	$5.3*10^{-6}$	0.22	$5.8*10^{-7}$	0.10	$6.3*10^{-8}$
Fine	$5.3*10^{-6}$	0.11	$5.8*10^{-7}$	0.05	$6.3*10^{-8}$
Wide	$5.3*10^{-6}$	0.44	$5.8*10^{-7}$	0.20	$6,3*10^{-8}$

To reduce the flow will as seen in Table 3-4 be more or less difficult dependent on the present aperture distribution. The “fine” and the “wide” joint systems have the same conductivity both before and after sealing, but the possibility to seal, will probably be much better for the “wide” distribution, which only have to seal joints down to 0.20 mm to reach 99 % flow reduction, which should be compared to joints down to 0.05 mm for the “fine” distribution.

A theoretical study based on the generalised rhomboid model, by Pusch (1993), showed that if the channels do have the same conductivity, then a higher sealing effect would be reach if the channels were few and wide instead of many and narrow. The distributions used by Pusch (1993) were similar to those in Figure 3-12.

The grout take, for the three different joint distributions, are presented in Table 3-5. The volume for V_{90} and V_{99} has been calculated for a 10 meters long grout hole and a 10 meters radial grout spread perpendicular to the hole. In Table 3-5 it is shown that the grout volume is related to the porosity, with high grout volumes for many small joints and small grout volume for few wide joints.

Table 3-5 Calculated theoretical grout take when assuming partial sealing and that all joints are sealed down to a certain aperture.

Joint distribution	Theoretical porosity [%]	Volume grout at K (90%) [m^3/m^3]	V_{90} [l]	Volume grout at K (99%) [m^3/m^3]	V_{99} [l]
Medium	0.0036	0.0015	60	0.0026	104
Fine	0.0143	0.0062	248	0.0103	412
Wide	0.0009	0.0004	16	0.0007	26

It is the opinion of the author that the real porosity may by higher then stipulated by the plane parallel model used here, as illustrated in Figure 3-13. This may lead to higher real grout volumes than the calculated theoretical volumes presented in Table 2-6.

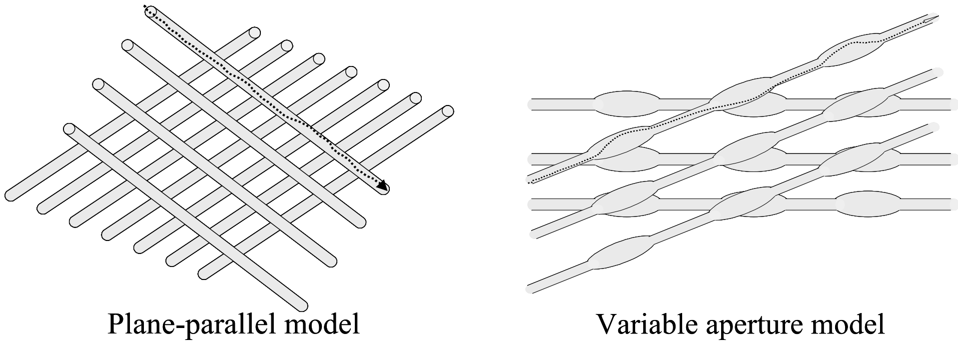


Figure 3-13 The calculated porosity varies with the assumed conceptual model.

The flow geometry, which is further described in chapter 3.2.3 is also effecting the possibility to seal a conductive joint system according to Gustafson & Stille (1996). For a two-dimensional joint model the grout can spread radial from the grout hole, but for the three-dimensional model, the radial spread is intercepted by channels sub-parallel to the bore hole. The pressure difference might then be too small to enable grout spread into the sub-parallel joints, resulting in unfilled or partially filled connecting channels. This emphasizes the importance of surveying the joint directions and adjusting the grouting method for the present rock joint situation.

The experience from grouting at the Äspö tunnel was that a single open two-dimensional joint may easily be grouted, while zones with more joint systems, which are equal to the three-dimensional joint model often, require several groutings to be sealed. Or as stated from Gustafson & Stille (1996), “This finding supports the theoretical consideration that the grout flow cannot easily fill a three-dimensional joint system from a single borehole”

3.3 Probing

Probing in the form of rock and joint investigations are performed; before, during and after a grouting project.

The purpose of the investigation before the grouting is to examine the possibility of grouting and the possible grouting methods. During the project the earlier investigation is updated and in addition the investigation can have the purpose to optimize the grouting method. After the grouting project probing is mainly carried out to measure the result of the grouting.

3.3.1 General rock and joint investigations for grouting

Before a grouting project is started, it is advisable to examine and determine the origin of the rock mass and how the rock mass and the joint system have been developed. As for a tunnel project the examination is carried out to get a perspective of the rock mass and the joint systems of the rock mass. The questions that need to be answered are for example:

- How has the rock mass been developed
- Is the rock mass transferred in any direction
- Where are the joints
- Why and how have the joints occurred
- Which joints will occur and be closed in the future
- Which stresses and joints may occur because of this project

The general survey of the rock mass to answer the above questions, could start with a visual observation of outcrops and cleaned rock surfaces. Thereafter holes could be drilled and inspected. Core drilling is common for general geology investigations, but for grouting the method has limited value. Joints widths are difficult to measure and joint directions cannot be determined, unless cores are oriented. Therefore ordinary drilling combined with water pressure tests and borehole TV could give valuable information for grouting.

Investigation of the rock mass for grouting has been described by a number of authors as for example Houlsby (1990) and Kutzner (1996).

Houlsby (1990) recommends an investigation of the grouting site concerning the following features, which can influence the design and construction of the grouting:

1. *Spacing of Joints*. If joints are widely spaced, the grouting is usually easier than if closely spaced.

2. *Joint Widths and Continuity*. The easiest joints to grout have widths in a range between about 6 mm and 0.5 mm.

3. *Joint Inclination*. The grout holes shall intercept the joint sets as much as possible.

4. *Uniformity of the Site*. Uniformity of grouting allows a regular layout of grout holes, whereas irregular jointing and disconformities may require placement of holes at various inclinations and spacing.

5. *Rock Soundness*. If the grouting holes are not stable, they may need to be shortened into sections and/or grouted through fibre glass pipes.

6. *Strength*. Grouting of strong, massive well anchored rock is usually easier than when the rock mass is weak.

7. *Stress in Rock*. If a rock mass contains tectonic stress, the grouting will need to take account of this.

8. *Piping*. Where material in joints can be removed by seepage, either by taking the material into solution or by eroding it, the grouting will need to be more intensive than otherwise.

9. *Chemical Attack*. The presence of material that may provide chemically aggressive seepage, as for example coal or other carbonaceous material, may require a higher standard of grouting.

10. *Karst and other Voids*. Large voids such as karst, and also old mines, shafts, and so on, require special provisions when grouted.

The joint system is important as it makes up the transport routes in the rock mass. The joint system could in different ways be described by the permeability and the porosity of the rock mass. Permeability measurements in the rock mass are therefore important information for grout work design.

In the next chapter the permeability of the rock mass and the joints are discussed based on water loss measurements.

3.3.2 Investigating of the rock mass joint system

The rock mass can be investigated by different test methods in drilled holes. The different methods shall be chosen and evaluated depending on the expected conductivity (Gustafson & Stille, 1996).

Hydraulic tests can be performed to indicate the permeability for the zones, which should be grouted. The test then gives valuable information for: grout mix proportioning, grout order, grout take and connections between grout holes as well as possible surface leakage.

In Gustafson (1986), ISRM (1996), Houlsby (1990) and Fransson (1997) some methods for hydraulic tests in drilled holes are described further.

Hydraulic tests could be performed hole by hole or in a number of holes simultaneously. When tests are performed in a number of holes simultaneously, so-called interference tests, information about the total hydraulic conductivity for the rock mass could then be measured (Gustafson, 1996).

The interpretation of a hydraulic test is dependent on the flow geometry i.e. one, two or three dimensions (Håkansson & Hässler, 1993). To be able to evaluate the test the dimension needs to be known, which could be done through transient tests (Fransson, 1999):

- Flow in one dimension could be expected if the joint is oriented as the drill hole.
- Flow in two dimensions could be expected if the joint is perpendicular to the drill hole.
- Flow in three dimensions could be expected if there is a joint system with high connectivity.

The evaluated value will vary depending on the method of interpreting the packer test. The difference could be as high as a factor 10 between different methods of evaluation, (Gustafson 1986, Andersson 1994).

Different test methods to investigate are shortly described here:

<u>Pressure building tests</u>

After drilling a test hole, the water is let out. The hole is closed with a packer and a gauge placed for pressure measurement. The hydraulic conductivity is evaluated from the pressure increase. A pressure building test is suitable for rock masses with low hydraulic conductivity (Gustafson, 1986).

<u>Pulse tests</u>
A pulse is discharged and the restoration of the pulse is evaluated. The pulse tests are suitable for low to moderate conductivity's (ISRM, 1996).

Packer tests

Packer tests are performed in situ by sealing off the opening to a drill hole by a sleeve and pumping water under constant pressure. Packer tests are also called Water loss measurements or water pressure tests. The flow is registered and the conductivity can be evaluated. For a single packer test, the measured length can be varied by placing the packet at different distances from the bottom as shown in Figure 3-14.

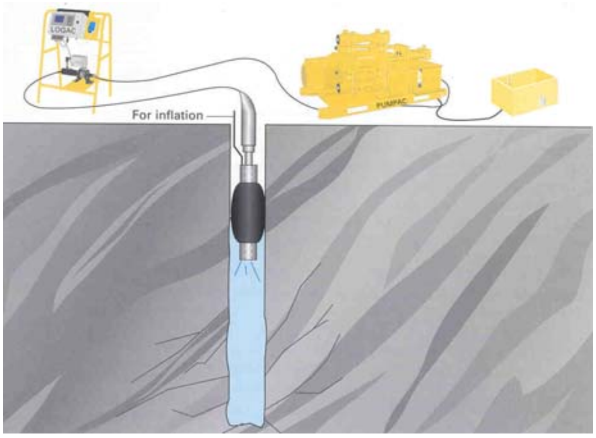


Figure 3-14 Single packer testing (Pettersson et al, 1999).

For packer tests with a double packer the test area can be adapted for the certain area of interest by varying the length between the packers, as shown in Figure 3-15.

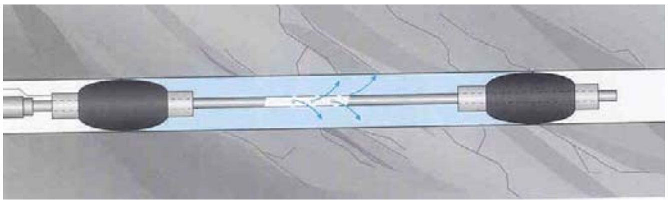


Figure 3-15 Double packer testing (Pettersson et al, 1999).

The flow is measured and the test continues until stable conditions have been reached. The packer tests are suitable for moderate to high conductivity rock masses (Gustafson, 1986). According to Swedish Road Administration (1993), the water loss measurement is performed at a pressure of 0.5 MPa above the ground water pressure and continued for at least 4 minutes after steady conditions have been received. For different situations other pressures and test times could also be applicable. The water loss is then calculated as:

$$V_f = \frac{q}{(L-b)\cdot p_o} \qquad \text{Equation 3-12}$$

Were,

q = water loss	[litres/minute/MPa]
L = length of hole	[m]
b = distance from tunnel wall to sleeve	[m]
p_0 = pressure above ground water table	[MPa]

The definition of one Lugeon is:

1 Lugeon unit= 1 litre of water taken per meter of test length, per minute, at 10 bars pressure.

In Houlsby (1990) the Lugeon is expressed according to Equation 3-13. The actual pressure is then not clearly defined. For a packer test in a tunnel, the applied pressure needs to be compensated for the water pressure and the Lugeon value is calculated by using the overpressure.

$$Lugeon\,[units] = \frac{Q\,[litrers / meter / Minute] \cdot 10\,[bars]}{P_{actual\ pressure}\,[bars]}$$ Equation 3-13

For surface testing, the actual test pressure is often less than Lugeon's defined pressure of 10 bars because it may be too high for many foundation conditions. For underground testing pressures higher than 10 bars may be necessary to be able to carry out the test. A straight-line relationship between the actual pressure and the defined pressure is then assumed, which is not strictly correct but may for practical purpose be an acceptable assumption (Houlsby, 1990).

A Lugeon or water pressure test must never be used on their own without some idea of the crack sizes and spacing to which they apply (Houlsby, 1990).

To perform a packer test in stages with different constant head is recommended by authors as for example Nonveiller (1989) and Houlsby (1990). This type of test is not common as test during tunnelling, as the available time to carry out tests usually is very limited. The test consists of five stages in a sequence: low pressure, moderate pressure, peak pressure, moderate pressure and low pressure. Each stage takes 10 minutes and the flow should then be plotted in a graph, which enables an evaluation. The relationship between pressure and flow can then indicate if the flow was laminar or turbulent. The relationship can also indicate the occurrence of joint dilation, wash-out of joint filling material or filling up of voids.

In Houlsby (1990) a stage test method is recommended as:

1. Apply water at a **Low** pressure for 10 minutes.
2. A **Moderate** pressure for the next 10 minutes.
3. A **Peak** pressure for the next 10 minutes.
4. The **Moderate** pressure again for the next 10 minutes.
5. The **Low** pressure for the final 10 minutes.

The five pressure runs should be performed directly after each other. The Lugeon value is then plotted for each pressure, as shown in Figure 3-16. The five values are then inspected to determine the flow regime or testing event. In Figure 3-16 the pressure flow relationship is shown for a typical laminar and turbulent response. As shown the flow is independent of the pressure for laminar flow and dependent of the pressure for turbulent flow.

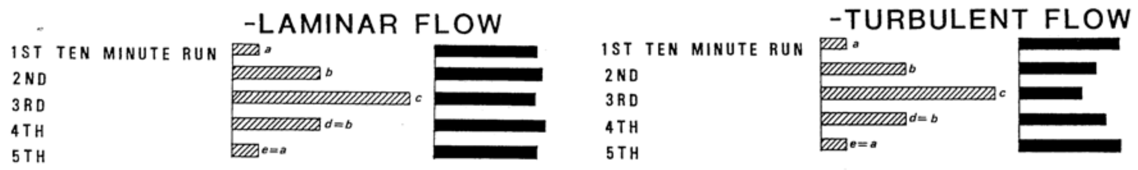


Figure 3-16 The pressures are shown as the left bars and the corresponding Lugeon values are shown as the right bars for both laminar and turbulent flow (Houlsby, 1990).

<u>Evaluation of packer tests</u>

If water is supplied through a packer to a long hole the flow will be distributed radial around the hole. The hole can then be approximated with a thin line according to Figure 3-17. The process is reversible, which means that it is valid for pumping out as well as for injecting to the hole.

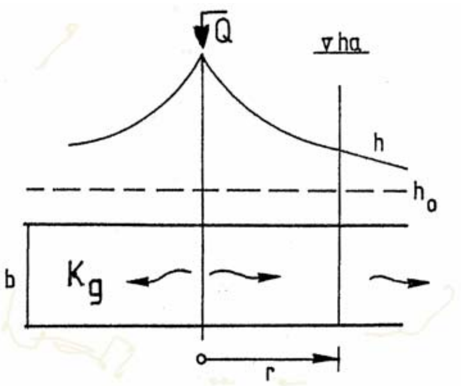


Figure 3-17 A drill hole considered as a line well (Gustafson, 1986).

Within the geo-hydrology the Thiems well equation is of great importance when studying two dimensional processes. The Thiems Equation for flow *Q* could be noted as:

$$Q = \frac{2 \cdot \pi \cdot K_g \cdot b \cdot \Delta h}{\ln\left(\frac{R_0}{r_w}\right)} \qquad \text{Equation 3-14}$$

Were Δh is the ratio between the pressure level at the test level during pumping and the initial pressure. K_g is the conductivity of the tested layer. R_0 is the influence area and r_w is the radius of the well and *b* is the thickness of the tested layer.

Depending on whether the flow to the well is considered as a line well or between packers the evaluation will be different. For a line well the conductivity could be evaluated according to a rewritten Thiems equation, with $\Delta h = \Delta p * \rho_w * g$, for a homogenous material based on Gustafson, (1986) as:

$$K_g = \frac{Q \cdot \rho_w \cdot g}{2 \cdot \pi \cdot b \cdot \Delta p} \cdot \ln\left(\frac{R_0}{r_w}\right) \qquad \text{(line, homogenous)} \qquad \text{Equation 3-15}$$

If an area in a hole is confined between two packers and water is pumped to the area. Then the conductivity, based on Gustafson (1986), for the confined area could be calculated for a homogenous material as:

$$K_g = \frac{Q \cdot \rho_w \cdot g}{2 \cdot \pi \cdot L \cdot \Delta p} \cdot \ln\left(\frac{L}{r_w}\right) \qquad \text{(packer, homogenous)} \qquad \text{Equation 3-16}$$

Where:

Q	flow	[m^3/s]
ρ_w	density of water	[kg/m^3]
g	acceleration due to gravity	[m/s^2]
b	thickness of tested layer	[m]
Δp	pressure difference	[Pa]
R_0	influence area	[m^2]
r_w	radius of the well	[m]
L	distance between the packers	[m]

A packer testing, in a hole with only one joint, confined between the two packers is shown in Figure 3-18.

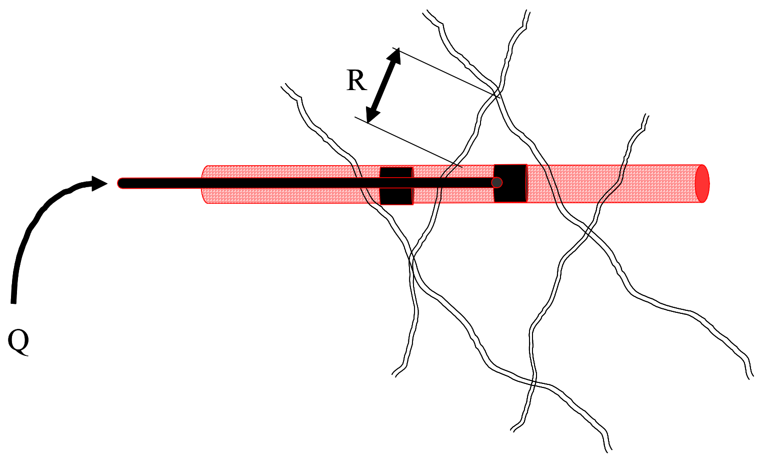


Figure 3-18 A single joint confined by two packers and subjected to pumping.

The expression for the conductivity of one joint could, based on Gustafson (1986), be noted as:

$$K_g = \frac{Q \cdot \rho_w \cdot g}{2 \cdot \pi \cdot R \cdot \Delta p} \qquad \text{(packer, one joint)} \qquad \text{Equation 3-17}$$

Where, *R* is the distance between the hole and the area unaffected by pressure.

The purpose of a packer test can be to estimate the joint aperture distribution to be subjected to grouting. This is evaluated indirectly through measuring its transmissivity. The in situ transmissivity [m^2/s] of a specific joint can, based on Gustafson (1986) and Equation 3-16, be evaluated according to:

$$T = \frac{Q \cdot \ln\left(\frac{L_b}{r_w}\right)}{2 \cdot \pi \cdot \Lambda h}$$ Equation 3-18

Where,

T	Transmissivity	[m^2/s]
Q	flow of water	[m^3/s]
Δh	pressure ratio	[-]
L_b	length of bore hole	[m]
r_w	radius of the well	[m]

Then the hydraulic joint aperture b_{hyd} [m] is calculated according to

$$b_{hyd} = \left(\frac{T \cdot 12 \cdot \mu_w}{\rho_w \cdot g \cdot N_w}\right)^{1/3}$$ Equation 3-19

Which is a re-writing of Equation 3-9, where,

ρ_w	density of water	[kg/m^3]
g	acceleration due to gravity	[m/s^2]
μ_w	viscosity of water	[N/m^2s]
N_w	Number of joints with equal aperture	[-]

If the hydraulic aperture b_{hyd} should be expressed as physical aperture b an empiric relationship was proposed by Zimmerman et al (1991) according to:

$$\left(\frac{b_{hyd}}{b}\right)^3 = 1 - 0.9e^{-0.56\frac{b}{\sigma}}$$ Equation 3-20

The equation regards the finding that an increasing difference between the hydraulic aperture and the physical aperture is found as the standard deviation σ increases.

As an example the hydraulic aperture from 279 measurements at the South Link is shown in Figure 3-19. The measurements were performed in grout holes with different lengths. The number of conductive joints was studied and the evaluation of apertures assumes that the aperture of the conductive joints in each section is equal. The number of conductive joints per measured section was between 2 and 4 (Dalmalm et al, 2000).

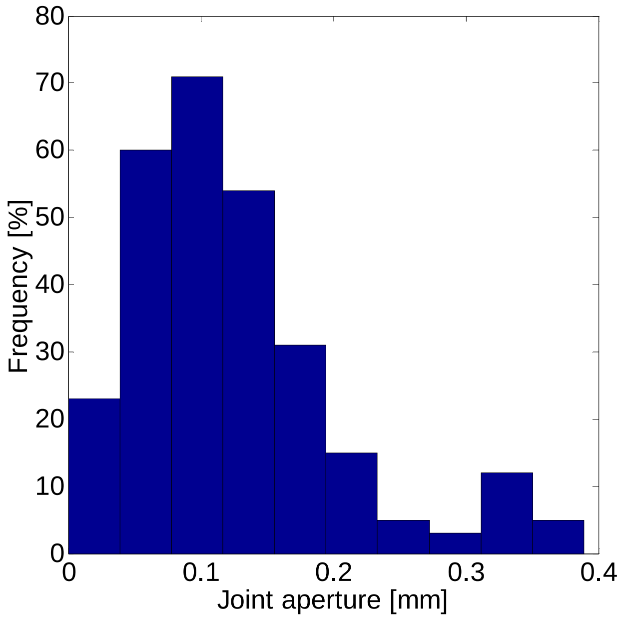


Figure 3-19 Hydraulic joint aperture distribution (b_{hyd}), 279 measurements (South Link).

3.3.3 Direct investigation of the aperture

As the aperture is an important parameter for the possibility to seal a rock mass, many investigations are done, in order to get an idea of the aperture distribution. The aperture can be measured directly, or indirectly through, for example, packer tests as described above.

The direct measurements can, for example, be done by using video imaging, which of course only provides an image of the joint aperture in the immediate vicinity of the drill hole. In addition, the aperture is most likely affected by the boring process. From computer aided tomography (CAT) X-ray scanning of three rock mass samples, Keller (1998) measured a joint aperture distribution correlation length of between 0.8 and 7 mm. This gives us an idea of the limitations with single aperture measurements in a joint plane. At present geophysical imaging methods such as; seismic reflection, electrical methods and ground penetrating radar are only successful in detecting very large features, with resolution, at best, of the same order as the dominant wavelength of the input signal for radar and seismic imaging 0.1 to 10 meters, and for electrical profiling 1 to 10 meters. Instead indirect techniques as pumping and tracer injection tests are currently the most accurate means of assessing joint aperture and its distribution in field (Keller, 1998).

3.3.4 Investigation with a borehole camera

A borehole camera can be used to examine the rock mass. There are mainly two types of cameras, one that view the hole axially and the another which has a mirror mounted in front of the camera lens, which provides a view perpendicular to the borehole axis.

During the grouting field trials at South Link, Stockholm, an axially viewing borehole camera was used. Some experiences from the use of the camera are therefore reported here. In the boreholes, it was possible to count the joint intensity and even to get an idea of apertures, but only for apertures above approximately 1 mm. The strike and dip of joints could be identified in holes, which had running water or drilling debris at the bottom of the hole as a vertical orientation. The visibility depends on the lights of the camera as well of the rock type colour. A bright type of rock mass gives good visibility and a dark type of rock mass gives poor visibility. In holes with running water, it could be difficult to exactly identify the conductive joints. The flow of water could more commonly be identified to a section of approximately 0.3 meters. This corresponds well to that found in a study by Gustafson & Stille (1996), during water pressure tests at the Äspö Hard Rock Laboratory. The examination of drill holes typically gave one open joint and some diffuse conductors as shown in Figure 3-20.

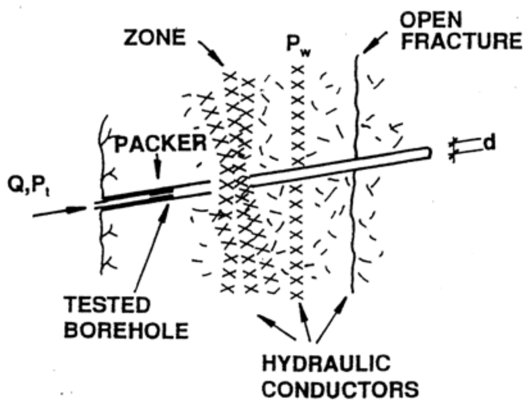


Figure 3-20 A hydraulic conductor could both be an open fracture or a conductive zone, (Gustafson & Stille, 1996).

3.3.5 Field investigation of grouting result

The result of grouting could be measured by a number of methods. Activities in the tunnel or rain fall etc., need to be regarded, as it may effect the investigation of the grouting result.

Some methods that could be used for measuring the grouting result are as follows:

- Measure for water loss in tunnel face by observation holes.
- Surveying drip and wet spots on tunnel face (easiest to identify on a shotcreted surface).
- Measuring overflow of section dams in tunnel (commonly one dam / 100 meter of tunnel).
- Measuring the ground water table in wells.
- Measuring pumped water from the drainage system for different sections.
- Drill holes perpendicular to tunnel and measure the spreading of grout, by sectioned water loss measurements.
- Radar, before and after grouting.

A common method is to measure the inflow or water loss measurement in a number observation holes in the tunnel front after grouting but before excavation. Depending on the tunnel size and the requirement, commonly between 2 to 10 observation holes are measured for water loss. If the water loss value is higher than the requirement, re-grouting is carried out. The value may be set for each individual hole or as an average for all observation holes.

The correlation between inflow in the control, holes and the total inflow to the tunnel is highly dependent on where the control holes hit. As the rock mass is jointed by a discrete joint system, one observation hole could hit the conductive part of the conductive joint, but another observation hole may hit in the closed part of the same conductive joint, which will give different results. As noted in chapter 3.2.2, only a small part of each joint plane is open for flow, which for one observation hole gives a 80 to 95 % chance to receive a no re-grouting decision even if the right decision may be to re-grout.

If water loss measurements are performed in observation holes at the tunnel face, the result can be analysed regarding the distance between the holes and the measured water loss by variance analysis. The variance analysis is used to interpret a correlation model, which is used to perform an ordinary kriging analysis. From a case study at the South Link (Dalmalm et al, 2000), based on water loss measurements in 42 observation holes, the correlation distance is shown as a variogram in Figure 3-21. The variogram was also used to perform an ordinary kriging as shown in Figure 3-22.

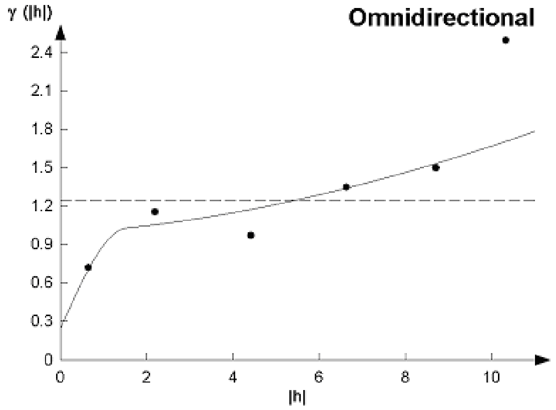


Figure 3-21 Variogram of Lugeon values from control holes at a tunnel face, (Dalmalm et al, 2000).

From the variogram it could be noted that there is a correlation between two spatial measurements of the water loss if the distance is less than ~3 meters. The variogram can then, for example, be used to estimate the hole intensity to hit the conductive part of a joint plane.

The variogram has then been used for to perform an ordinary kriging as shown in Figure 3-22. If the 20 meters long observation holes intercept one joint plane, the interpretation of the kriging could be that two major joint directions, one vertical and one with ~60 degrees angle crossing the vertical joint are recognised. The spatial variation in the figure is also shown to be large, which needs to be regarded when, for example, using observation holes as a method of measuring the sealing effect. If instead, the observation holes intercept more than one joint plane, the interpretation will be more complicated, which is not further developed within this work.

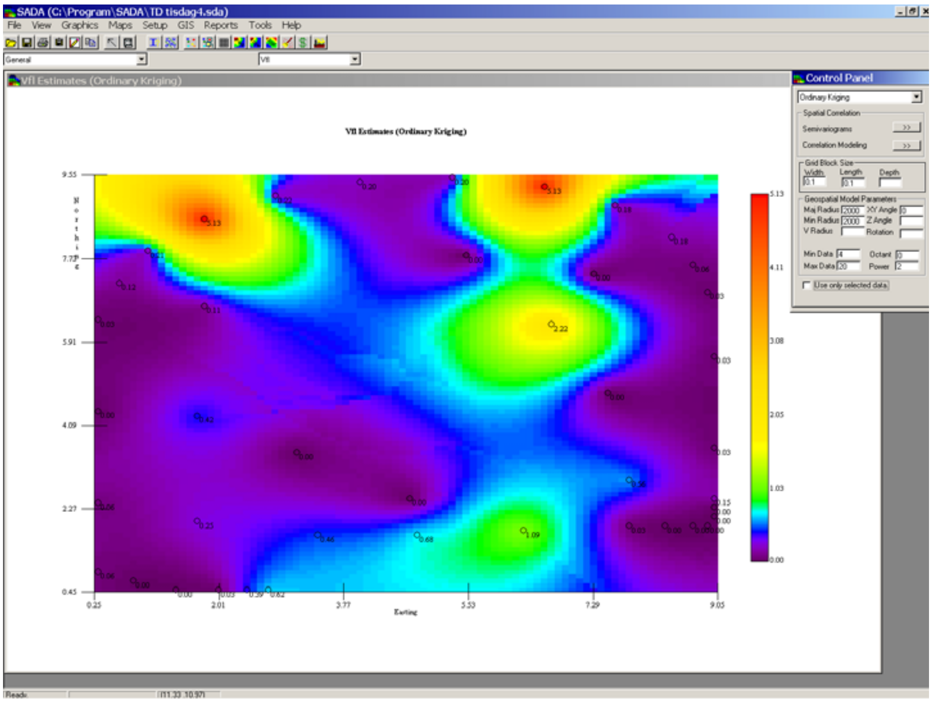


Figure 3-22 Correlation calculated with ordinary Kriging based on 42 water loss measurements for a 9 by 9 meter tunnel section using the software SADA, Dalmalm et al (2000).

In Figure 3-22 the dark purple areas dominate with a Lugeon value below 0.22. The colours are further categorised as: dark blue ~0.4, light blue ~0.6, green ~1, yellow ~2, orange ~3 and red is >5 LU.

Another method, which has been used for decades, to measure the grouting result is to build dams in the tunnel, as illustrated in Figure 3-23. The dams are preferably made by concrete or shotcrete with some weir board for measuring the flow (Thomson overflow). If the dam/rock connection is tight, the inflow for different sections could be measured.

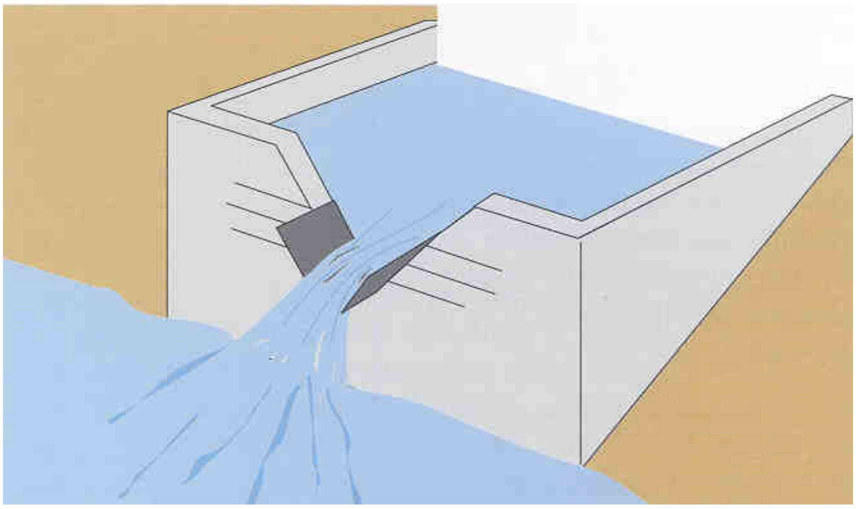


Figure 3-23 Weir board placed in a dam crossing the tunnel floor (Pettersson & Molin, 1999).

If full-face grout trials should be performed, each fan could preferably be done as during the grout trials at South Link, Stockholm (Dalmalm et al, 2000). Each fan was divided into two halves with equal Lugeon sums and grouted with different grouts. Ten observation holes, five in each half were measured and the Lugeon value noted booth before and after grouting. Thereby grouts could be compared for reasonably equal environments. Of course, one joint is never equal to another, but it is not possible to grout the same joint twice. This method therefore provides reasonably close equal surroundings for testing two different grouts against each other.

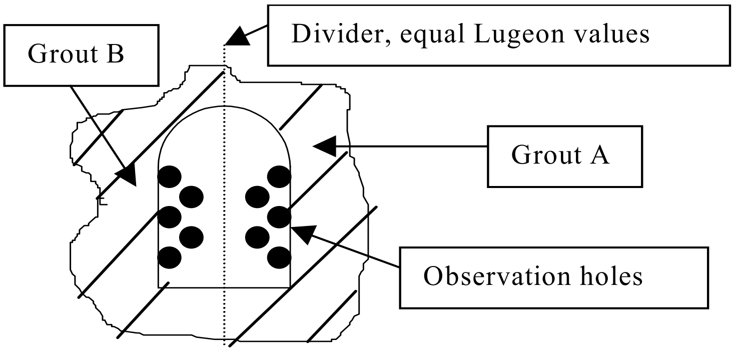


Figure 3-24 Principal layout of trial fan at South Link, Stockholm The fan is divided into equal Lugeon sums and corresponding observation holes are shown, Dalmalm et al (2000).

3.4 Grouting Technique

3.4.1 Introduction

The injection of grout into the rock mass joint systems can be performed by applying different methods. The key to success is to choose the right method for the particular situation.

The correlation between the needed grouting effort and a sealing result, which fulfils the inflow requirement, is often difficult to determine, but may be possible using methods developed during the latest years, see for example Table 3-4.

3.4.2 Some principles for grouting

When the joint systems of the rock mass have been identified, the grouting should be adopted for the particular situation. The rock mass normally consists of different types of joint systems. To reach a satisfactory level of sealing it is often necessary to outline at least two different main joint systems which need different kinds of treatment, regarding grout proportion, grout pressure and hole setting. Therefore, for example Palmqvist (1983) recommended that grouting is performed in two stages. During the first stage the sealing is concentrated on course sealing and during the second stage the sealing is concentrated on fine sealing.

The geometry of the joint system could be generalised by two events, the horizontal open joints and the vertical open joints, which run perpendicular to the tunnel (Bäckblom, 1986). The existence of open vertical joints can normally be easily counted, but the existences of horizontal joints close to the floor or ceiling of the tunnel are more difficult to recognise. The existence of horizontal joints can be difficult during grouting if for example the water conductive structure is unsealed and only some of the horizontal planes are sealed as illustrated in Figure 3-25.

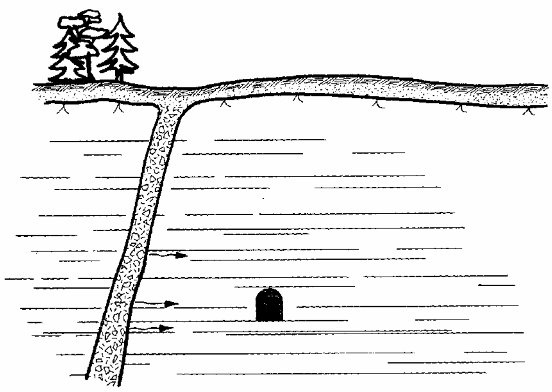


Figure 3-25 An example of water flow distribution to horizontal joints (Bäckblom, 1986).

The identification of the rock mass, which is based on the groutability of the rock mass, can then result in a classification of the rock mass. The classification is used to facilitate the decision making about different grouting activities.

Bäckblom (1986) classified the groutability as a function of the joint geometry and the filling of the joints. The joints were then divided into vertical-, horizontal- and combined vertical-horizontal joints. In addition each joint system could be: open, gouge filled or have a combination of filled and unfilled joints as shown in Figure 3-19. The different joint systems were then classified for groutability according to the four classes:

1. Straight forward
2. Moderately difficult
3. Difficult
4. Very difficult

According to Priest (1993), the vertical joints are easier to grout than the horizontal joints, which are confirmed by Figure 3-26. According to Stille & Palmström (2003) two main objectives of a classification system were distinguished; one to facilitate communication between users and one to be a base for decision-making. A classification of the rock mass is preferably rather simple, with few classes, as too many classes may complicate communication between users.

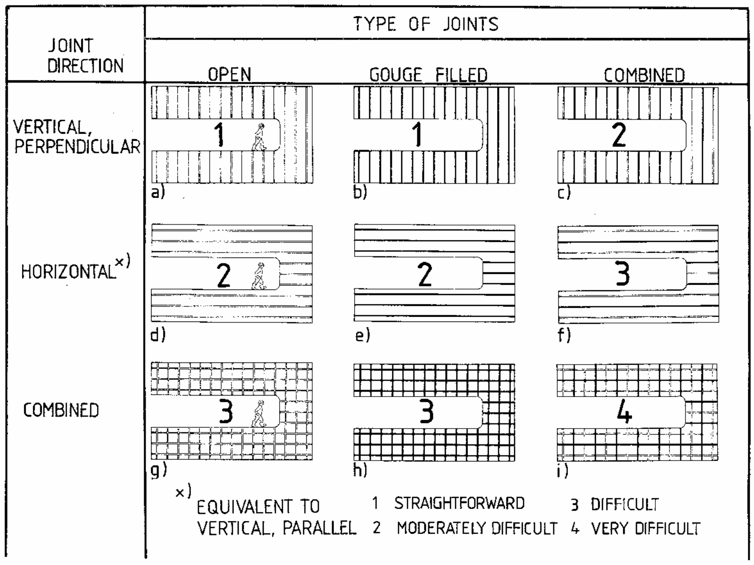


Figure 3-26 An example of a grouting classification (Bäckblom, 1986).

In the literature two different philosophies could be found regarding the placing of the grout holes. In Handboken Bygg, Geoteknik, (1984) it is recommended to drill all holes that should be grouted at once if the filling material of the joints is relatively watertight. The other philosophy is the split-spacing philosophy, where the distance between the grout holes are gradually decreased by adding an extra grout hole in-between the earlier ones. The split spacing philosophy is commonly used for grout curtains. In Houlsby (1990) the split spacing philosophy is recommended because of the possibility to evaluate the sealing result between the different grout sessions. Most tunnel projects in Sweden today are strive to drill all holes at once and thereby reduce time for an extra drilling cycle.

According to Palmqvist (1983), the anisotropy in the rock mass should be regarded for design of the fan layout. A more anisotropy rock mass requires a larger number and shorter grout holes than a homogeneous rock mass. An example of this was shown during the construction of the Hallandsås rail tunnels. On the north side the joint system consisted of many short joints, which for best result were threaded by a large number of short holes (Sturk & Nelson, 1996). If the hole is stable it is preferable drilled to full length and then stepwise grouted from the bottom to top (Kutzner, 1996).

The aperture distribution of the joints which need to be sealed has a large impact on the choice of grouting technique. Based on numerical calculation Eriksson (2002) graded the importance of some factors in relation to the aperture which needs to be sealed. As shown in Table 3-6, for small apertures; the pressure needs to be high, the minimum flow needs to be low and the distance between the grout needs to be short to receive a high sealing effect.

Table 3-6 Proposed matters to consider when choosing a grouting technique for joints of different apertures. ++ represents high importance, + important, - not important, (Eriksson,. 2002).

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

3.4.3 Fan geometry

The geometry of the grout fan will be different for different kind of projects. For dam grouting one strives to achieve an elongated screen, which stops the water from flowing under the dam, and for tunnel grouting, the grout fan should surround the tunnel. For both situations, the placing of the holes should be adapted to hit the conductive structures, which calls for certain efforts from both the geologist and the drilling personnel. Depending on the tunnel axis in relation to the joint planes, the same fan layout could be both favourable and unfavourable. It is therefore as seen in Figure 3-27.

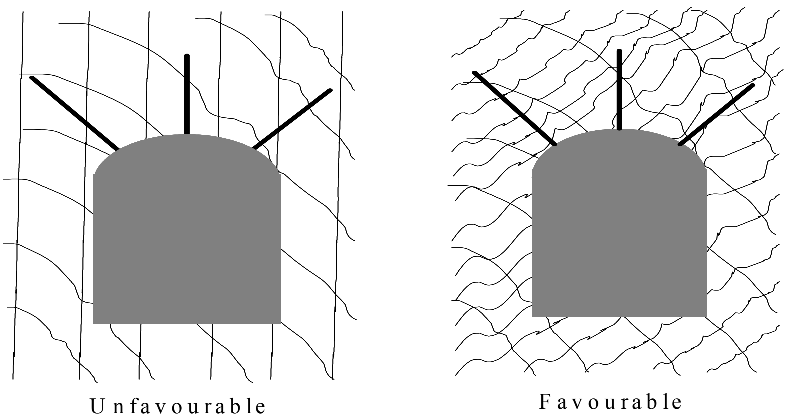


Figure 3-27 Different joint systems need different fan layouts.

The typical fan surrounds the tunnel with holes parallel or inclined out from to the tunnel. The inclination of the grout holes are normally between 10 – 20 degrees out from the tunnel. For pre-grouting in tunnels the variation of the fan layout is limited due to limited space for drilling. For most situations the fan will therefore have the form of a trumpet as shown in Figure 3-28.

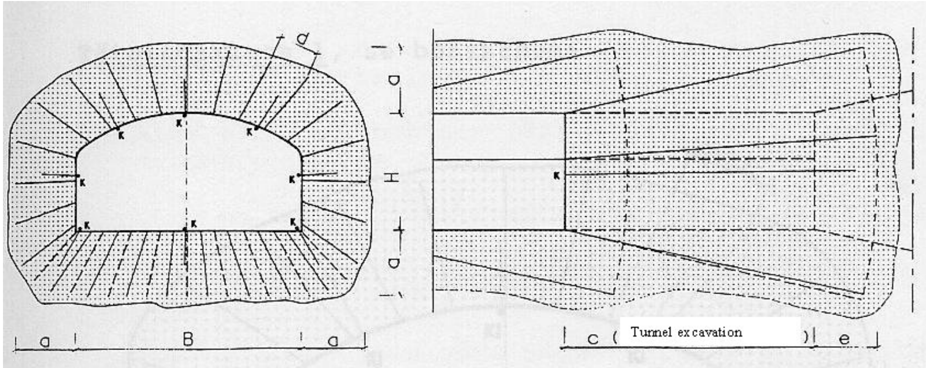


Figure 3-28 Typical layout of a grout trumpet surrounding a tunnel.

3.4.4 To intersect the conductive part of a joint

In chapter 3.2.3 it was reported that only 5-20 % of a joint plane is conductive. This accentuates that grout holes need to intersect this conductive part of the joint plane, otherwise, no grout spread will be possible. To intersect as many joints as possible, the grout holes should be drilled at right angles to the main joints, which for pre-grouting of tunnels can be difficult due to limited space. To illustrate openness in a joint plane, which was discussed in chapter 3.2.3, two hypothetical events are shown in Figure 3-29. The conductive part of a joint plane can, depending on the geological origin, look very different. The bright spots are contact areas, or areas with small or no aperture open for grout flow.

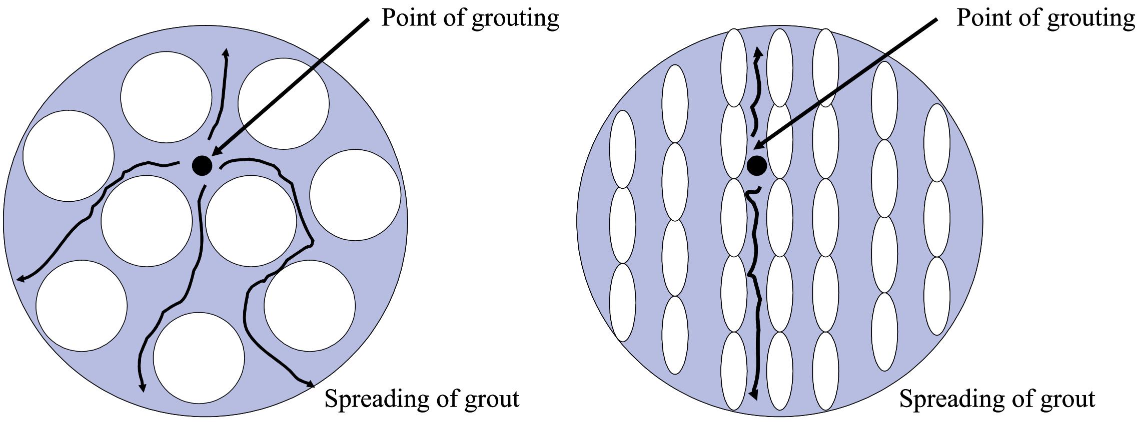


Figure 3-29 Illustration of openness in a joint plane. Bright areas are contact areas.

Depending on the directions of the joint systems, the inclination of the grout holes can be varied to facilitate a successful sealing, as illustrated in Figure 3-30.

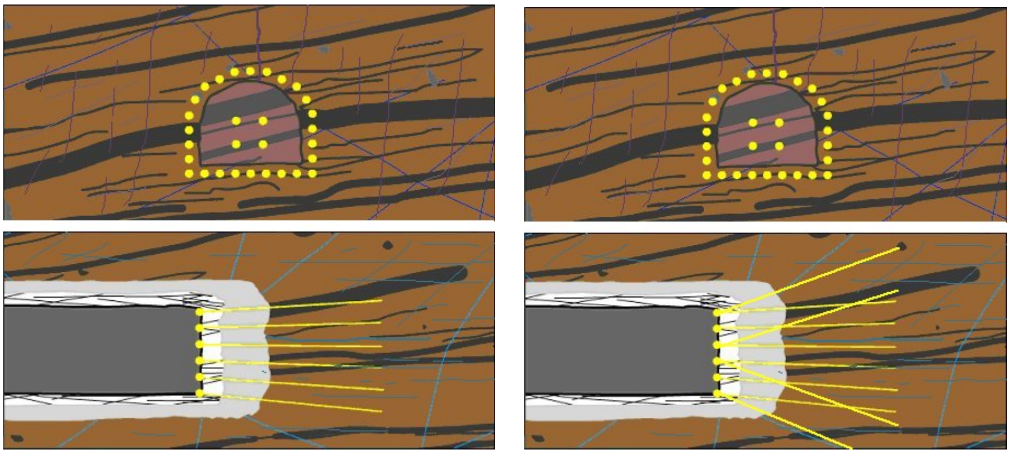


Figure 3-30 Grout holes with standard and varied fan layout.

From a grouting point of view the focus is commonly to seal the conductive joints, while the non-conductive are given less focus. To seal the non-conductive joints are difficult as an escape route is required if the grout is to fill the volume.

After grouting and excavating a rock mass the stress direction in the rock mass will to some extent be changed, this may give an alteration between conductive and non-conductive joints. It is therefore the opinion of the author, that all open joints, within the stress alteration area, may need to be sealed if the requirements are high.

To improve the sealing effect the inclination of the grout holes was varied for the fans carried out during the construction of the Hallandsås tunnel. To further improve the sealing effect the number of grout holes was increased from 18 to 36. Both improvements showed to be successful for increasing the sealing effect (Sturk & Nelson, 1996).

3.4.5 Volume, flow, pressure, GIN and INJ-criteria

Different grout criteria are set up to limit the amount of grout in the rock mass. The maximum volume is usually restricted because excess volume can damage surrounding constructions or lift the rock mass. The maximum allowed pressure is generally set in relation to the water pressure and the rock overburden, which should not be lifted. Depending on the grout mix, the rock mass could be lifted or even fractured after a very short time of high pressure. The maximum grout volume considers the risk of long penetration lengths which may affect the uplift as well as the filling of unwanted areas.

In combination with the pressure, the minimum flow is also used as a stop criterion for grouting. The flow could be regarded as an expression for the increase in penetration length (Dalmalm et al, 2003). The flow may therefore be considered as a parameter to regard when deciding whether to continue grouting in present hole or to continue grouting in a new hole, as further described in chapter 7.3.

A criterion which combines the volume and the pressure is the GIN method (Lombardi & Deere, 1993), which could be explained as the total amount of energy that is allowed to be injected into the rock mass. The energy consideration is especially important to regard when grouting already stressed rock masses, which may only withstand a little extra energy before they get uplifted or fractured. The GIN-method was developed to control grouting operations during dam construction.

The aim is to perform grouting in a more effective and economic manner, which is achieved by:

- A minimised risk of hydraulic fracturing.
- A more uniform grout spread.

The GIN method could be noted as in Equation 3-21,

$$GIN = p \cdot V = grouting\ energy \qquad \text{Equation 3-21}$$

Where:

p	grout pressure at zero flow (stop pressure)	[bar]
V	grouted volume per meter of hole at stop pressure	[litres/meter]

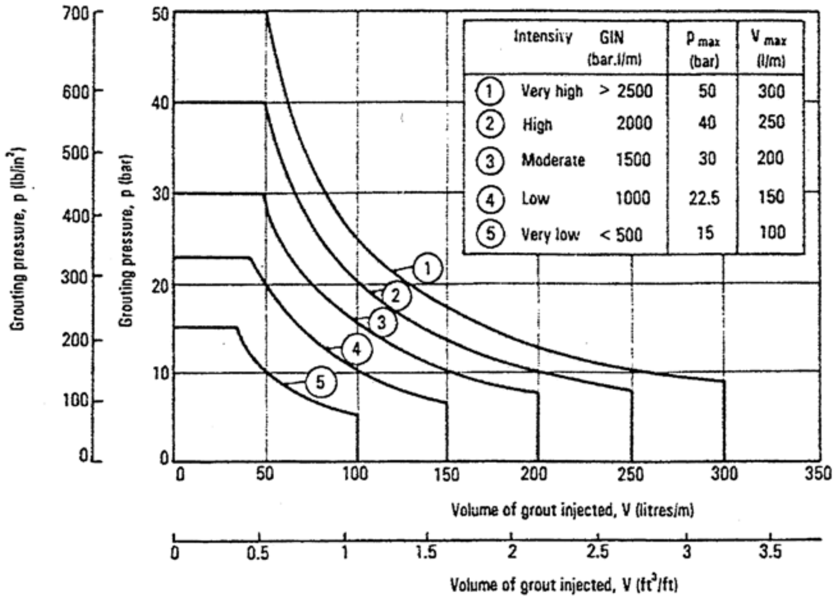


Figure 3-31 Some standard GIN limiting curves, as a starting guideline (Lombardi, 1996).

The GIN principle has limitations such as, it does not regard the time dependent grout properties and it does not fully describe the risk for hydraulic fracturing (Ewert, 1996 & 1996b), (Brantberger, 2000).

When grouting in tunnels the risk for uplift and the penetration length may need to be regarded. The risk for uplift depends both on the pressure and the penetration length, and the penetration length may also for other reasons be reduced.

To avoid lifting of the rock mass, the uplift force needs to be less than the force from the resisting overburden rock mass. By introducing a normalized pressure p_n, according to Equation 3-22 and a normalized volume V_n, according to Equation 3-23, the relationship for determining the risk of hydraulic uplift could be noted according to Brantberger et al (2000) as shown in Equation 3-24.

$$p_n = \frac{p \cdot k_2}{3\rho \cdot g \cdot h}$$ Equation 3-22

$$V_n = \frac{V}{h^2 \cdot \pi \cdot k_2 \cdot N}$$ Equation 3-23

$$p_n < 1 + \sqrt{\frac{b}{V_n}} + \frac{1}{3} \cdot \frac{b}{V_n}$$ Equation 3-24

Where:

p_n	normalized grouting pressure
p	grouting pressure
k_2	factor accounting for that the joint is not completely open to grout flow
ρ	density of rock mass
g	acceleration due to gravity
h	joint depth below surface
V_n	normalized grout volume
V	grout volume
N	number of joints
b	joint aperture

For a given GIN-value, the product of the pressure and volume is constant. For a given aperture b according to Equation 3-25, the relationship between the normalized pressure p_n and the normalized volume V_n, is not constant as also shown in Figure 3-32. Therefore the conclusion is that the risk for uplifting could not solely be expressed by the pressure and the volume, but the joint aperture was also required.

If the normalized volume V_n is divided by the aperture b, an expression for the normalized grout spread I_n, can be noted as:

$$I_n = \frac{I}{h} = \sqrt{\frac{V_n}{b}}$$ Equation 3-25

Where:

I	grout spreading distance

And Equation 3-25 can then be rewritten as:

$$p_n < 1 + \frac{1}{I_n} + \frac{1}{3 \cdot I_n^{\ 2}}$$ Equation 3-26

The relationship between the normalized pressure and the normalized grout spread I_n is also shown in Figure 3-32. The area above the lines in the figure represents uplift and the area below the lines represent no risk for uplift.

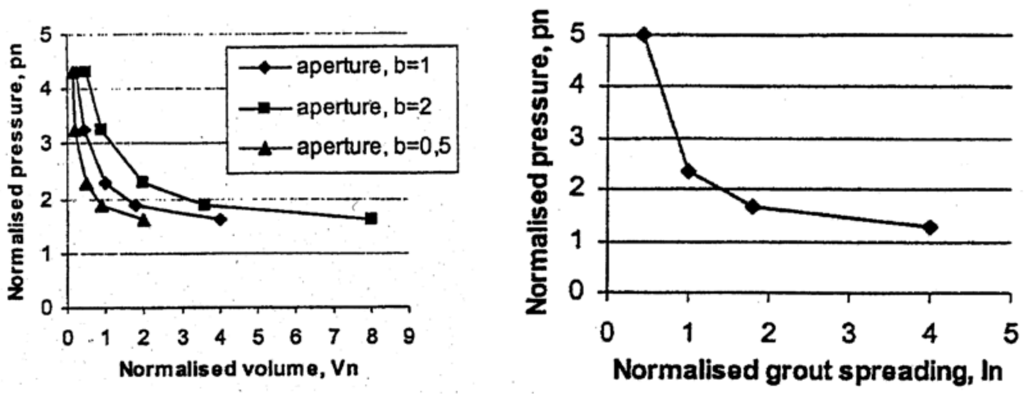


Figure 3-32 The normalized grouting pressure as a function of the normalized volume and the normalized spreading (Brantberger et al, 2000).

There are two ways of calculating the normalized grout spread. Either the grout spread is calculated based on the measured grout volume and an assumption of the aperture. Or the groutability factor *K* by Lombardi (1996) is introduced. The grout spread *I* can then be noted as:

$$I = K \cdot \sqrt[3]{\frac{p \cdot V}{\tau_0}} \qquad \text{Equation 3-27}$$

Where τ_0 is the yield value of the grout. If theories developed by Hässler (1991) are used, the following expression for the groutability could be derived:

$$K = \sqrt[3]{\frac{1}{N \cdot \alpha}} \qquad \text{Equation 3-28}$$

Where α is the spreading angle according to Hässler (1991). By introducing the concept of a normalized pressure and a normalized grout spread by Brantberger et al (2000), the risk of hydraulic lifting can be plotted in a diagram, and thereby observed.

3.4.6 Grout order

The grout order can be important for the sealing result. An erroneous grouting order can result in blocking of flow paths that needs to be open until other flow path have been fully reached. A correct handling of grout order and flow path for a satisfactory penetration and filling of the joint system is a complex and difficult job.

Three types of grout order are common within tunnel grouting:

- Grouting is started in a randomized hole, commonly at the bottom, and continued around the tunnel.
- Grouting is started in two holes at the bottom and continues upwards along each wall of the tunnel.
- Grouting is started in the most conductive hole and thereafter continued with the less conductive holes.

Grouting is commonly started with a thin mix and continued after a certain criterion to a thicker mix (Houlsby, 1990). For some situations with high water inflows, grouting could be started with a thicker mix in order to block the flow.

For dam grouting of the substrata before 1970, holes were often grouted in large sections or at full length, which tended to only fill the wider joints, which is indicated by the number of re-drilling and re-grouting of the rock mass according to Cambefort (1964).

Models for calculating the grout volume do usually assume that the rock mass is ungrouted during the procedure. In practice, this means that only the first grout hole is possible to calculate, thereafter the surrounding rock mass is effected by grout. If the grout holes are connected in some way, the grout volume will decrease as the number of holes increases, according to Janson (1998) and shown in Figure 3-33.

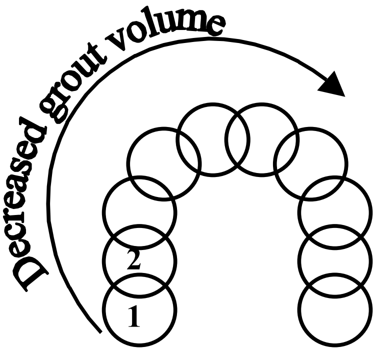


Figure 3-33 Grout volume decreases when the number of holes increases if holes are connected (Dalmalm, 2002).

To decrease the immediate water inflow to a tunnel the grouting could start in the hole with the highest water loss value. A high water loss value is often equal to a high grout volume (Dalmalm et al, 2000). For some situations even a low water loss value could give high grout volumes, depending on the pressure difference between grouting and water loss measurements (Batres-Estrada & Graan, 2003). From calculations it has been shown, that a high grout volume is not equal to a high sealing effect (Eriksson, 2002). A common argument to start with the hole with the highest water loss is that water routes are not cut of by small amounts of grout that may be blocking a full penetration of the highly conductive joint, as illustrated in Figure 3-34.

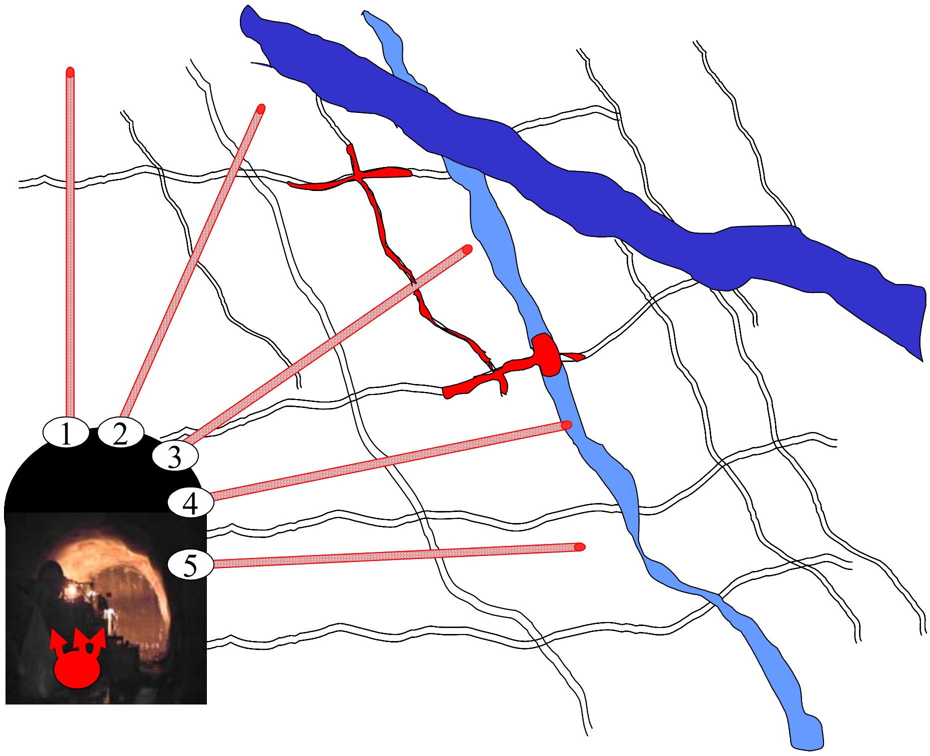


Figure 3-34 An erroneous grout order may block further sealing of the highly conductive joint. Red indicates grout holes and the penetration of grout in joints caused by grouting in hole 3. As can be seen an initial grouting of this hole may restrict further sealing.

In Figure 3-34 a simplification in two dimensions of a joint system is illustrated. The highly conductive joint (dark blue) is best reached by grout hole number 4, through the medium (light blue) conductive joint. It is therefore also likely that the measured water loss will be highest in hole number 4. If grouting is started in hole number 4, it is likely that the highly conductive joint could be reached and sealed by grouting. If instead grouting is started in grout hole 1 and then continued to number 2, 3, 4 and 5, grout hole number 3, which is connected to the medium conductive joint through smaller joints, may only fills the medium joint with a small amount of grout and thereby restricts further penetration when grouting is continued in grout hole number 4. Due to the stop criterion grouting in hole number 3 may be stopped before filling the highly conductive joint. The result of not grouting in order of highest water loss value may then be that the highly conductive joint through very fine ungroutable joints continues to distribute water to the tunnel.

Different grout holes could be grouted simultaneously, but different pumps are then recommended for the different joint group so that the pressure and flow could be adapted for the individual joint system situation.

3.4.7 Grout practice today

Common concepts of grouting practice have here been summarised in the below Table 3-2.

Table 3-7 Grout practice for hard jointed rock in Sweden (based on Brantberger 2000).

Parameter	Today's practice in Sweden
Fan	10-20 meters holes, with 2-4 meter bottom distance in first grouting. Hole angle 5-20 degrees.
Drilling and grouting cycle	Drilling and grouting of the hole, to full length in one operation
Grout	Two alternatives: - Separation stable grout with low W/C ratio, fine grained cements and additives. - High W/C ratios with or without stabilisation additives, fine grained cements. The grouting is started with a thin grout and thickened after a certain volume. Commonly the same procedure for all re-grouting, sometimes different cement types. Choice of cement type and W/C ratio is sometimes based on water loss measurements. Sometimes the cement grouting could be complemented with chemical grouting, for example sodium silicate or polyurethane, but the chemical grouting has during the last years become very sparsely used.
Grouting equipment	Colloidal mixer type (app. 1500 rpm), one or some agitators, piston- or plunger pumps, varying types of registration and dosage equipment.
Grout pressure	Normally a stop pressure of 1-2.5 Mpa over ground water table. With close constructions or poor rock overburden pressures of 0.5 Mpa are used. For special purposes, like a positive fracturing of the rock mass, pressures up to 5 Mpa have been used.
Stop and grout criteria	Normally pressure and volume criteria. Sometimes flow, pressure and time criteria. Commonly a maximum volume of 50-100 litres per meter hole. Lower volumes if risk of spreading to surrounding constructions.
Grout order	Normally the grouting starts with the bottom holes and continues up along the walls. Sometimes the grouting starts in the hole with highest Lugeon value.
One- or multiple hole grouting	Normally two holes, one on each side, simultaneously with different pumps. Sometimes hole by hole with one pump. Connected holes are grouted simultaneously with the same pump.
Grouting length / Interval grouting	Grouting in one round from the bottom to the top. Stage grouting of the hole is not common.

3.5 Drilling

3.5.1 Introduction

Grouting is performed in drilled holes which, in order to distribute the grout, should hit the conductive structures of the rock mass. Two types of drilling are of concern to the grouting process, the drilling for exploration and the drilling for grouting production. The extent of both depends on the chosen grouting method. For a situation with large uncertainties, the method may suggest more extensive exploration and vice versa.

Drilling for grouting in tunnels may be performed as shown in Figure 3-35.

Figure 3-35 Drilling for grouting (Pettersson et al, 1999).

There are two major drilling methods; the rotary drilling and the percussive drilling. For rotary drilling the rotation takes place while the drill bit rests against the bottom of the hole. For percussive drilling the rotation takes place while the drill bit is rebounded from the rock in the bottom of the hole.

Rotary drilling is commonly diamond- or core drilling and the rotation speed can then be in the range of 600 rev/min, which is much higher than for percussive drilling, which uses rotation speeds between 10 to 160 rev/min.

The advantageous of high speed rotary drilling are that: the rock mass can both be explored and grouted with the same holes and the grout holes have less deviation than if drilled with percussive drilling. In general, rotary drilling is considered better for grouting as rotary drilled holes will be clogged by drill cuttings to a lesser extent. The size of the cuttings are larger for the percussion drilling than for the rotary drilling. In some situations the small cuttings from rotary drilling may be more disadvantageous than the larger cuttings from the percussion drilling as they clog the fine joints.

For grouting, percussion drilling is today dominant as the cost of rotary drilling is much higher.

3.5.2 Holes (length, size, deviation and number of holes)

For grout holes in tunnels the length is adjusted to be sure of a safe overlap between the grout fans. A common length of the grout holes are between 20 to 25 meters, which together with 15 to 20 meters of excavation gives 5 to 10 meters overlap between the grouting fans.

The common hole diameter for grouting- and exploration drilling is in Sweden today 64 mm. For Exploration drilling the diameter may sometimes be increased up to 76 mm and for grouting the diameter may sometimes be decreased to between 46 and 56 mm. The hole diameter should be adapted for the packers on site for an effective tunnel production cycle. If the drill bits are too worn, it may cause problems for the placing of the packers.

The number of grout holes for a fan varies depending on the adopted grouting method. The number of holes has for some projects been reported in chapter 2. The optimal number of holes depends on the joint situation in combination with the sealing effect requirement, the grout and the requirement of hydraulic conductivity after grouting. Too few holes will result in many re-groutings, which generates high costs for establishment. Too many holes will increase the drilling time, without improving the sealing effect.

The conductive part of the joint plane and the connectivity in between joints affects the distance between the grout holes. Ewert (1992) divides between direct and indirect connections, were direct is a large number of short joints with many connections and indirect is a few long joints with few connections. For direct connections the pressure fall is large even over a short distance from the grout hole, which can demand a closer grout hole placing. Indirect connections allow, on the contrary, a larger spacing between grout holes.

The deviation of the drill hole is less for rotary drilling than for percussion drilling. There are many explanations as to why hole deviation occurs, like poor positioning, drilling across or along joints or gravity acting on horizontal holes etc. (Pettersson & Molin, 1999).

Based on a combination of openness of a joint plane and hole deviation, it may be assumed, that the hole deviation will not affect the sealing result as long as the deviation is unbiased and of minor magnitude. In the literature review by the author, no case studies have been found where poor sealing effect could be explained by hole deviation.

3.5.3 Cleaning of holes

It is important that the grout holes are cleaned after drilling (Houlsby, 1990). The holes could be cleaned by:

- Flushing water through the bit after drilling
- Using a special nozzle with back flowing water
- Using both water and air
- Using high pressure water

The use of high-pressure water was studied during the construction of the Royal Library in Stockholm by Andersson (1994), the rock mass were sparsely jointed, but no positive results could be shown from raising the cleaning pressure. In Helsinki high-pressure water was used for hole cleaning in a rock mass, which was highly jointed with fine joints, whereof some were filled with kaolin clay. The cleaning was improved by the high pressure and some of the joints were also cleaned from clay. The grout holes are sometimes cleaned using the drill bit for flushing after drilling. Grout holes cleaned with the drill bit were studied with a borehole camera by Dalmalm et al, (2000). The flushing sometimes succeeded, sometimes resulted in poor results with drill cuttings remaining in the hole even after flushing as shown in Figure 3-36. It is not known if this was due to the method or due to lack in performance.

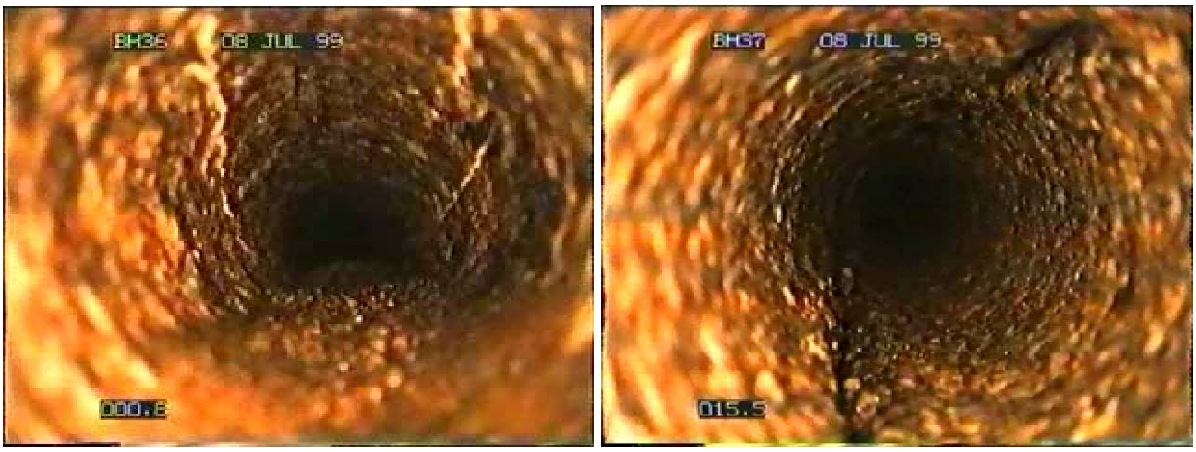


Figure 3-36 (Left) Drill cutting shown at the bottom of a grout hole, which has not been sufficiently cleaned before grouting. (Right) A wide open non conductive joint in a grout hole.

3.6 Grout mix properties

A good grout mix is very important for the grouting result. The grout mix consists of cement, water and commonly one or more additives. The additives could further be of liquid- or solid types. In this chapter the cement, the additives, the properties and the mixing will be discussed, with emphasis on the properties that affect the sealing.

Chapter 3.6.1 and 3.6.2 is based on work by Lagerblad (1997) and Fjällberg & Lagerblad (2003).

3.6.1 Cement

Cement grouting can for example be performed with Portland cement, aluminate cement or ground granulated blast-furnace slag (slag cement). Portland cement is the preferred material today in Sweden. The slag cement has a longer hardening time and the aluminate cement has a shorter hardening time than Portland cement. Burning limestone and clay to a cement clinker, at 1450 degrees Celsius will result in a Portland cement. The clinker is thereafter milled together with gypsum, to avoid flash setting. Slag cement consists of 45 – 80 % fine ground blast-furnace slag from iron production and remaining part 20-55 % is Portland cement. Burning limestone and bauxite at a temperature of 1450 to 1600 degrees Celsius will result in aluminate cement. An admixture of aluminate cement added to a Portland cement can give a very short hardening time. Depending on the proportions, the hardening can be much faster than would otherwise be achieved by the two components separately. The final strength becomes much lower, which for certain sealing situations can be of minor importance. For grouting purposes, the maximum cement particle size is important and often expressed as d_{max} or specific surface.

3.6.2 Additives

<u>Liquid additives</u>

A number of different additives are used in order to control the hydration of the cement paste and its flow properties. Depending on the situation the hydration can either be accelerated or retarded. Within grouting the accelerating additives have their foremost use within grouting for stopping the water when highly conductive zones are hit.

The accelerators can be split in two types:

- Binder accelerators that give an early hardening of the grout.
- Strength accelerators that give an increase of the early strength.

The most common accelerator is calcium chloride, which affects both the binding and the early strength development. For extreme cases up to 15 % of calcium chloride has successfully been used as an accelerator. The long time stability of grouts with high contents of calcium chloride is today unknown. It should be noticed that, calcium chloride is a retarder for aluminates cement and slag cement. Other early strength accelerators are for example: potassium carbonate, sodium carbonate, calcium formiate, tri-ethanol-amide, calcium acetate, calcium propionate, and calcium butyrate (Fjällberg & Lagerblad, 2003).

In the group of binder accelerators there are for example: sodium silicate, sodium aluminates, aluminate chloride, sodium florid and calcium chloride. Addition of aluminate cement or calcium aluminate in Portland cement is also an accelerator.

The use of retarders is not common during grouting works. If needed superplasticizer are commonly acting as retarders.

The most common liquid additive is the use of surface-active superplasticizers. With reduced cement particle size the inter-particular forces between the particles are increased. The particles are charged and attract each other (flocculating). In order to break these bindings, surface-active superplasticizers can be used for a steric- or electrostatic repulsion of the cement particles (Uchikawa et al, 1997).

Generally a superplasticizer affects the grouting works in a positive way because the grout can penetrate and reach further before hardening begins.

Different additives have different effects on different cement types, depending on the dominating cement clinker. For Slite cement (C_3S, C_3A) both the naphthalene- or melamine-based superplasticizer can be used. However the naphthalene-based superplasticizer have much better effect on the rheology, especially if the W/C ratio is beneath 0.4. For Degerhamn cement (C_2S, C_4AF) both melamine-naphthalene and acrylic-carboxyl based superplasticizers can be used. Though, the acrylic-carboxyl based polymers are to be preferred if a low viscosity is required. The melamine-naphthalene based superplasticizers use electro static forces to separate the cement particles and the acrylic-carboxyl based polymers use steric repulsion. At low W/C ratios or for very fine ground cement, the steric superplasticizers are to prefer. If the hardening time is important, the melamine-naphthalene based superplasticizers may be preferred as they retard the grouts less than the acrylic-carboxyl based polymers (Fjällberg &Lagerblad, 2003).

<u>Solid additives</u>

For various reasons the strength of hardened cement is not always required for grouting purposes. Therefore different solid additives like mineral particles can be added to the grout. A normal addition could be 20 to 100 % of the cement weight. The additives can be both active and inert. The most common solid additive is pozzolan which chemically reacts with the cement pore solution. The pozzolan can be either a natural mineral, for example fine coursed volcanic tuff, or an industrial mineral such as slag, fly ash or silica fume. A pozzolan will not react with water by itself, but if it is milled to fine particles and moisturised, it can react chemically with calcium hydroxide and thereby consume calcium hydroxide, which increases strength and durability. The pozzolans can, in the same way as silica fume increase the strength and alter the mobility of the grout. Many pozzolans are waste products from other processes and therefore added in order to reduce the cost. There are therefore many possible reasons to add a pozzolan to the grout mix.

Pozzolans

The most common pozzolans are fly ash and silica fume; fine grained volcanic tuff, calcined kaolin and rice ash. Pozzolans are added in order to increase the durability and increase the strength (silica fume). A pozzolan activity is when the product chemically takes part in the cement hydration process, generally by consuming calcium hydroxide. Some of the pozzolans for example shells and clays demand a lot of energy to transform, which is costly, and are therefore not regarded as an alternative.

Natural pozzolan

A natural pozzolan is a fine-grained fume, milled from a volcanic tuff.

Silica fume

Silica fume is a waste- or bi product from Ferro-silica manufacturing. The particles are spherical, as fly ash, but much smaller. The diameter is less then 0.1 μm (D_{50}). Silica fume is used as an addition to concrete to make it denser. For grouting there are dispersed silica slurries, which stabilize the grout mix and thereby reduce the separation.

Fly ash

Power production in coal power stations produce the waste product fine-grained fly ash, which can be used as an additive in concrete or grout. The particle size can vary between 1 - 150 μm. There are no good fly ashes available in Sweden today to mix in concrete or grout. Mixing of fly ash to the grout will increase the density and increase the chemical resistant. However, fly ash is less effective pozzolan than silica fume.

Rice ash

When burning rice shell, the waste product will be rice ash. Rice ash is pozzolanic and often used in Asia as an additive to grout and concrete.

Fillers

As mentioned above, high strength of hardened grout is not so often required and therefore the cement can be mixed with inert filler. Suitable fillers are fine milled stones of different origin, preferably limestone, blast-furnace slag or fly ash. The fine milled limestone will stabilize the grout and accelerate the hydration process.

<u>Expanding additives</u>

To increase the sealing effect an admixture of fine milled aluminate or aluminate/sulphate could be used. These components will expand the grout in the rock mass so that the sealing effect is increased. Activated coal could also be used as an expanding additive. When the coal is moisturised gas bubbles will be released which expands the grout. An expansion of the grout could also be reached with burned chalk (CaO) or periklas (MgO). When they hydrate to calcium- or magnesium hydroxide an expansion will occur. For ordinary concrete constructions, burned chalk or periklas often result in a weak construction due to swelling. For grout constructions, the situation is different. The grout is fixed in the joint, which is fixed in the rock mass and the expansion will therefore probably result in a densification of the hardened grout and/or maybe a filling of the unfilled edge zones of the joints.

<u>Stabilising additives</u>

Bentonite (montmorillite) is a volcanic clay, which can absorb large amounts of water. The bentonite is inert and is normally added to stabilise the grout against separation. Since the introduction of micro cements, the use of bentonite has decreased. The micro cement based grouts are much more stable than conventional cements, even for relatively high W/C ratios, The bentonite will also make the grout more thixotropic.

3.6.3 Properties of cement based grout which affects the sealing

To reach a good sealing result, some specific properties of the grout mix are desirable. The grout itself can either be a particle suspension (cement, bentonite) with viscosity μ_B or a liquid (chemical grout) with viscosity μ. The liquid grouts can be described as a Newton fluid as shown in Figure 3-37, with similar properties as water, and the particle suspensions grouts can be described as Bingham fluids, with similar properties as "ketchup". A particle suspension cannot seal all conductive joints because the particles need a larger aperture than water to penetrate a joint. In addition the yield value τ of the grout, which is higher for grout than for water needs to be exceeded for grout propagation.

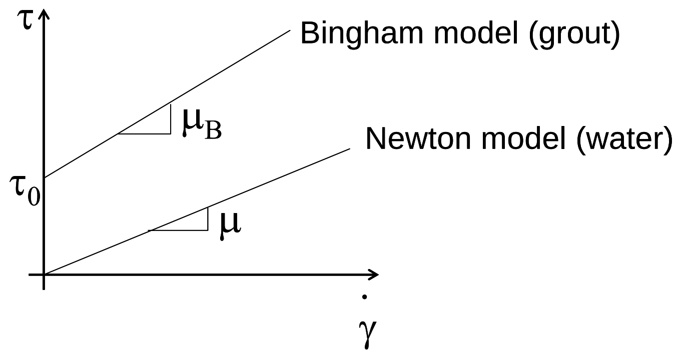


Figure 3-37 Bingham model for grout and Newton model for water.

To reach a good penetration and filling of the joint system in a rock mass the grout should (for both solution- and suspension grouts):

- Have good penetration
- Have good joint filling
- Harden fast and without shrinking

The sealing result for a cement-based grout depends on the ability to penetrate, which is dependent of the flow properties (yield value, viscosity), the stability against separation and the particle size of the cement particles (Håkansson, 1993). The joint filling is dependent on the stability against separation and the size of the cement particles, as is further described in this section. The filtration at narrow passages is in chapter 3.6.4 shown to have a large influence on the penetration.

Flow properties (rheology)

There are many parameters, which influence the rheology of the grout. In 1993, Håkansson presented some of them, as is shown in Table 3-8.

Table 3-8 Influence in the rheology by different factors and additives (Håkansson 1993).

Additive / action	Influence on:		
	Yield stress*	Viscosity*	Binding time*
Decreased W/C ratio	+ + +	+ + +	- -
Increased specific cement surface	+ +	+ +	- -
Decreased temperature	+ +	+	+ +
Addition of bentonite	+ +	+ +	+ / -
Addition of silica	+ + +	+ +	-
Addition of superplasticizer	- - -	- - -	+ +
Addition of sodium silicate	+ + +	+	-
Addition of calcium chloride	+	+	- - -

*) + + + large increase; - - - large decrease
+ + moderate increase; - - moderate decrease
+ small increase; - small decrease
+ / - indifferent

An important factor that influences the flow property of the grout is the W/C ratio. A decrease in the W/C ratio will result in an exponential increase in both the yield stress and the viscosity (Håkansson, 1993). The development of superplasticizers has during the latest years also improved the possibilities for achieving low viscosity grouts from earlier "thick" mixes.

An increase of the W/C ratio will result in a high mobility, but a decreased stability as shown in Figure 3-38. The mobility is also dependent of the particle distribution.

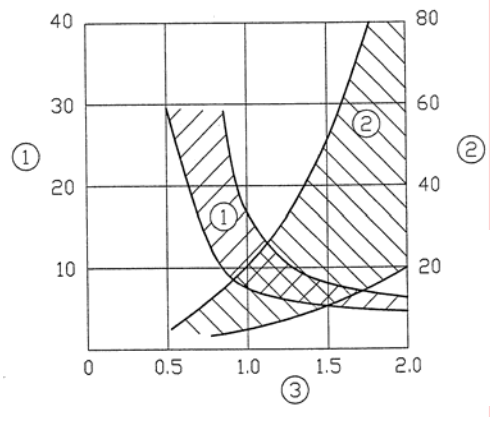


Figure 3-38 Viscosity and sedimentation velocity of cement based suspensions, Bonzel & Dahms (1972). [1. Viscosity (s MPa), 2. Sedimentation velocity (10^{-4} cm/s), 3. W/C ratio].

In addition, the rheological properties of a cement suspension are considerably changed by ageing of the grout.

Particle distribution

If all particles have the same size the cavity volume will be large, but with varying particle sizes the cavity volume will be decreased and as shown in Figure 3-39.

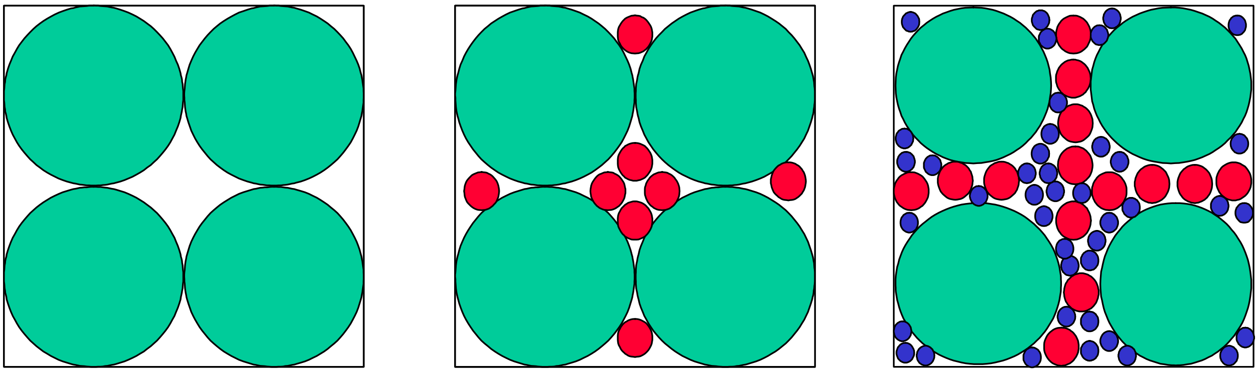


Figure 3-39 Illustration of particle packing. The small particles can fill the cavities and act as a lubricant for the larger particles.

Small particles can also give a loosening effect for the larger particles. The small particles can act as a lubricant for the larger particles and thereby reduce the inner friction of the grout. This gives an opportunity to lower the W/C ratio and thereby increase the strength of the grout. Nehdi et al (1997) showed that the rheological properties were improvement by adding calcite filler with a particle size less than 3 μm to the cement. If the added particles are too small, problems like colloid dispersion and flocking can occur. For the smallest particles the inter particle action is mainly controlled by chemical surface forces such as van der Waals forces or hetro-coagulation. Hetro-coagulation depends on electrostatic interaction i.e. that particle surfaces with opposite charging attract each other. The forces can be released by using surface-active superplasticizers (Lagerblad, 1997).

An appropriate particle distribution of the grout will therefore improve the penetrability of the grout mix and increase the impenetrability of the hardened grout (Eklund, 2003).

Stability against separation

Wide joints can also easily be blocked if the grout mix is allowed to separate, which will decrease the possibility for further penetration of the joint system (Houlsby, 1990). The sedimentation rate is dependent on the W/C ratio (Kutzner 1996). A possible way to avoid separation is to use stable grouts. A stable grout is, according to ISRM (1996), defined as a grout that after 2 hours show a separation of less than 5%. In Sweden a common requirement is to restrict the separation to 2 % after 2 hours.

The separation is, according to Tan et al (1997), described by two mechanisms:

- Sedimentation
- Consolidation, due to mass

Sedimentation is when particles due to gravity fall without contact to each other. When particles get contact, the consolidation can start. Tests by Tan et al (1997 & 1998) showed that for high W/C ratios of 1.6 the separation was a result of both sedimentation and consolidation, but with a decreased W/C ratio of 1.4, the consolidation was the dominant factor. When the grout separates at certain trial heights a channelling in the suspension can occur. The channelling increases both the separation speed as well as the total separation. The risk for channelling in a suspension increases with increasing height of the trial (Eriksson et al, 1999).

Except for the channelling effect, even the hydration of the cement can affect the separation course. In an early part of the hydration a surface film of calcium silicate hydrates (CSH) and etringite are developed as shown in Figure 3-40. In addition there are free crystals of calcium hydroxide, gypsum and etringite. After a couple of hours this layer is dissolved and the cement starts to bind (Betonghandboken, 1994). A fast hydration of the grout can counteract the separation, because a denser structure increases the stiffness and decreases the mobility. Exactly how the hydration affect separation is not yet explained (King & Raffle, 1976), (Cambefort, 1977), (Tan et al, 1997), (Tan et al, 1998).

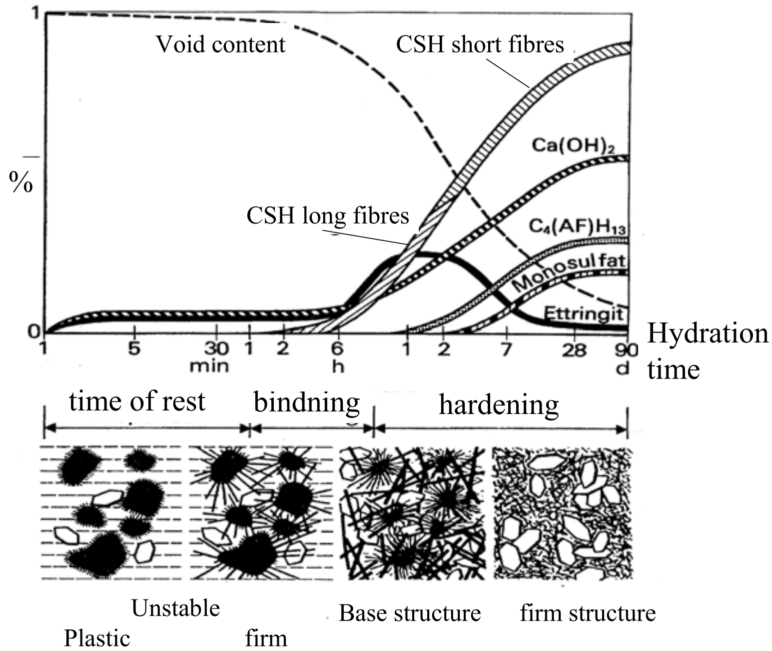


Figure 3-40 Schematic representation of hydrate phases and the structure development during cement hydration (Locher et al (1976), Betonghandboken, (1994)).

The separation is, as mentioned, dependent of a number on mechanisms; sedimentation, consolidation, hydration and channelling. Which mechanics dominates and how they interact is related to both the grout proportioning as well as the trial set up (Cambefort, 1977), (Jefferis, 1998), (Eriksson et al, 1999).

In Cambefort (1997) it was shown for grouts with short hardening time that the separation was less for high trials heights than for low trial heights.

For a grout mix with long binding time, less separation for a smaller trial height has been noticed by Eriksson et al (1999) as is shown in Figure 3-41. From the figure it could further be noticed that a short test time gives less separation and long test time gives more separation. For trial heights of 4 and 16 cm a test time of 120 minutes is enough to reach the final separation. For trial heights above 16 cm, the time also needs to be increased to reach the final separation.

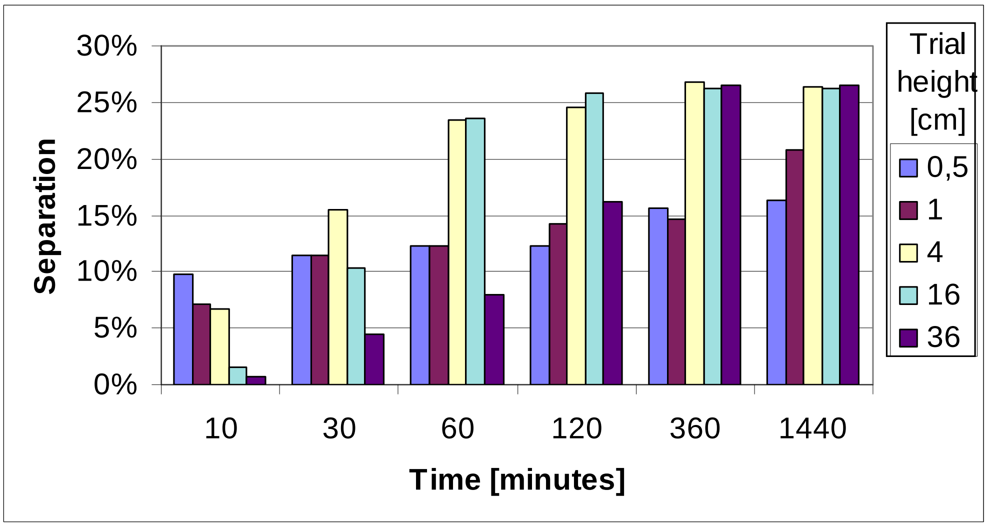


Figure 3-41 Measured separation [%] for different trial heights [cm] and time [minutes], based on Eriksson et al, (1999). The grout was based on Cementa Injektering 30, W/C ratio 1.2, superplasticizer 1 % HPM.

To estimate the separation of a grout in a specific rock joint situation could, based on the above conclusions, be difficult, as the real height is often unknown. For vertical joints, the trial height is very high, and for horizontal joints the trial height may be between ~0.5-10 mm. Depending on the specific situation, separation may or may not need to be regarded as a parameter for the sealing result.

When grouting a tunnel, the grout holes are commonly horizontal, which gives the height affected by separation in the holes equal to the diameter. Unsealed parts in a grout hole due to separation may result in new conductive routes for the water. The separation of the grout should therefore be regarded in grout holes. To regard the separation for all grout mixes, which are to be injected, may result in poorer penetration ability for the grouts. It is therefore recommended that the separation is regarded during the termination of each grouting hole.

Today superplasticizers are today commonly used to obtain mobility and separation stability for grouts with low W/C ratios. The mobility and separation stability are strongly time dependent properties. The mobility will decrease and the grout will separate with time. The grout needs mobility throughout the whole transport in the rock mass. Therefore the time from mixing until the grout has reached its final destination in the rock mass has to be limited. This emphasises the importance of small batches, which are less affected by ageing.

To grout with high W/C ratios (W/C 3), where the excess water is supposed to flush away the fillings in the joint is according to Kutzner (1996) condemned to fail. The grout can then separate and thereby block joints for penetration.

The majority of work sites and theoretical studies recommend the use of stable grouts. However, some work sites have reported that the unstable grouts with high W/C ratios have been more successful than the stable grouts. It is possible that the use of both stable grout and unstable grout can be justified, but for different geological situations. Whether stability is positive or negative for the grouting result is a question that needs further research according to Bodén et al (1995).

3.6.4 Penetrability of grout

The penetrability of a grout describes the ability to pass constriction in the grout channel route without building filter plugs. The phenomena of filter plugs for grouts has earlier been described by Hansson (1994, 1995), Schwartz (1997) and Eriksson et al (1999). The penetrability of grout is related to the filtration stability of the grout.

According to Kutzner (1996), it is possible with a stable cement based grout to penetrate rock joints down to an aperture of 0.1 mm. Smaller joints than 0.1 mm can be penetrated, with an additive of, for example, bentonite clay.

It is common to read in product data sheets for cement and additives that joints down to a certain aperture can be penetrated. It is the author experience that these values need to be regarded with some caution as the values sometimes only are based on the rule of thumb that a joint can be penetrated if the grain penetrating is less that three times the joint aperture. In Table 2-10 different author's opinions of possible joint penetration are shown.

The rule of thumb, that a joint can be penetrated if it is 3-5 times the particle size is according to the Table 3-9 still valid.

Table 3-9 Different author's opinions of possible joint penetration.

Name	Notation	Joint aperture (mm)	The particle size times
Kutzner 1996	Cement grouting. with stable grout	0.1	
Mitchell 1970 & Bergman et al 1970			3 – 5
Bogdanoff 1990			2
Pusch 1993	Cement grouting	0.05-0.1	
Pusch et al 1991	d_{max} 16 μm. dynamic grouting	0.02	1.25
Hansson 1994	d_{15} for earth. d_{85} for grout		$d_{15}/ d_{85} \geq 25$
Hansson 1994	Rule of thumb		3
Melbye 1993	Micro cement Rheocem 900	0.25-0.3	

* d_{15} means that 15 % is passing

From examinations of the penetration with the NES equipment it was reported that the column width needs to be much wider to enable penetration than stipulated by the rule of thumb (Carlsson, 1998), (Eriksson et al, 1999). In Figure 3-42 the penetration difference between two cements are shown. As noted from the figure, the coarser cement can penetrate a width, which is 3 times the grain size, but the finer cement needs 8 times the grain size for penetration. This could also be expressed that there is almost no difference in penetration between the two different grain size distributions, both penetrate approximately a column of about 90 μm. Similar results have also been noted for a number of different types of micro cements and a possible explanation could be the chemical filtration or the clogging (Dalmalm et al, 2000), (Eklund, 2003).

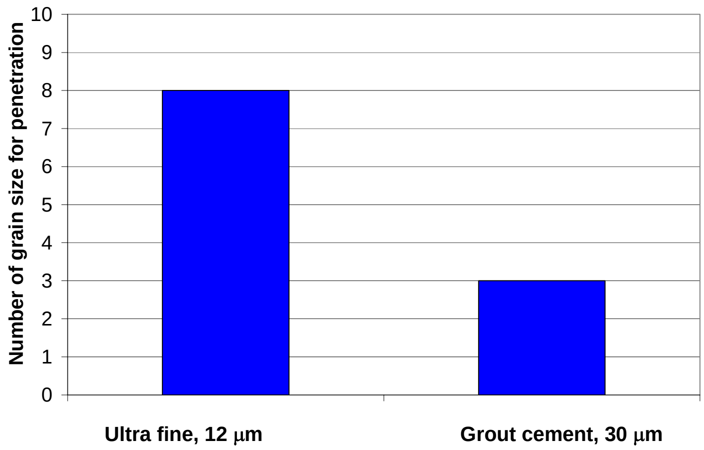


Figure 3-42 In relation to grain size, a fine-grained cement may need a larger aperture for penetration, than a coarser grained cement, (Carlsson, 1998), (Eriksson et al, 1999).

Eriksson (2002b) studied the grout spread from a borehole into a joint with a measured hydraulic aperture of 39 μm. The grout was based on a micro-cement with a grain size, d_{95} equal to 12 μm and a W/C ratio of 2.5. Three different relationship of contact area in the joint plane were assumed, 30 %, 50 % and 70 %. With a constant hydraulic aperture a higher percentage contact area means that the channel is wider, and vice versa. After grouting the joint was opened and studied visually and by digital scanning. The contact area could then be observed to approximately 50 percent, which means that the geometric joint aperture open for flow was wider than 39 μm. The spreading of grout from the borehole was only a few centimetres. The result is similar to other examinations of micro cements, which indicates that the rule of thumb, $d_{95} \geq b/3$, should be used with some caution (Houlsby 1990, Hansson 1994, Eriksson et al 1999) and that the channel aperture may need to be approximately $d_{95} < b/6$-$b/10$.

When comparing results regarding penetrability of grouts in relation to the aperture, it is vital to differentiate between the hydraulic aperture, b_{hyd} and the arithmetic mean aperture b. In literature, this is not always made clear and may therefore be a source of error upon which an erroneous judgment may be made.

Later developments by Eriksson (2003) have shown that the penetration of grout instead of relating to the grain size may be noted by the minimum b_{min} and the critical b_{crit} aperture of a grout mix. The density after a constriction gives an indirect value of the parameters b_{min} and b_{crit} as shown in Figure 3-43.

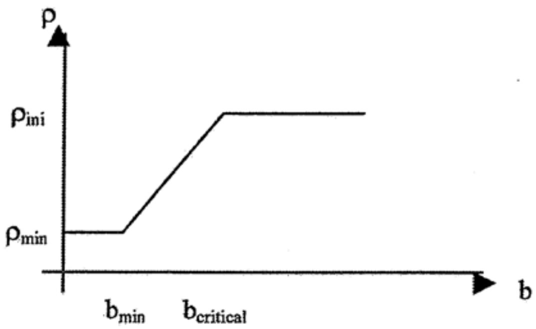


Figure 3-43 Illustration of the density intervals for filtered grout (Eriksson, 2003).

For penetration in a joint, the aperture has commonly been set to 3 times the particle size, which is an empirical value. Martinet (1998) showed that a vault consisting of more than 3 particles is unstable, which mathematically explains the empirical relationship.

Filtration can though occur even for relationships higher than 3 as shown by Eriksson (1998) who studied mechanical filtration in narrow pipes, with an inner diameter of 0.5 mm. The filtration was measured as the density difference before and after a constriction. The measured difference in density, reveal that filtration has occurred. The used grouts had a d_{95} of between 64 - 120 μm, which is equal to an aperture particle size ratio of 4 – 8.

The filtration of cement grouts was also noticed in joints with an aperture up to 10 times the maximum particle size (Hansson, 1995).

Different cements have different critical widths of penetration (Eriksson et al, 1999). In general, the critical width could be expressed as a joint aperture interval instead of a critical width. At apertures within the joint aperture interval the influenced factors have a large importance on the penetration. At apertures over and under the joint aperture interval the influenced factors have little importance, when varying within reasonable limits. With apertures larger than the joint aperture interval, the grout can pass without filtration and with apertures below the interval no grout can pass (Eriksson et al, 1999).

The NES equipment was used to test for repetition trials without cleaning the grouts. It was noted that even for grouts with high W/C ratio, which in a single test showed no filtration, the repetition test gave filtration (Carlsson, 1998).

According to Schwartz (1997), the filtering phenomena can be divided into mechanical and chemical filtration.

Mechanical filtration

Mechanical filtration is when particles of the suspension block the grout route. The filtration then starts when separate particles get stuck in the grout route and with time build a vault.

Filtration is thus a successive process with an increasing filter cake measured in size and extent. Schwarz (1997) describes the formation of the filter cake with a start of only a few particles and an initially high permeability. As the process continues more particles get stuck resulting in a decreasing permeability.

Chemical filtration

Chemical filtration is not a univocal concept. Studies by Lagerblad (1998) at the Äspö HRL showed that chemical alteration of grout during grouting has occurred. In the report, the chemical separation is described as easily soluble components of the grout which can be transported further than the remaining part of the grout.

Another type of chemical filtration is the one described by Schwarz (1997), which is dependent on increased, inter particle forces between particles, as for example fine grained cement. By using more and more fine-grained cements other phenomena, not studied earlier will affect the filtration stability of the grout (Schwarz, 1997). Schwarz also noted that the Van der Waals forces and the electric repulsion forces increase for decreasing particle size, which also influences the chemical filtration.

3.6.5 Grout mixing

The grout properties are strongly dependent on the mixing. The dispersion of the grout is dependent on the type of mixer and the performance. If the time and speed of mixing is defined, it is also necessary to specify the exact mixer to be able to estimate the quality of mixing. The dispersion of the grout has further been studied by Hjertström et al, (2003). The penetrability, measured with a filter pump, (75 µm, Hansson, 1994) of the grout mix was examined after mixing with a colloidal type mixer Craelius CEMIX 203. By increasing the speed of the mixer from 1450 to 1750 rpm during 4 minutes of mixing, the penetrability of the grout was increased by 80 %, as shown in Figure 3-44. By increasing the mixing time from 2 to 6 minutes, the penetrability was increased by 100 %. From the trials it was further concluded that mixing of micro cement (UF16) required much more mixing, than mixing of conventional grout cement (INJ 30).

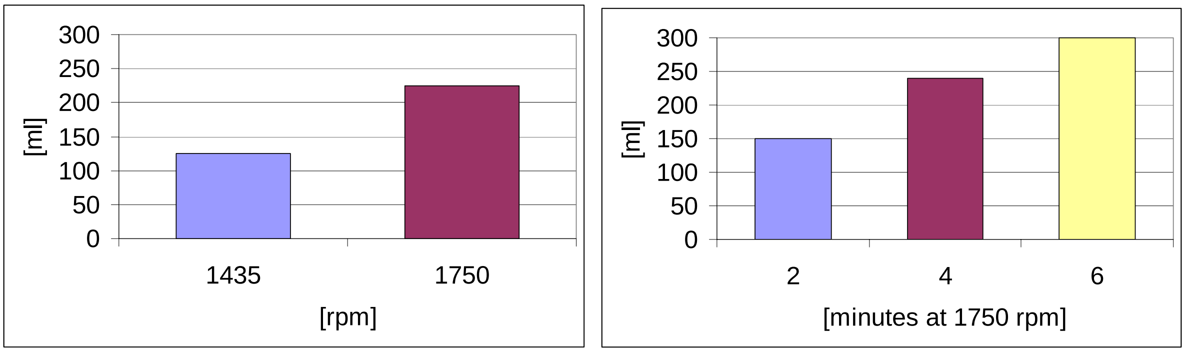


Figure 3-44 Penetrability of grout measured for two different mixing speeds and for three different mixing times, based on Hjertström (2003).

In a study by Axelsson & Turesson, (1996), the grout had a maximum value of stability after 2 to 4 minutes of mixing with a colloid type mixer (Craelius CEMIX 102 E) at 1450 rpm.

During the trials at the South Link it was noted that the mixing of the cement suspension were much more important than earlier understood, especially for the micro cement types. The penetrability measured with a filter pump was increased by as much as 75 % by using the laboratory mixer instead of a field mixer as reported by Dalmalm et al (2000), and shown in Figure 3-45. This emphasizes that properties defined from the pre-testing of the grouts, may not be the properties with which the rock mass are treated, which for some occasions may explain why expected sealing results were never reached.

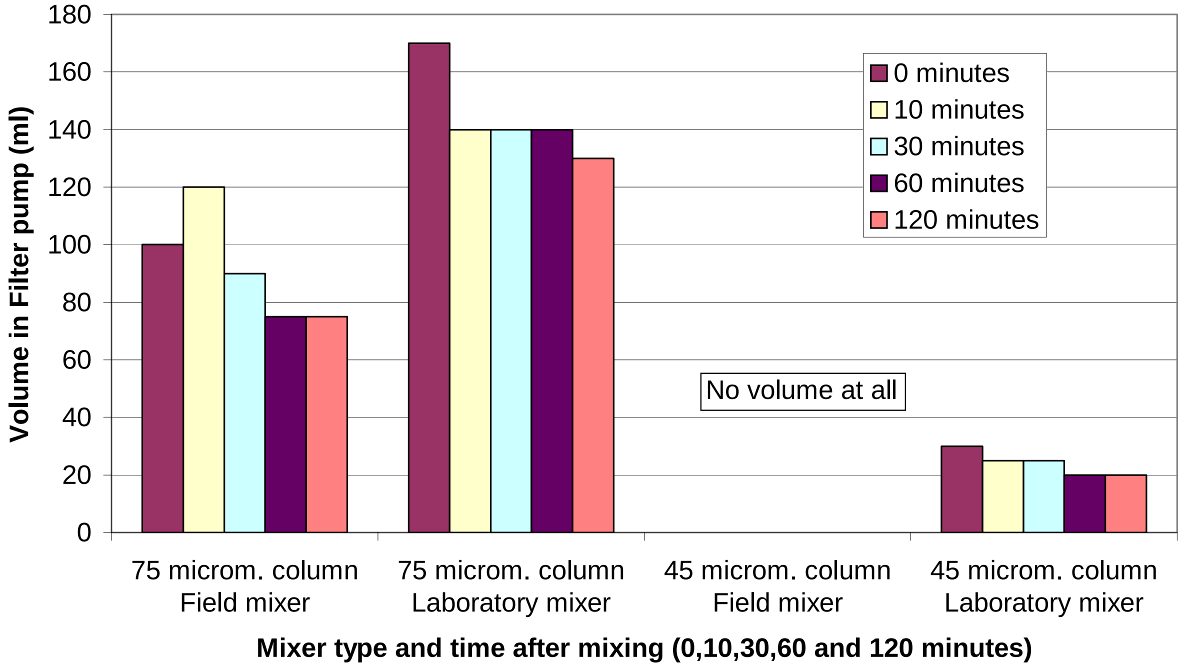


Figure 3-45 Penetration ability for grouts mixed with a field mixer and a laboratory mixer. The studied grout mix was based on Rock U with a d_{95} of 9.5 μm, (Dalmalm et al, 2000).

Similar results have also been shown by Johansen et al (1991) and from the pre-trials of the City tunnel in Malmö (Eriksson et al, 1999). The difference between a field mixer and a laboratory mixer measured with the NES equipment (Sandberg, 1997), resulted in a 100 % increase of the penetration ability, for the laboratory mixed grout as shown in Figure 3-46.

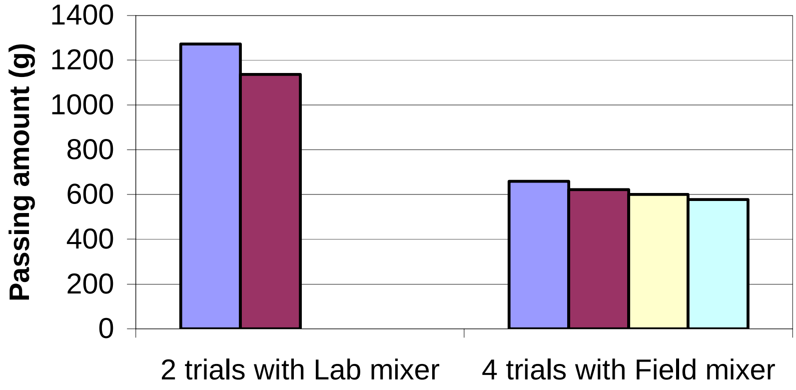


Figure 3-46 Penetration measured with NES equipment for a filter width of 100 μm. The used grout was based on Cementa Injektering 30, W/C ratio 0.8, 0.54% superplasticizer HPM, (Eriksson et al, 1999).

If the results reported here should be compared to real grouting in a rock mass the difference in volumes need to be regarded. In real rock the volume will be larger, which means that if a grout shows a tendency of filtration in the laboratory, there will be filtration and blocking of joints in a real rock mass, which may affect the sealing.

The grout mixing procedure is very important for the grout properties. Therefore, the producer recommendations should be carefully studied. To make inter-disciplinary discoveries can be useful. Grout mixing could for example be compared to: paint, pulp and medicine, and even the pastry and cake baking industry. Therefore, a start mix with only part of the fluid followed by mixing and thereafter adding the remaining part of the fluid was tested in the laboratory. The grout mix was based on Cementa Injektering 30 with a W/C ratio of 0.6 and with an additive of superplasticizer 1 % HPM. The compared mixing procedures were as follows:

Single step mixing: Cement and water with a ratio of 0.6 were mixed for 2 minutes, thereafter the superplasticizer was added and the mixing continued for another 2 minutes. The total mixing time was 4 minutes.

Two step mixing: Cement and water with a ratio of 0.35 were mixed for 1 minute, thereafter water was added to a ratio of 0.6 and mixing continued for another 1 minute then the superplasticizer was added and mixing continued for another 2 minutes. The total mixing time was 4 minutes.

As shown in Figure 3-47 the two step mixing procedure resulted in much higher penetration measured with a filter pump. Different filter widths, not shown here, were used during the trials with similar results. For the first trial, the start W/C ratio was 0.3, which resulted in even better penetration, as shown in the figure. Due to problems with a stiff mix the ratio was increased to 0.35 for the second and third trial. In Figure 3-47 the filter pump used during the trials are also shown.

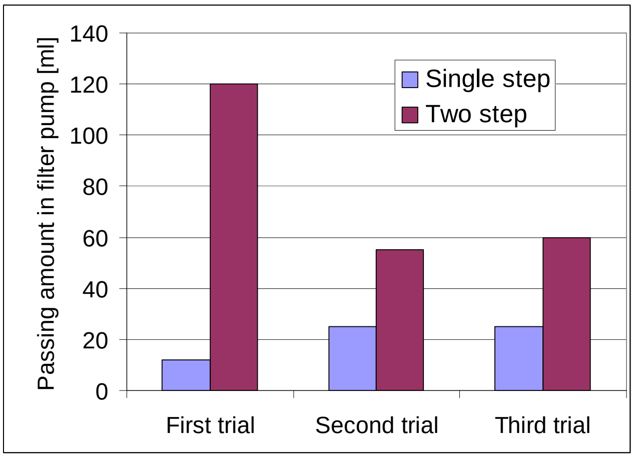


Figure 3-47 Measuring with a filter pump (Dalmalm, 2001). (left) Penetrability of grout compared for single- and two step mixing, measured with a 75 μm filter pump. (right).

3.7 Waiting

3.7.1 General

The total time for sealing a tunnel depends both on productive and non productive time. Extensive work has already been carried out to optimize the different operations categorised as productive time. In a study presented by Koskela (2000), the production at a construction site in general was studied. Only 33 % of the working hours were spent on productive work while 67 % of the working hours were spent on non productive work. This accentuates the importance of focusing on the non-productive time, which in the following will be denoted as waiting time.

The waiting time during sealing of tunnels can further be categorised in two groups:

- Waiting within operations
- Waiting between operations

The waiting within operations is, for example, waiting for mixing or waiting for pressure to stabilize during a water pressure test.

The waiting between operations could, for example, be that production can not start because equipment needs to be established or when waiting for the grout to harden. These activities will be further regarded as waiting time within this work.

3.7.2 Establishment time for different operations

The grouting process can, as shown in chapter 2, consist of a varying number of operations. Each operation will generate a time for establishment. The time can be short if operations are well planned or long if poorly planned. The time for establishment has not been regarded separately in chapter 2, but will be in the following calculations in chapter 4 and 5.

3.7.3 Hardening time for grout mixes

The hardening time is very dependent on the grout mix and can vary between ~0.5 – ~20 hours to reach, for example, 12 kPa. In Figure 3-48 an example of some hardening times are shown for three different kinds of cements. As noticed, the time depends on the type of cement, the W/C ratio and additives such as accelerators.

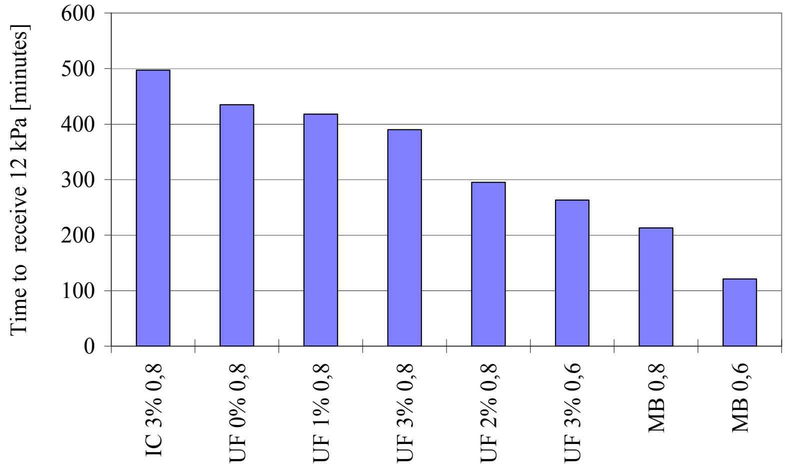


Figure 3-48 Average hardening time for different grout mixes. IC = Cementa grout cement (64 μm), UF = Cementa Ultra fine cement (16 μm), MB = Master builder cement. The W/C ratio was varied between 0.6 and 0.8 and the percent numbers indicate that accelerator has been used (Dalmalm, 1999).

In chapter 2, the grout hardening time from different projects was noted to be between 4 – 5 hours, depending on the grout mix used. It was also noted that the waiting time for hardening of the grout on average was 17 % of the total sealing time.

The time for hardening of the grout is extensively studied in literature, but the relationship between hardening time of the grout and waiting time before drilling can start, is not so well studied.

3.7.4 Waiting time before drilling

The waiting time before drilling can start after grouting is based on the relationship between the shear strength of the grout and forces on the grout caused by flushing of water from the drilling process. So far the available information regarding the relationship tends to be very limited.

The grouting process, from a flushing destruction point of view, can be described as:

The joints of the rock mass have during grouting been filled with grout, the time for grouting depends on the number of holes that are to be grouted, the different grout holes have therefore to different extent reached some shear strength. The question is how much strength they need and how much flushing they will be exposed to.

Today's design of grouting works assumes that a specific strength of the grout needs to be reached, because flushing can destroy the previous grouting. In Figure 3-49 a possible situation for flushing of grout by drilling is illustrated.

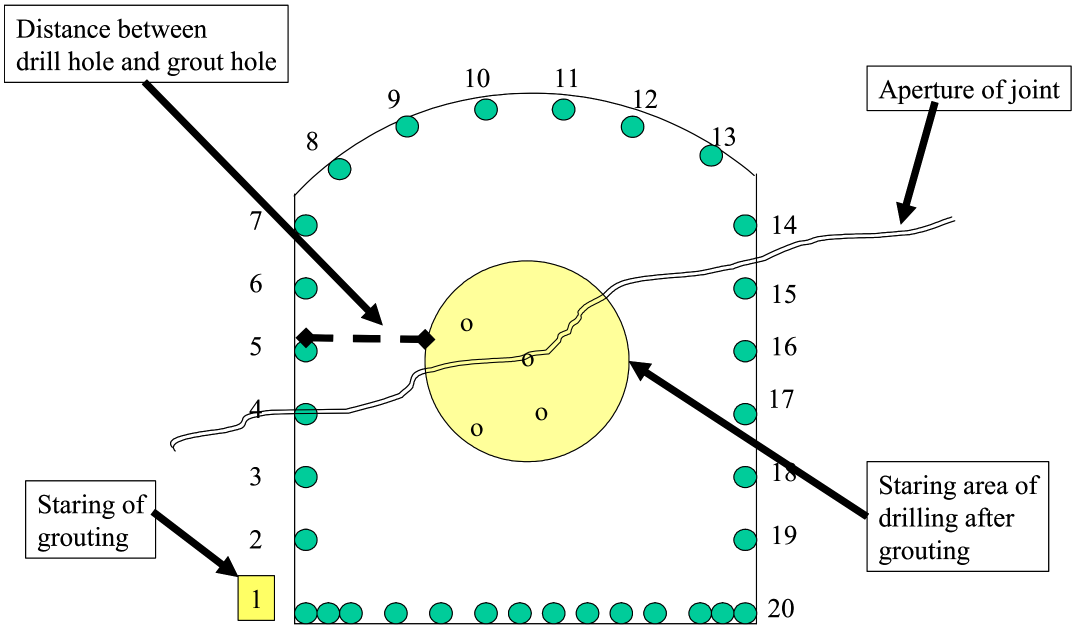


Figure 3-49 A possible grout flushing situation for a tunnel.

For an assumed tunnel with a width of 12 meters and with a height of 8 meters, the distance between a grout hole and the initial drilling for excavation could be around 3 meters. The total drilling time for a 100 m^2 tunnel could further be assumed to around 4 hours, which gives 3 hours of hardening time for the grout, before the drilling gets close to the grouted area.

The grouting is further commonly performed with a ~5 meters of overlap, which is equal to that the drilling of blast holes, directly after grouting, are performed in an area which has been grouted twice, with the earlier grout fan at some distance (~5 meters) out from the tunnel periphery.

In a study by Johansson (1997), the flushing resistance of cement grouts in a joint model was examined. The joint was modelled by a 520 x 520 mm^2 box, which on both sides were cover by a 10 mm thick Plexiglas, to study the destruction of the grout. The studied joint width could be varied, but for all results reported here the width was set to 10 mm.

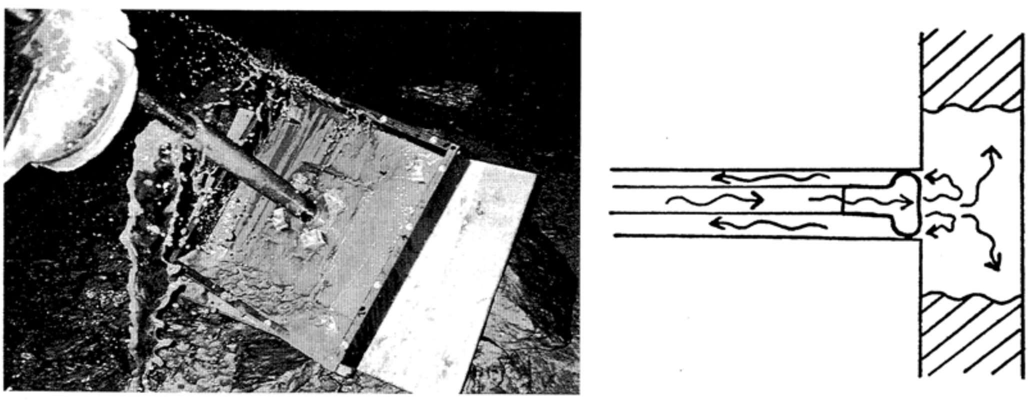


Figure 3-50 Equipment and principle layout for testing of grout flushing resistance, Johansson (1997).

The destruction of the grout was studied and as shown Figure 3-50. A clear difference in destruction behaviour was noted in the region of 11 kPa. Below 11 kPa, the grout was destroyed particle by particle, and above 11 kPa the grout was destroyed as aggregates. The phenomena with aggregates falling apart would not be possible in a rock mass where clean plane Plexiglas surfaces aren't used. The recommendation could be that the required shear strength should be at least 11 kPa, before starting to drill.

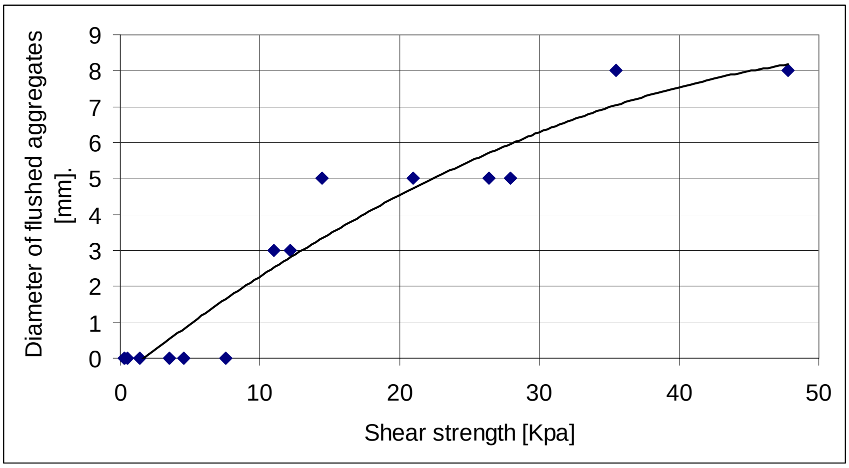


Figure 3-51 Diameter of the aggregates which are flushed out from the joint model, based on Johansson (1997).

In Figure 3-52 the size of the destructed area is shown together with the achieved shear strength of the grout. For the very low shear strengths of below ~0.5 kPa the destructed area was extended to a diameter of 300 mm. For shear strengths of above ~10 kPa the destructed area was extended to a diameter of 100 mm or less. The conclusion is that if 300 mm destruction zone could be accepted, then the waiting time for micro cements with a low W/C ratio could be decreased to around 2 hours, which is equal to around 1 kPa and if the accepted zone of destruction is set to 100 mm, a waiting time of between 4 and 5 hours is necessary, which is equal to around 10 kPa.

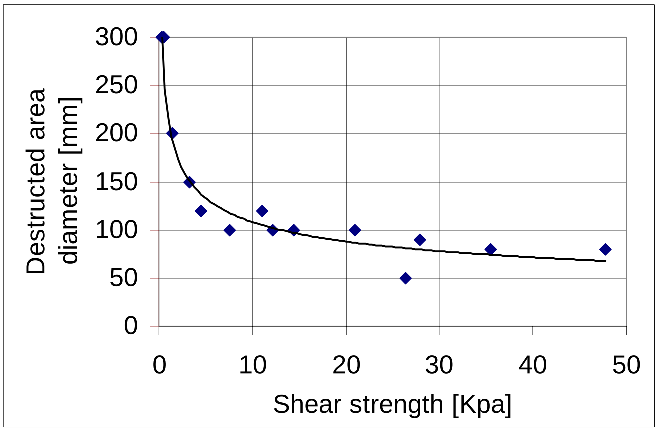


Figure 3-52 The destructed area as a function of the grout shear strength, based on Johansson (1997).

The studied joint width of 10 mm is considered as very wide compared to commonly occurring joint widths in a rock mass. The wide joint width was chosen to be on the safe side, as wide joints will be flushed much more easily than narrow joints. In addition, the real rock mass commonly has a much rougher surface than the Plexiglas. The breaking up of grout aggregates, as noticed during the trials would never be possible in a real rock mass, as the real rock mass is both rough and undulating.

Based on the results from the test of flushing in a Plexiglas, the question arises whether waiting time before starting to drill is necessary at all, if drilling starts at a distance from grouting. Further studies of flushing of grout are required, as the possible time saved by using the right waiting time may be considerable.

3.8 Grouting prediction and modelling

3.8.1 General

The purpose of a grouting prediction system is to, in advance of an occurrence, be able to decide the grouting necessary to fulfil the requirement.

A grouting method is dependent of a number of parameters. Some of them can be easily defined; others can only be described to a varying degree of uncertainty. To predict the outcome of a grouting method the uncertainties need to be quantified.

The prediction of groutability from grout properties and hydrogeological data was studied by Gustafson & Stille (1996). The study consists of theoretical developments as well as an evaluation of the theoretical findings from practical situations at the Äspö Hard Rock Laboratory.

The importance of aperture variation was shown from expressions of the maximum penetration length, I_{max}, and the grouted volume, V_g, according to Gustafson & Stille (1996) as:

$$I_{\max} \sim \frac{(P_g - P_W)}{2 \cdot \tau_0} \cdot \left(\sum b^3 \right)^{1/3} \qquad \text{Equation 3-29}$$

$$V_g = \sim \left(\frac{(P_g - P_W)}{2 \cdot \tau_0} \right)^2 \cdot \pi \cdot \sum b^3 \qquad \text{Equation 3-30}$$

where,

P_g	grouting pressure	[Pa]
P_W	water pressure	[Pa]
b	joint aperture	[m]
τ_0	yield value	[Pa]

A prediction system could be more or less comprehensive. Today there are a few systems, which to varying degrees could be recognised as a grouting prediction system (Hässler, 1991), (Janson, 1998), (Eriksson, 2002). The systems predict part of the grouting process, but none of them predicts the sealing time for a grouting method. Some systems for prediction are briefly presented here as a base for the evaluation of grouting methods and the prediction of sealing times presented in chapter 4.

3.8.2 Predicting the grout spread of Bingham fluids

Models and theories for the calculation of grout spread have been developed by, for example, Wallner (1976), Hässler (1991) and Eriksson (2002).

In 1991 Hässler developed a theory to calculate and simulate the grout spread of Bingham fluids and a geometrical model for the classification of the rock mass joint system. In a one dimensional case the flow in a joint plane could be calculated as a function of the penetration length. The theory assumes:

- Plane parallel joints
- Laminar flow
- Constant channel cross section (does not change by grout pressure)
- Sharp boundaries between different fluids in the same channel
- Incompressible fluids

For a Bingham fluid, a rigid core forms around the centre of the joint, if the shear stress is less than the yield value as illustrated in Figure 3-53.

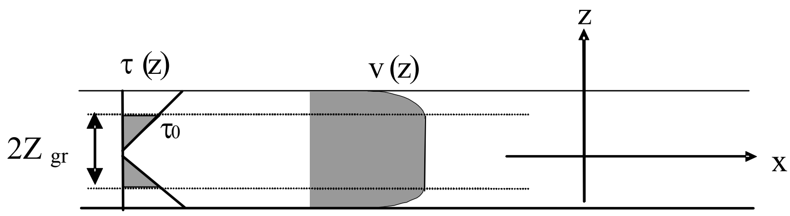


Figure 3-53 Velocity profile for Bingham flow in a joint plane. A solid core of thickness 2 Z_{gr} is located at the centre of the joint (Eriksson, 2002).

The thickness of half of the core, Z_{gr}, could be calculated using Equation 3-30.

$$Z_{gr} = \min\left[\frac{\tau_0}{b \cdot \rho_w \cdot g \cdot \left|\frac{h_o - h_l}{L}\right|}, \frac{1}{2}\right]$$ *Equation 3-31*

where,

b	joint aperture	[m]
τ_0	yield value	[Pa]
ρ_w	density of water	[kg/m^3]
$h_0 - h_l$	pressure difference in x direction	[m]
L	penetration length	[m]

When Z_{gr} is equal to b/2 the core totally occupies the joint and grout flow stops as the imposed shear stress over the joint section is less than the yield value of the grout. The maximum penetration length *I* could then be calculated as:

$$I = \frac{\Delta p \cdot b}{2 \cdot \tau_0}$$ *Equation 3-32*

Where, Δp is the excess grout pressure.

It should be noted here, that Equation 3-32 has only been expressed for a grout with a constant yield value, τ_0. In a reality the yield value will vary with, for example, the age of the grout, which will affect the maximum penetration length.

In a two dimensional case, the rock mass joint system was characterised and expressed as an α value, representing the conductive part of a joint plane, according to the "Piece of cake model", shown in Figure 3-54. The flow in a joint plane could then be calculated as a function of the penetration length and the characterised rock mass value, α (Hässler, 1991).

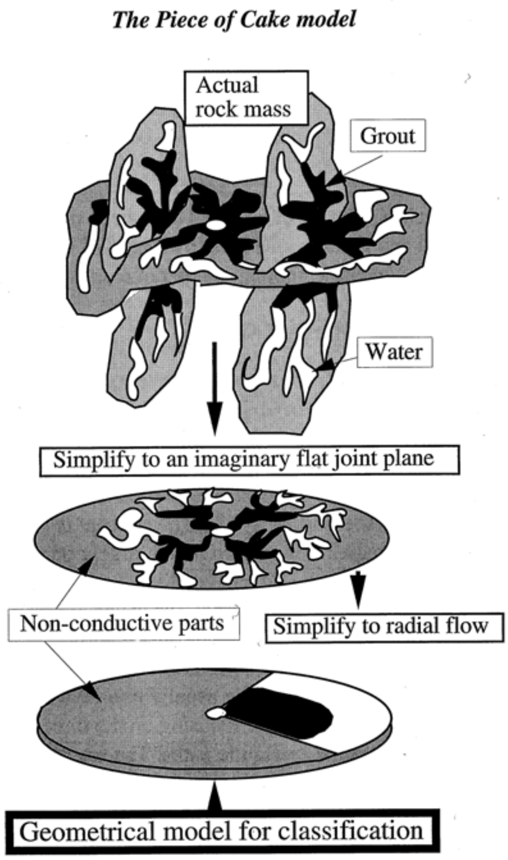


Figure 3-54 Geometrical classification of the rock mass according to the "Piece of cake model" (Hässler, 1991).

3.8.3 Model for predicting the grout spread and the result of grouting

In a study by Eriksson (2002), a model for predicting the time dependent spreading of grout and the sealing result in a single joint, based on the work by Hässler (1991), was presented.

In the model, the joint plane, which should be grouted, is represented by a network of elements where the elements all have individual properties. The model assumes that the rheology of grout resembles the Bingham model and that the grout has limited penetration ability. The computation involves numerically solving the equations for the flow of water and grout in the network and is based on the work presented by Hässler (1991), Håkansson (1993) and Wallner (1976).

The geometrical representation of a single joint is made by an orthogonal network of conductive elements. The realisation of a network is based on given values concerning dimensions, aperture distribution, boundary conditions and position in space in the form of dip, strike and depth. In Figure 3-55 a visualisation of the joint in 3D is shown.

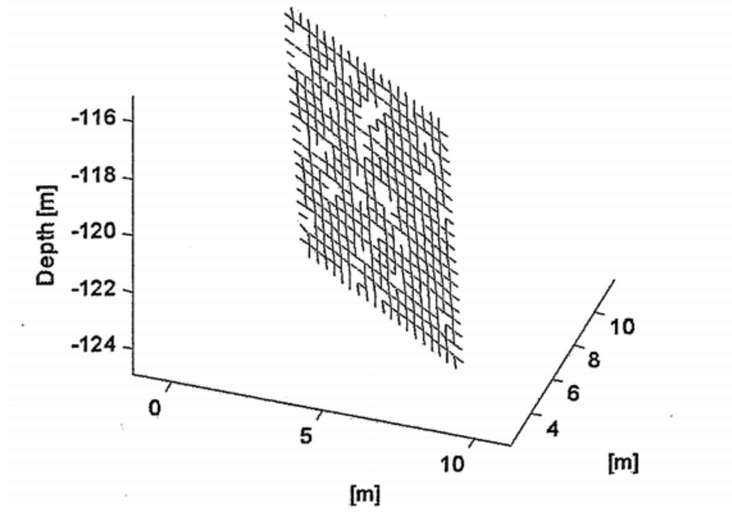


Figure 3-55 An illustration of the network in 3D representing a joint, where white areas are contact areas without conductivity (Eriksson, 2002).

The distribution of aperture fields is obtained by randomly distributing widths to the nodes in the generated geometry according to a log-normal distribution. The boundary conditions that can be stipulated are basically no flow conditions and constant pressure. The contact length of a grout hole depends on the hole diameter and how the holes strikes the joint. The length of the conductive elements that are in contact with the grouting hole are adjusted in length in order to establish a fictive hole radius.

Studies of the penetrability of grouts have resulted in the knowledge that filtration of the grout can occur when a constriction in the flow path is met. Based on measuring the parameters b_{min} and $b_{critical}$ of the grout, the restricted grout penetration due to filtration is regarded in the method.

Studies of bleed for different grout mixes have resulted in knowledge, which has been incorporated into the model. Conceptually the sealing effect is relative to the bleed in the calculations, i.e. 0 % bleed results in a 100 % sealing, if the element in the network is grouted. The bleed after filtration is also regarded.

The flow of grout in the joints is not only restricted due to the parameters of the joints and of the grouts. Some practical aspects can also influence the flow, as for example the capacity of the pump and the refusal criteria. These restrictions are treated as boundary conditions within the calculation of the grouting result. For a more detailed presentation see Eriksson (2002).

The model has been tested both in the laboratory and in the field. In Eriksson (2002) it is stated that the model is accurate when the flow geometry is known. In the field, the exact geometry of the joints are not known, which is why a probabilistic approach for estimating a span of possible grouting results is proposed. This approach has proven valuable in a small field test (Eriksson, 2002) and for an actual grouting project (Emmelin et al, 2004).

3.8.4 Predicting the grout take

For a joint with constant aperture, a simple expression for the final penetration and therefore the maximum volume can be obtained. In reality, however, a joint is not absolutely flat; the aperture varies and is difficult to determine, it is not of infinite length and contains contact areas and filling material, so that a simple expression for maximum volume cannot be used directly in practice (Lombardi, 1985).

Three calculation models for grout take were developed by Janson (1996), based on theories from Hässler (1991) and Håkansson (1993). The models are valid for grout take in the first grouted hole of each fan. According to Janson, (1993) and Sturk (1996), the first hole normally takes between 10 and 25 percent of the total grouted volume of a normal fan, if the grouting starts in the hole with the highest Lugeon value.

Model I, is based on the assumption that grouting takes place in flat circular discs of constant aperture. The discs are intersected at its centre by a drill hole, and the grout from the drill hole fills the disc symmetrically.

Model II, is based on the assumption that grouting is assumed to flow out in a system of channels into the rock. According to Hässler (1991) the system of channels are simplified, by a projection, into a plane with a degree of openness for grouting, and an α value, representing the conductive part of a joint plane,.

Model III, is based on the principle that grout flows out from the drill hole into a system of circular discs, as shown in Figure 3-56.

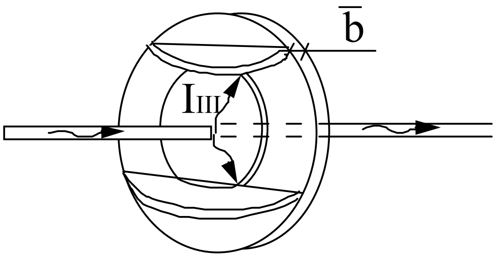


Figure 3-56 Penetration takes place in a system of discs (Janson, 1998)

Model III is a development of model I, with a more detailed description of grout penetration in rock when compared with the previous models. The geological variables ψ, κ, λ, θ, β can be estimated, e.g. on basis of a rock survey.

The expected volume can thus be calculated from the transmissivity T and estimated values of the five geological variables, according to Janson (1998), as shown in Equation 3-33.

$$V_{III} = \left(\frac{\Delta P}{2 \cdot \tau_0}\right)^2 \cdot \frac{12 \cdot T \cdot \mu_W}{\rho_W \cdot g} \cdot \frac{\psi \cdot \kappa^2 \cdot \theta \cdot \beta \cdot \pi}{\lambda^2} \qquad \text{(Equation 3-33)}$$

where:

ΔP	difference between the grouting pressure
τ_0	yield value
ρ_w	density of water

μ_W	viscosity of water
ψ	number of grouted joints
κ	channel aperture
λ	grout path in rock
θ	average aperture
β	secondary joint plane

The model III was verified with grout volumes measured at the Äspö tunnel in Sweden and at the LaFortuna dam in Panama, as shown in Figure 3-57.

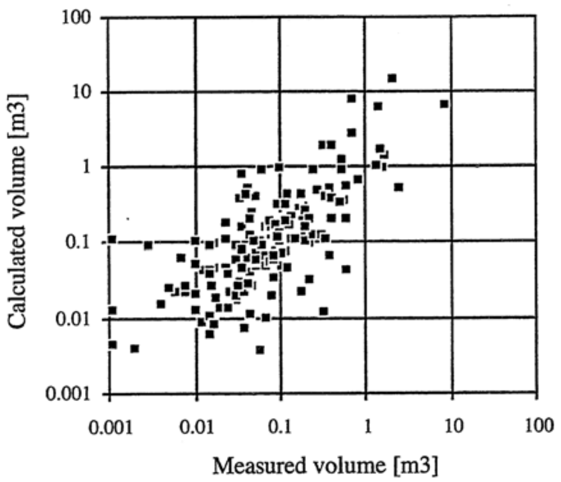


Figure 3-57 Comparison of measured and calculated volumes on the basis of model III for the Äspö tunnel in Sweden and the La Fortuna dam in Panama (Janson, 1998).

As noticed from the Figure 3-57, there is a relationship between the calculated and the measured grout take according to model III by Janson (1998).

A modified Q for grouting Q_i was presented in 1997 by Johansen et al, as shown in Equation 3-34. The Q_i is based on the Q-system for rock mass classification and complemented with a Lugeon value.

$$Q_i = \left(\frac{RQD}{J_N}\right) \cdot J_R \cdot J_A \cdot \left(\frac{1}{1 + Lugeon}\right) \qquad \text{Equation 3-34}$$

where:

RQD	Rock Quality Designation
J_A	Joint alteration number
J_N	Joint set number
J_R	Joint roughness

An increased Q_i –value means that less grout, expressed as kilogram cement per meter of tunnel, is needed for sealing. In Figure 3-58, grout take for some tunnels in the Oslo area are shown. The grout take, measured as kilogram cement per meter of tunnel, has been plotted against the Q_i -value. From the figure it could be noted, that for a Q_i -value of 10, the amount of cement in general varies between 50 and 400 kilogram and with some outliers of 1000 kilogram. The variation is large, which may give poor result if grouting prediction should be based on this model. Personal communication with Löset (2001) confirmed the above conclusions. The intention of the model was to test if further development would be valuable or not in solving the grouting prediction problem.

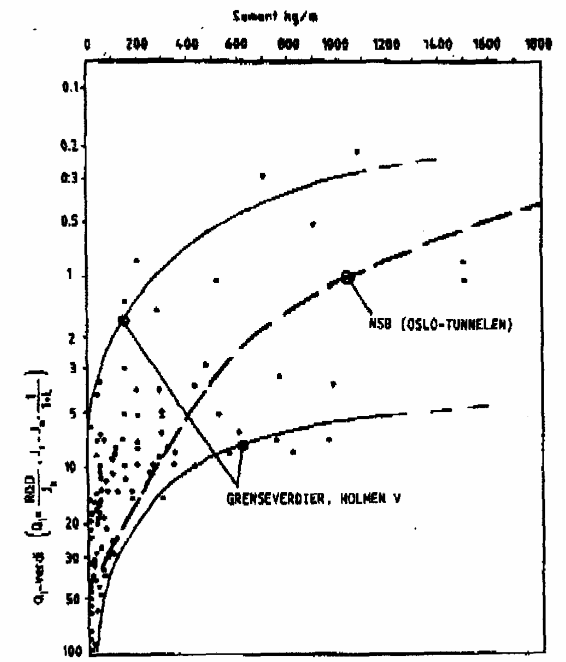


Figure 3-58 The Q_i value on the y-axis is shown against the amount of cement used in kg/m tunnel on the x-axis.

During the excavation of the Hallandsås tunnel in southern Sweden, the rock mass was classified according to the Q-system. The correlation between the acrylic based grout materials and the Q-index were studied with three different statistical methods. It was not possible to prove any statistical significance between the amount of grout used and the Q-value. The correlation was in the range of 0.11 to 0.17 and is shown in Figure 3-59.

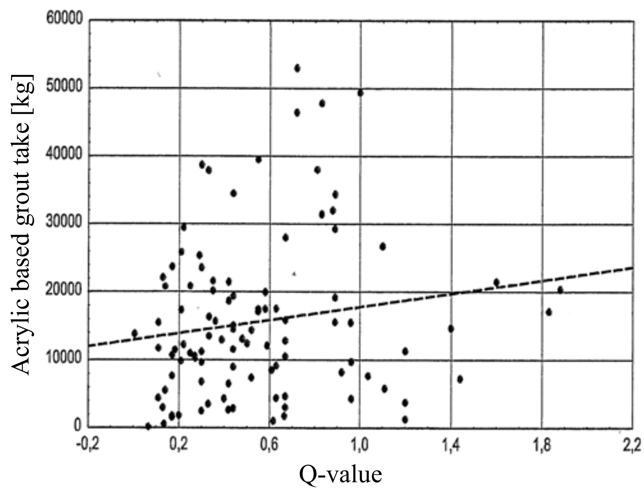


Figure 3-59 Correlation between Q-value and amount of used the acrylic based grout material (Miljögranskningsgruppen, 1998).

The correlation between the Q-value and the flow log [litre/minute] for each separate grout hole was low, but better than the Q-value/amount of used grout correlation, as shown in Figure 3-59.

3.9 Conclusions

3.9.1 General

The different activities are, to a varying degree, dependent on the rock mass joint system. The joint system can be analysed by different methods, whereof packer tests in holes are common. The purpose with a packer test can, for example, be to examine the transmissivity of the joint system and the connection between the different joints.

The inflow to a tunnel is dependent on the initial rock mass conductivity, the water pressure and the conductivity of the grouted zone. The thickness of the grouted zone is of minor importance, it is instead the conductivity of the grouted zone which mainly affects the inflow to the tunnel.

The water inflow to a tunnel can be calculated based on Thiems well equation, adjusted for the present situation regarding the pressure loss in the near field. However, the uncertainties may be high as the evaluation of the conductivity is rather difficult.

As the rock mass consist of a discrete network of joints, the studied number of joints needs to be large if the conductivity of the rock mass is to be expressed based on the measured transmissivity from the individual joints.

The arithmetic mean joint aperture is regarded as log-normal distributed and is very important for the possibility to seal the rock mass by grouting. When the hydraulic aperture is evaluated based on water loss measurements, the method of evaluation should be chosen carefully as different joint situations needs different methods for evaluation.

The necessary number of grout holes for a grouting fan depends on the openness of the joint planes, the geometric relationship between the grout holes and the joint, the grout mix and the grouting pressure. An optimal number of holes may be decided based on trial and error or by a model.

The sealing effect is dependent of the grouting order as joints may be blocked and made unreachable due to an unfavourable grouting order. Different stop criterion will affect the sealing result as the penetration length is dependent on the stop criterion. Both the grouting order and the use of different stop criteria may be evaluated by a model.

Different situations may need different grouting methods. The prediction models presented in this chapter may be used to evaluate different parts of a grouting method. Development is needed whereby different methods can be evaluated and compared for different situations.

3.9.2 Time

The time to seal a rock mass to a certain requirement level depends on a number of activities and circumstances. In chapter 2 the activities were grouped into five main activities: drilling, grouting, waiting, probe/inspection holes and re-grouting. The main activities have been further discussed in this chapter.

The time for the activity *drilling* depends on the number of holes, the hole length and other activities connected to drilling such as cleaning of holes etc. The time for drilling also depends on the rock mass properties and the capacity of the drilling equipment. The capacity for drilling depends on, for example, the type of drill rig, the hole diameter, the hole length and the rock mass properties. The capacity has earlier been studied in literature, and is left out of this study.

The activities *grouting* or *re-grouting* have been modelled regarding the penetration length, the grouted volume and the maximum volume suitable for different situations. They have also been modelled regarding the grouting time for different situations. The time for grouting and re-grouting have, however, not then been connected to other activities of the sealing process.

In the numerical model by Eriksson (2002) the grouting- and re-grouting time of the individual joints can be calculated. The model regards: joint aperture distribution, openness of joint planes, joint lengths, grout properties, grout separation, grout filtration, grouting order, ground water pressure, grouting pressure. The grouting- and re-grouting time for a fan can then be calculated by adding the times from the individual joints. The model is therefore regarded as a good base to be able to evaluate different grouting methods, which is the subject of the next chapter.

The activity *waiting* regards general non productive time, time for establishment and hardening time. The waiting time could, depending on the grouting method, make use of a large share of the total time for sealing. The time of waiting for the grout to harden before drilling can start may not, based on the times of waiting shown in chapter 2 and the discussion from chapter 3, be regarded as adapted for the current situation. Each specific grout, each specific joint situation and each specific drilling layout needs to be regarded in order to decide a correct waiting time. One model has been found that discusses the waiting time, but no model has been found that regards the waiting time together with other activities of the sealing process.

The activity *probing* mainly depends on the decided method. If the properties of the rock mass vary and there are high sealing requirements, extensive probing may be necessary. If instead, rock mass properties are unchanged and the consequence of failing with grouting is not regarded as a serious problem, then probing may be limited. No model has been found that regards the time of probing as a part of the sealing time.

Chapter Four

4 Grouting method evaluation

In this chapter a system for evaluating different grouting methods is presented. The sealing time to fulfil the inflow requirement for the different grouting methods is predicted by the system. Three grouting methods are theoretically tested in three different hypothetical rock masses for evaluation.

4.1 Introduction

A grouting method can be evaluated in many different ways. For example, amongst others, the grouted volume, the grouting time and the cost for grouting could be compared.

The grouting method evaluation presented here has two purposes:

1. Emphasize differences between grouting methods.

2. Predict the grouting- and sealing time to fulfil the requirements and thereby enable the grouting to be optimized, with regard to the time needed.

Based on the studied projects in chapter 2, three different grouting methods are presented in this chapter. Parts of the methods have, in principle, been used earlier for grouting projects and other parts are simplifications and assumptions which are made to be able to evaluate different grouting behaviour.

Grouting method evaluation in real rock masses at real grouting sites has earlier been studied and reported in, for example, Dalmalm et al, (2000). Due to the large variation of rock mass properties and due to difficulties measuring the grouting result to a certain level of accuracy, numerical grouting models in hypothetic rock masses have here been considered advantageous for the comparison of different grouting methods.

Grouting can be performed in a number of different ways. If it is possible to choose the most advantageous grouting method, it should also be possible to optimize the grouting process. *Within this work, the most advantageous grouting method is defined as the method which gives a minimum sealing time and fulfils the predetermined requirements.*

The grouting method evaluation procedure is performed as:

1. Define one or a number of grouting methods

2. Present grout mix parameters

3. Define hypothetical rock masses for evaluation of the grouting method

4. Evaluate the methods, as further described in chapter 4.2.

5. Analyse the results

The evaluation is started by describing one or a number of grouting methods. The methods include information such as hole setting, number of holes, grout mixes, grouting pressure, criterion for completion.

In the second step the grout mix suggested within the grouting methods are evaluated and parameters such as separation, b_{min}, b_{crit} (chapter 3.6.4) and viscosity are presented.

The evaluation is then continued by describing properties of the hypothetic rock masses in which the evaluation should be performed. The rock masses have, within this project, been chosen to be able to discuss the usefulness of different grouting methods for different rock mass conditions. Each rock mass is described by e.g. the mean aperture, the aperture distribution, the aperture openness, the water pressure and the number of joints.

The grouting method evaluation is started by randomly selecting rock mass parameters from each chosen rock mass. The rock mass is then grouted according to the chosen grouting method. If the required criterion is fulfilled, the grouting time is recorded, otherwise re-grouting is continued according to the method.

The sealing time is then calculated based on the time to drill, the time of waiting, the time for establishment and the grouting time, complemented with re-grouting time and post-grouting time. The later two items being based on achieved sealing effect.

The grouting time for different grouting methods is calculated by adopting theories as described in chapter 3.

Different grouting methods may be optimized for different rock mass situations and for different inflow requirements, which should be regarded when discussing the result of the different grouting methods.

4.2 Method evaluation procedure

4.2.1 General

Each selected grouting method is evaluated for each chosen rock mass.

The joints belonging to each rock mass are, when calculated, grouped into aperture groups (e.g. 75-125 μm). For each aperture group, distributions for grouting time, grouting volume and sealing result etc. are calculated.

Evaluation of the grouting methods, have been divided into three levels to be able to study the different behaviour of the methods:

- The first level is *joint level*, which means that grouting time and sealing effect etc. are studied for each aperture group, with a randomized channel structure, chosen from the present rock mass.
- The second level is *fan level*, which means that the grouting time and the sealing effect etc. are studied for randomized fans with a length of 20 meters formed by randomized joints.
- The third level is *tunnel level*, which means that the randomized fans are added to represent a tunnel.

The sealing time distribution for the different grouting methods in the different hypothetical rock masses could then be evaluated as presented in Figure 4-1.

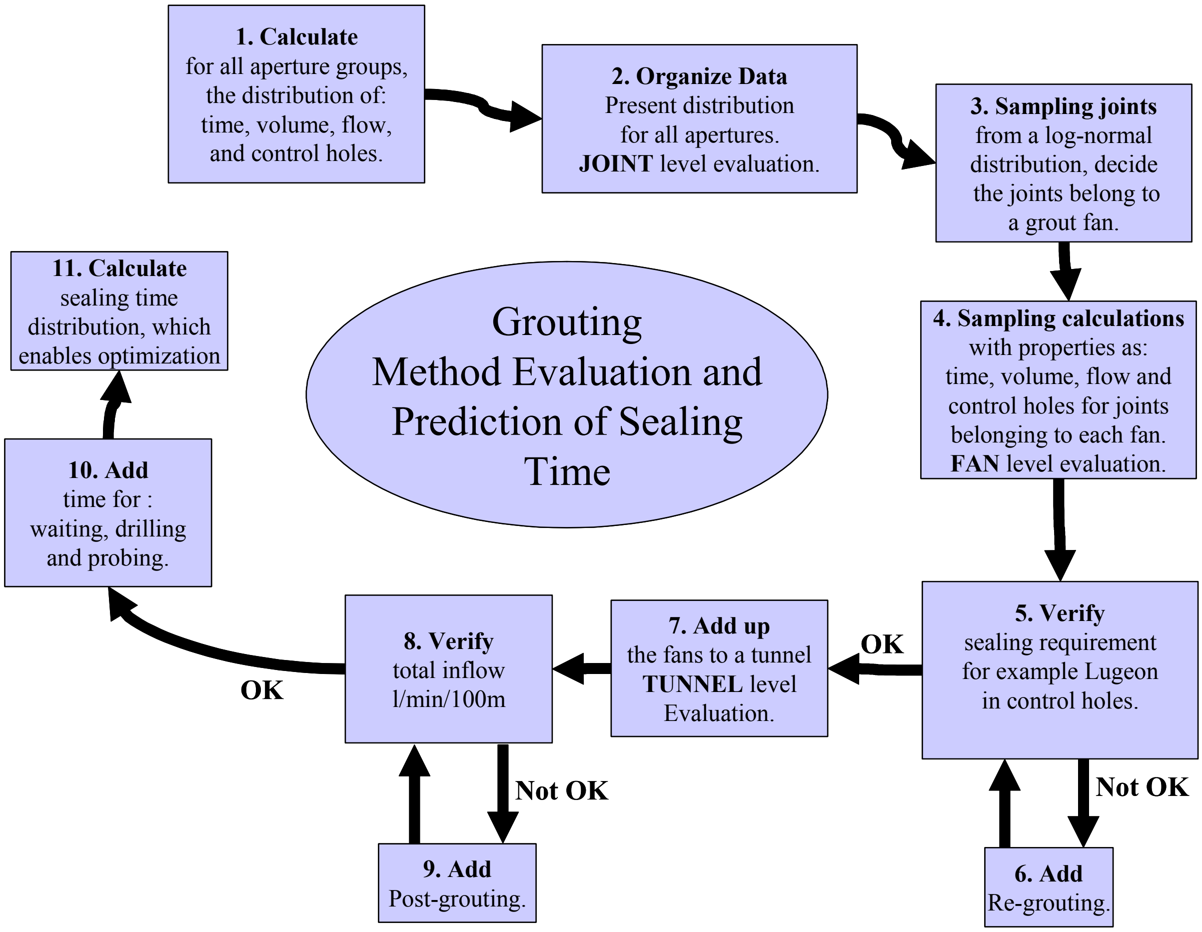


Figure 4-1 The procedure of Grouting method evaluation and prediction of sealing time.

1. For each type of rock mass, the randomized joint planes are divided into aperture groups, with an aperture multiple of 50 μm (e.g. 75-125 μm with mean aperture 100 μm). The distribution of mean apertures for the joint planes may differ from one rock mass to another according the rock mass description. For each aperture group the grouting time, re-grouting time, grout volume, re-grouting volume, flow before, flow after and Lugeon values from control holes before and after grouting and re-grouting are calculated.

2. The data from the joint plane calculations are organized and distributions for the different joint groups are presented as “comparison on joint level”.

3. Joints belonging to a grout fan are sampled from a log-normal distribution.

4. Parameters as grouting time, grouting volume, flow in joints and Lugeon values from control holes etc. are sampled from the in (1) calculated joint planes to the in (3) sampled fans. The result is presented as “comparison on fan level”.

5. Depending on the grouting method, different control methods may be used to verify the result.

6. Re-grouting is performed until the requirements are fulfilled.

7. The randomized fans are added up to the equivalent of a tunnel and the result is presented as "comparison on tunnel level".

8. The grouting time to reach a certain level of inflow is presented here. The total inflow is compared with the inflow requirement.

9. Time for post-grouting is, depending on the method, added until the requirements are fulfilled.

10. Time for drilling, waiting, probing and establishment is added.

11. The sealing time distribution for each grouting method is calculated (predicted). The sealing time can be a base for grouting optimization, as it enables comparison between different grouting methods.

Calculations on joint plane level (step 1)

For calculating each grouting method in each rock mass, a conceptual geometrical model of a joint plane, described in chapter 3 and developed by Eriksson (2002) has been used. The conceptual geometrical model for one joint plane has the dimension of 30 x 30 m^2 as is illustrated in Figure 4-2.

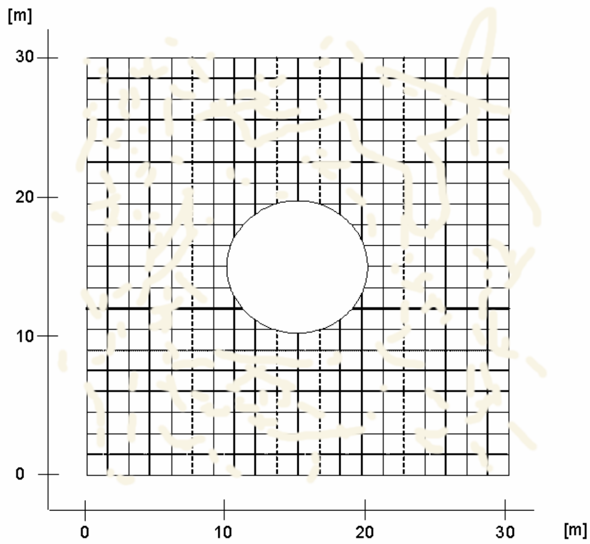


Figure 4-2 Illustration of the conceptual geometrical model for an exemplified joint plane used for the calculations, with dimensions 30 x 30 m^2. In the centre of the joint plane the circular tunnel is shown. The element size in the net has the size of 0.25 x 0.25 m^2).

The joint plane has open boundaries, representing its connection to other joint planes. A circular tunnel with the diameter of 9.7 meters is positioned in the centre of the joint plane. The flow can enter the tunnel as illustrated in Figure 4-3.

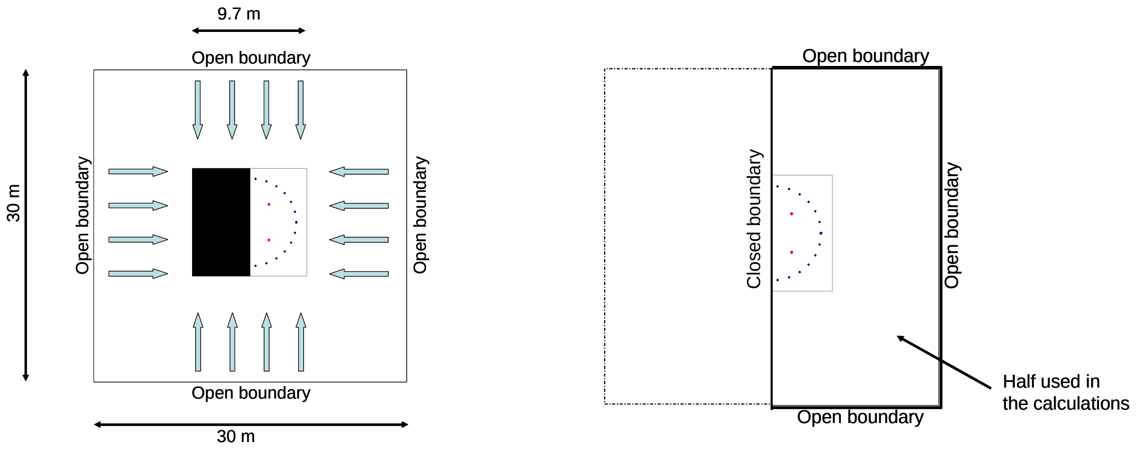


Figure 4-3 Conceptual model of one joint plane (left) and illustration of a half joint plane used within the calculations (right).

To reduce the calculations axisymetrical geometry has been used, the grouting methods are modelled and calculated in one symmetric half of a tunnel section as shown in Figure 4-3. The properties of each halves are assumed to be symmetric. Calculated values are then adjusted, as the purpose is to compare the inflows to the whole tunnel.

The model is fixed during grouting, which means that the joint aperture is constant during grouting. In reality grouting would to some extent affect the joint aperture because of the applied pressure. In the model this is not regarded, which is a limitation. This may affect the absolute values calculated with the model. The deformation of the joints depends on the grouting pressure, grouting volume and the penetration length. The comparison between grouting methods are regarded as un-affected by the chosen fixed model, as is normally assumed for grouting calculations.

In the presented result, grouting time and grouted volume for hole filling is excluded. The sealing effect is calculated as shown in chapter 3.1.2.

In the calculations of each joint plane the following scheme is followed:

- The un-grouted tunnel is excavated and the flow into the tunnel is calculated, as “cross joint flow” with full water head on the outer boundaries and zero head in the tunnel.
- The original model is recreated and then numerically grouted. The grouting time, grouting volume, re-grouting time and re-grouting volume are calculated.
- The grouted tunnel is excavated and the flow of the grouted section is calculated and compared to the flow into the un-grouted tunnel.
- Water loss measurement calculations in probe holes are performed before, between and after the grouting.

These calculations are repeated for different combinations of grouting methods and rock mass models. The filling out of the rock mass joint plane is illustrated in Figure 4-4.

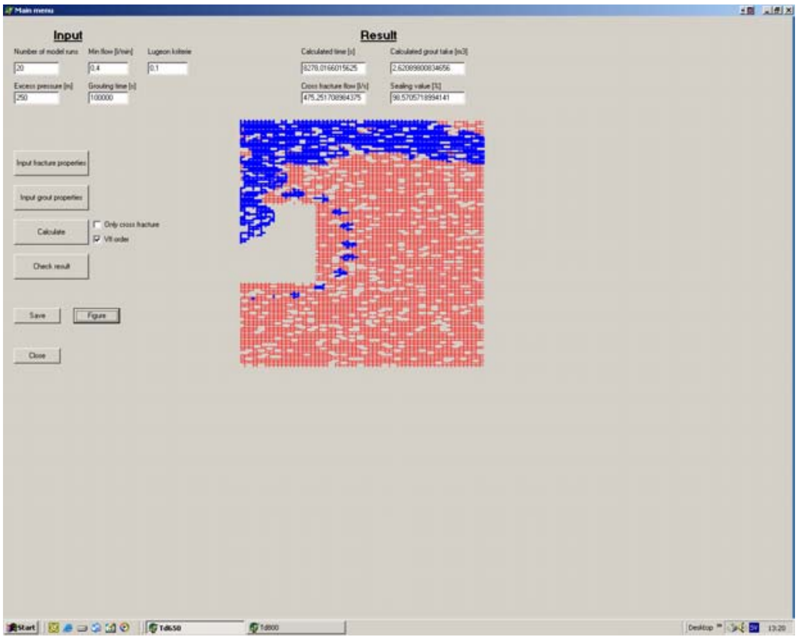


Figure 4-4 An example of a half joint plane model after numerical grouting. The open flow path of a joint plane is shown in red and the spreading of the grout is shown in blue.

The result from each joint plane is then expressed as shown in Table 4-1.

Table 4-1 Output reported from the calculations of each joint plane.

Grouting	Grouting	Re-grouting	Re-grouting	Inflow	Inflow	Inflow	Lugeon	Lugeon	Lugeon
Time	Volume	Time	Volume	Cross fracture flow	Cross fracture flow	Cross fracture flow	Control holes	Control holes	Control holes
				Before Grouting	After Grouting	After Re-grouting	Before Grouting	After Grouting	After Re-grouting

<u>Sampling calculated joint properties to a fan (step 4)</u>

A grout fan consists of a number of randomly selected joints planes which are individually grouted. The governing grouting time is taken as the longest grouting time for the joints belonging to a particular grout fan. As calculations are made for a half section, the water flow to the tunnel is taken as twice the sum for each fan and the grout volume is taken as twice the sum for each fan.

In the numerical model each joint is grouted individually. The defined methods have completion criteria per full length grout hole. The calculated values from the model are therefore compared to adjusted completion criteria and re-grouting criteria for each joint and for each tunnel section i.e. the values are divided by the average number of joints crossing the grout hole. The number of joints crossing differs for each fan in the hypothetic rock mass as described in chapter 4.4.

4.2.2 Sample size

A jointed rock mass can, for example, be described by the number of joints, the aperture and the joint direction. In the following, joint intensity is determined as an average number of joints per meter crossing the grouting fan. All joints are modelled as perpendicular to the tunnel. If the number of joints is defined as the total number of joints as shown in figure 3-5, the open and closed joints need to be separated, as it is only the open joints which are of interest here. The open and closed joints can be modelled as a Bernoulli distribution, as shown in Figure 4-5.

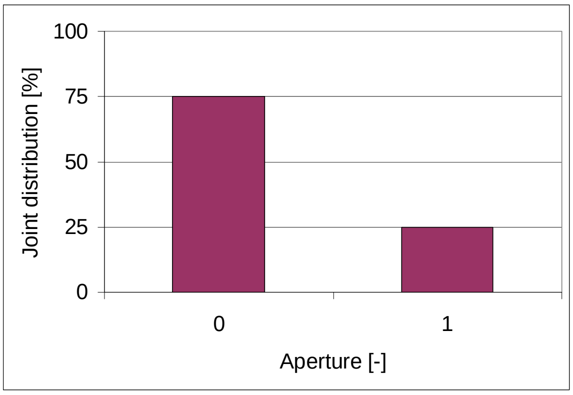


Figure 4-5 The Bernoulli distribution of open and closed joints for a rock mass with 75 % closed joints and 25 % open joints.

The mean μ and standard deviation σ can then be calculated based on the probability *p,* as shown in Equation 4-1 and 4-2:

$$\mu = p \qquad \text{Equation 4-1}$$

$$\sigma = \sqrt{p \cdot (1-p)} \qquad \text{Equation 4-2}$$

For the typical rock masses, which are further described in chapter 4.4, the mean number of open joints are defined. For rock mass A, which here will be used as an example, the expected number of open joints is defined as 5 for a fan length of 20 meters. Based on the Bernoulli distribution the probability of an open joint in rock mass A is 0.25 open joints per fan.

The Bernoulli distribution can then be used to sample the number of open joints of rock mass A, as illustrated in Figure 4-6.

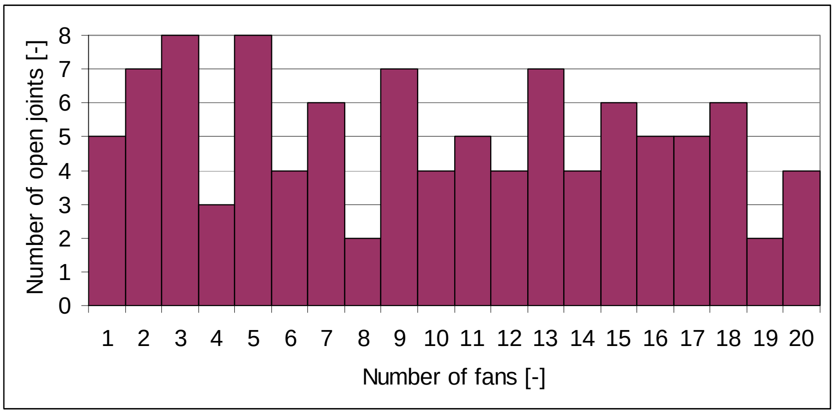


Figure 4-6 Example of discrete distribution of open joint intensity for a fan with a length of 20 meters in rock mass A.

From a number of Bernoulli trials the number of successes gives a discrete probability distribution or a Binomial distribution. If the number n is high the binomial distribution will approach the normal distribution (Weisstein, 2004). For the binomial distribution the mean μ value can be calculated based on the probability p and number of objects n according to Johnson (1994) as:

$$\mu = n \cdot p \qquad \text{Equation 4-3}$$

The variance of a binomial distribution can be calculated according to Johnson (1994) as:

$$\sigma^2 = n \cdot p \cdot (1 - p) \qquad \text{Equation 4-4}$$

The variance σ^2 of open joints in rock mass A can, based on Equation 4-4, be calculated to 3.75.

The probability to get an exact number of joints in a fan can be calculated for the binominal distribution according to Johnson (1994) as:

$$b(x; n; p) = \binom{n}{x} \cdot p^x \cdot (1 - p)^{n-x} \qquad \textit{for } x = 0,1,2,....n \qquad \text{Equation 4-5}$$

The probability to get exactly 5 joints for a fan with a length of 20 meters can then, based on Equation 4-5, be calculated to 20 %, which corresponds to the example in Figure 4-6.

In the simulation we want to have a rock mass which has a mean of 5 open joints per fan. Thus, we must define what is representative (confidence interval) and calculate the necessary number of fans to get this mean for a certain level of confidence.

The necessary number of fans can then be calculated according to Johnson (1994) as:

$$n = \left[\frac{z_{\alpha/2} \cdot \sigma}{E} \right]^2 \qquad \text{Equation 4-6}$$

Where,

n	number of grout fans
$z_{\alpha/2}$	1.96 for a normal distribution with known variance
σ	standard deviation
E	single side confidence interval

For large numbers and for normal distributions with known variance the variable $z_{\alpha/2}$ for 95 % confidence equals 1.96. For smaller numbers than 30 and with estimated variance the variable $z_{\alpha/2}$ is chosen from the student "t" distribution (Johnson, 1994). According to Johnson (1994), "The assumption that the sample must come from a normal population is not so severe a restriction as it may seem. Studies have shown that the distribution of the random variable is fairly close to a *t* distribution even for samples from certain non-normal populations".

With confidence levels of 95 % and 99 % ($z_{\alpha/2}$ (95 %) = 1.96, ($z_{\alpha/2}$ (95 %, n=17 fans) = 2.110, $z_{\alpha/2}$ (99 %) = 2.576) and σ = 1.936 the different number of fans can be calculated for different degrees of confidence and interval as shown in Table 4-2

Table 4-2 Number of fans to fulfil the 95 % and 99 % confidence level for different single sided confidence intervals.

Confidence level	Length of single sided confidence interval 0.1	0.2	0.5	1.0
95 %	1440	360	58	17
99 %	2488	622	99	26

To be within an area with similar rock mass properties it is assumed, that if the mean number of open joints for a fan is 5, the acceptable variation is between 4 and 6 open joints for the studied fans.

With a single sided confidence level of 1.0 and a confidence level of 95 %, the number of randomly selected grout fans needs to be 17, as shown in Table 4-2.

In the following grouting method evaluation the number of randomly selected grout fans is chosen to 20, which is above 95 % confidences.

The average number of joints per fan will then, for 20 fans, be within an interval 4 to 6 number of joints per fan which has been regarded as sufficient for grouting prediction purposes.

To test the effect of the sampled population in reality being non-normal, a number of tests were made to prove that the mean converges for the chosen number of samples.

In a similar manner as here shown for the number of fans to be calculated, the number of single joints to be grouted with a given aperture can be calculated.

In the concluding calculation of sealing time for a characteristic region of a rock mass the number of fans has been increased to 25, which is equal to a tunnel with a length of 500 meters and above the confidence level. The only reason for this was to facilitate the separation of the numerical values.

4.2.3 Variance reduction

The rock mass properties are location dependent. However, within a region, the properties are similar and vary between certain limits.

For several applications within rock engineering, as for example within grouting and sealing of rock, the design is not governed by a point value. Instead it is a sum or the mean value, which is of interest. The inflow requirement for a tunnel is usually expressed as a mean inflow for a tunnel length of 100 meters. In practice, the requirement may also be set up as an allowed inflow for the whole tunnel, as for example the Hallandsås tunnel, which had an inflow requirement of 100 litre per second for the 2 times 8.6 km. of tunnel. The requirement may also be divided into regions as for the Baneheitstunnelen in Kristiansand, where the inflow requirement was set to 12 litres/min./ 100m, under the lakes and 6 litres/min./ 100m, for other regions. Based on the above and on the discussion in chapter 2, the requirement should be related to a specific rock mass type and for a defined water source. The requirement could then be expressed as a total inflow for the specific rock type or as a mean for each specific rock type. The grouting design can then be specified as a mean or sum for a region corresponding to the length of the specific rock mass condition.

In Figure 4-7 a tunnel through a rock mass is shown. The rock mass may, after investigation be described by one specific rock mass description, which comprises of a mean aperture and an aperture variance. Most likely, such a description will be of very little use for the grouting design, as an average grouting design will not perform well in any of the three different specific rock masses. Instead the rock mass should be described by three specific rock mass descriptions, as for example '(A) medium jointed with small apertures, crystalline rock mass with low water pressures', '(B) highly jointed with medium apertures, crystalline rock mass with medium water pressures' and '(C) complex deformation zone with high water pressures.

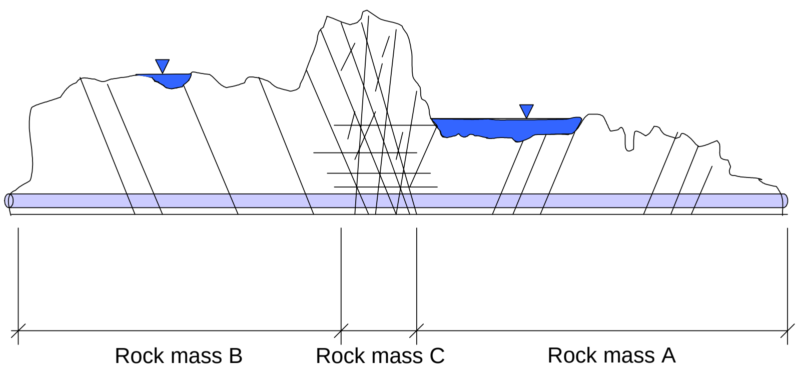


Figure 4-7 Rock mass with three different characteristic regions.

The three different rock mass descriptions with mean aperture and aperture variance or even better the aperture Probability Density Function (PDF), have a much better chance to be a base for a successful grouting design. An example of aperture probability density functions for the three rock mass regions is shown in Figure 4-8.

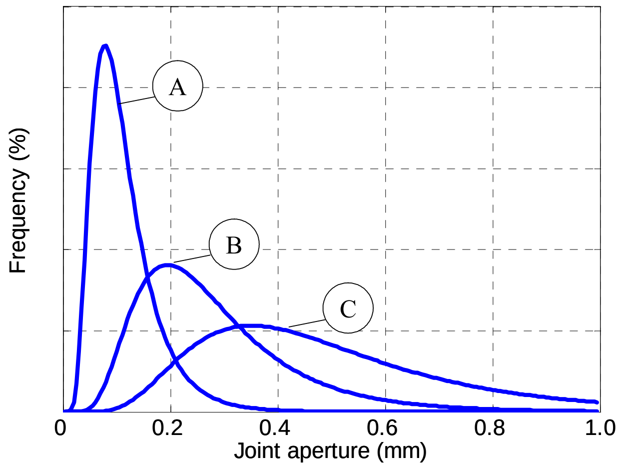


Figure 4-8 Example of aperture Probability Density Function (PDF) for rock mass region A, B and C.

The number of joints encountered a certain distance along a line, or in a bore hole, often follow a Poisson distribution, since jointing is an (almost) random process (Stille et al, 2003). Within each of the three probability density function shown in Figure 4-8, the occurrence of a specific joint aperture is therefore assumed as random.

When sealing a rock mass by grouting the grout is distributed through a number of holes, which need to intersect a conductive part of a joint to enable grout spread. The chance to intersect is illustrated in Figure 4-9 and depends on the joint direction, the openness of the joint plane, the hole diameter and the hole length. In chapter 3, the openness of a joint plane was compared as a "piece of cake" model by Hässler (1991). As jointing is a random process, openness is assumed to be a random property. With a fixed fan layout a random openness gives a random chance of intersecting a conductive part of a joint plane.

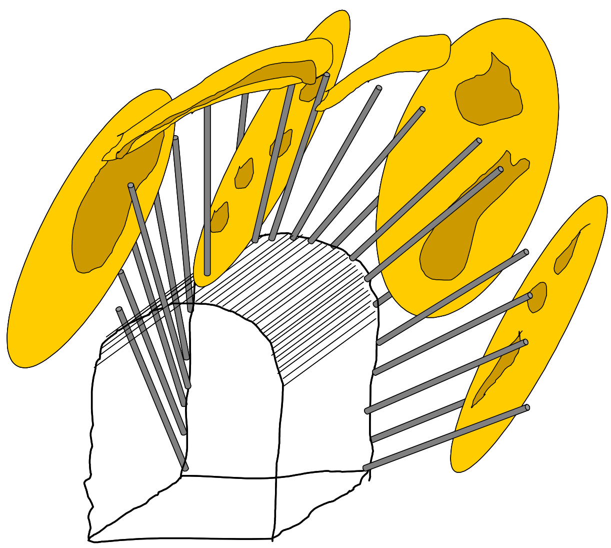


Figure 4-9 Illustration of a grout fan and the chance to intersect a conductive part of a joint plane.

Therefore based on the above discussion it is assumed, that grouting of a fan within a specific rock mass or region could be considered as a more or less random event.

The geotechnical characteristics tend to vary from one fan to another. The grouting effort for each fan depends on the geotechnical characteristics, expressed as vector X_i, with a given joint distribution, $f(X_i)$. The grouting effort is a general expression representing the grouting time, the inflow reduction and the sealing effect. The relationship between the characteristics and the grouting effort can be expressed as a stochastic function, $g(X_i)$. It is assumed that the characteristics for a fan are uniformly distributed but not necessarily uncorrelated. This results in a uniformly distributed grouting effort for each fan within a region, but not with the same distribution as the one for the geotechnical characteristics (Isaksson, 2002).

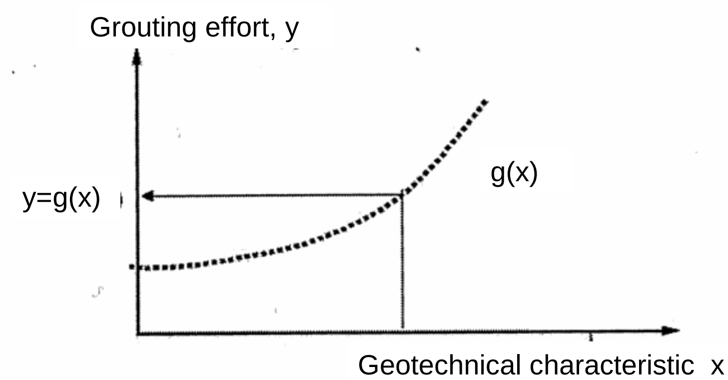


Figure 4-10 Illustration of the correlation between grouting effort (y) and the geotechnical characteristics, after Isaksson (2002).

The grouting effort is regarded as a stochastic variable, Y, and could, for a fan i, be noted as:

$$Y_i = g[X_i] \qquad \text{Equation 4-7}$$

with an expected value $E[Y]$ and a standard deviation σ_y.

The grouting effort within a region over a length L_R, for a number n of fans with length l can then be estimated from:

$$W = \sum_{i=1}^{n} Y_i = \sum_{i=1}^{n} g[X_i] \qquad \text{Equation 4-8}$$

where $n=L_R/l$. The mean grouting effort can then be expresses as:

$$E(W) = n \cdot E(Y) \approx n \cdot g[E(x)] \qquad \text{Equation 4-9}$$

and the standard deviation of the grouting effort can be expressed as:

$$\sigma_w = n \cdot \sigma_y \qquad (\textit{for correlated fans}) \qquad \text{Equation 4-10}$$

As the length of the region L_R (number of fans) is increased, the standard deviation of the total grouting effort tends to be reduced in the process of averaging. Methods used to estimate the reduced standard deviation have been described in literature by e.g. Ang & Tang (1975), Vanmarcke (1977), and Olsson (1986).

Based on Ang & Tang (1975), the standard deviation could, if the correlation is regarded, be noted as:

$$\sigma_w = \sigma_y \cdot \sqrt{n}\sqrt{1+(n-1)\cdot\rho} \qquad (\textit{for correlated and uncorrelated fans}) \qquad \text{Equation 4-11}$$

Where ρ is a normalized version of the covariance between the elements (X, Y) called the correlation coefficient and defined according to Råde et al (1988) as:

$$\rho = \frac{COV(X,Y)}{\sqrt{VAR[X] \quad VAR[Y]}} \qquad (-1 \leq \rho \leq 1) \qquad \text{Equation 4-12}$$

The above correlation coefficient can be used if the correlation is the same for all fans. For a rock mass this is often not the case, as a rock mass exhibits a spatial variation, where distance governs the degree of correlation. Vanmarcke (1977) suggested a method to calculate the variance of a sum (or a mean) of spatially correlated variables with a reduction factor Γ according to:

$$\sigma_{mean\ of\ L} = \Gamma \cdot \sigma_y \qquad \text{Equation 4-13}$$

Based on that an expression for the standard deviation which regards the correlation for the sum over L could be noted as:

$$\sigma_w = n \cdot \Gamma \cdot \sigma_y \qquad (\textit{for correlated and uncorrelated fans}) \qquad \text{Equation 4-14}$$

The distance, in which the geotechnical characteristic shows relatively strong auto correlation from a fan to another for the studied length L, is according to Vanmarcke (1977) defined as the "scale of fluctuation" δ_l. In Figure 4-11 the grouting effort y, the standard deviation for a fan σ_y and the "scale of fluctuation" δ_l, is shown.

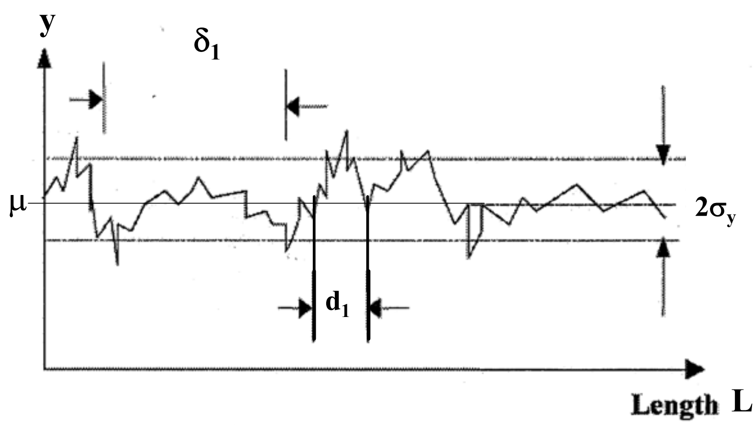


Figure 4-11 Illustration of the grouting effort (y), the standard deviation for a fan (σ_y), the "scale of fluctuation" (δ_l) and the distance between "mean crossings" (d_l) for the studied length (L).

In the following, the studied length is equal to the length of a region, L_R.

The reduction factor Γ depends on the relation between the studied length (number of fans) and the "scale of fluctuation" δ_l as shown in Figure 4-12.

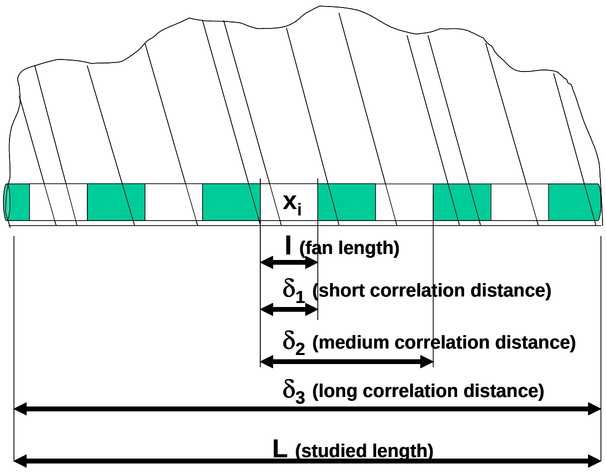


Figure 4-12 Illustration of correlation level dependency on scale of fluctuation δ_i, fan length l and studied length L for one type of rock mass consisting of n number of fans x_i.

If the studied length is less than the "scale of fluctuation", then the standard deviation is not reduced. However, when the studied length exceeds the scale of fluctuation, the reduction factor can be estimated as the ratio between the scale of fluctuation and the studied length and is expressed, based on Vanmarcke (1977), as:

$$\Gamma(L) = 1 \qquad for\ L \leq \delta_1$$ Equation 4-15

$$\Gamma(L) = \sqrt{\frac{\delta_1}{L}} \qquad for\ L \geq \delta_1$$ Equation 4-16

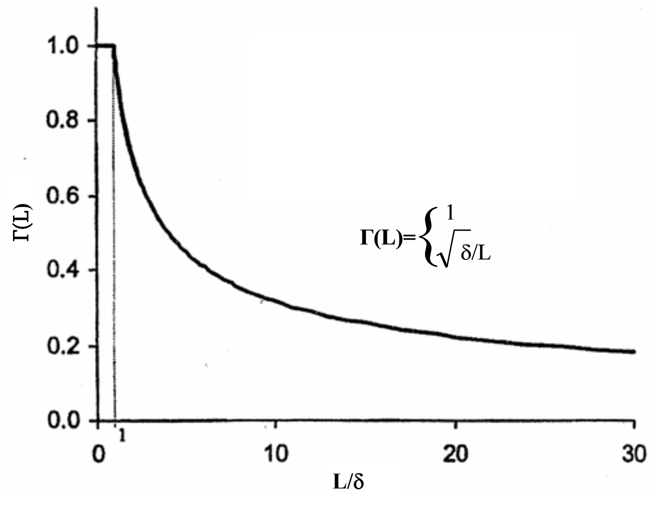


Figure 4-13 Simplified reduction factor Γ(L) as a function of the ratio between the studied length and the scale of fluctuation δ_l, proposed by Vanmarcke (1977).

The correlation distances are in Figure 4-12 illustrated as short, medium or long. For different lengths of correlation distance the reduction factor Γ can, based on Figure 4-13, be noted as shown in Table 4-3.

Table 4-3 The reduction factor Γ depends on the scale of fluctuation.

correlation distance, δ	Γ/δ	Γ
1	20.0	~0.24
3	6.7	~0.39
20	1.0	1

The determination of the scale of fluctuation was by Vanmarcke (1977) also estimated by using the distance between "mean crossings" (d_l) according to:

$$\delta_1 = \sqrt{\frac{2}{\pi}} d_1 \qquad \text{Equation 4-17}$$

Deutsch (2002) described another way to estimate the scale of fluctuation by analysing the spatial correlation. The spatial correlation describes the variation of properties from one point to another in space. The spatial correlation structure can be expressed in terms of the variogram which describes the dissimilarity between two points in space, separated by a distance *h*. The variogram of a property $Z(x)$ is expressed as (e.g. Deutsch, 2002):

$$2\gamma(h) = E(Z(x_i) - Z(x_i + h))^2 \qquad \text{Equation 4-18}$$

$2\ \gamma(h)$ is the variogram value at a separation distance h. $Z(x_i)$ is the value of the random variable at location x_i, and $Z(x_i+h)$ is the value of the random variable at distance h from $Z(x_i)$. The relation is valid for stationary random fields were the mean and variance are constant throughout the specific volume; the value of the variogram depends only on the separation distance h and not on the location x_i of the measured value $Z(x_i)$. If the studied rock mass section is large enough, the spatial correlation structure can be assumed isotropic. The semivariogram or the experimental variogram, based on experimental data is expressed as (e.g. Deutsch, 2002):

$$\gamma(h) = \frac{1}{2n(h)} \sum_{i=1}^{n(h)} \left[Z(x_i) - Z(x_i + h) \right]^2 \qquad \text{Equation 4-19}$$

Where n is the number of data. Spatial correlation structures are characterised by their model type. Commonly used models are Sill-, Exponential-, Gaussian-, Spherical- and Power-model.

Based on work by Vanmarcke (1984) the scale of fluctuation δ_l has been expressed as a function of the range a, for the exponential-, the Gaussian-, and the spherical semivariogram models.

Exponential semivariogram model $\quad \delta = \frac{2}{3} a$

Gaussian semivariogram model $\quad \delta \approx a$

Spherical semivariogram model $\quad \delta = \frac{3}{4} a$

For grouting of rock within a rock mass region, the scale of fluctuation has earlier been discussed and assumed to be equal to the length of a fan.

The randomized calculated fans studied in this thesis has also been analysed for correlation. As shown in Figure 4-14, there is no correlation between grouting of different fans.

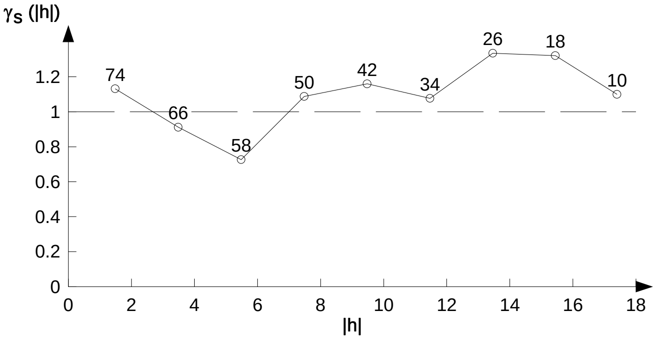


Figure 4-14 Standardized variogram for grouting times of MSA-1. As shown there is no correlation between the grouting times for different fans, which is due to randomization of the calculated fans.

The standard deviation σ_w for the sum of a number n of uncorrelated fans could then be calculated based on the standard deviation of the individual standard deviation σ_i according to:

$$\sigma_w = \sqrt{n} \cdot \sigma_i \qquad (\textit{sum of independent elements}) \qquad \text{Equation 4-20}$$

For some occasion a correlation between fans could for different reasons not further investigated here, be noticed. In *Figure 4-15* a variogram for a series of grout fans, which to some extent are correlated is shown. The variogram has also been modelled with a spherical variogram model. The range can be observed to be around 3 fans, which means that the nearest 3 fans, for this example, show a correlation.

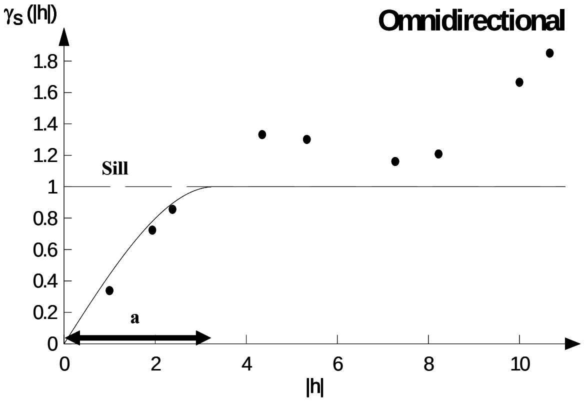


Figure 4-15 Standardized variogram for grouting time from the Arlanda rail link. The grouting fans are chosen from one rock mass section.

The standard deviation could then for a variogram with a range of 3 fans shown in *Figure 4-15* be reduced and calculated according to:

$$\sigma_w = \sqrt{\frac{9}{4}} \cdot \sqrt{n} \cdot \sigma_i \qquad (\textit{sum of partly correlated elements}) \qquad \text{Equation 4-21}$$

Where n is the number of fans. The reduction factor used within the randomized evaluation in this thesis is to some extent, as shown by the relation between Equation 4-20 and 4-21, higher than what would be the case for real grouting.

4.3 Modelling of sealing time

4.3.1 General

The time to seal a section of a tunnel to a required level is defined here as the sealing time, t_s.

$$Sealing\ time = \sum_{i=1}^{n} t_s(x_i) \qquad \text{Equation 4-22}$$

The sealing time depends on a number of activities defined by the grouting method, which have earlier been discussed in chapter 2. To evaluate the time for drilling-, waiting-, probing, establishment, grouting, re-grouting and post-grouting are difficult to varying degrees. The time of grouting can affect the achieved sealing effect, which consequently affects the re-grouting time and the need for post-grouting. The grouting time and the achieved sealing effect are therefore more focused upon than other activities within this work.

To seal a section could result in either success or failure. If the requirement is not fulfilled after first grouting a re-grouting is necessary. A probability can then be calculated for the need of more than one grouting round to achieve the requirement. The probability for more than one grouting has been calculated for five projects and is shown in Table 2-1.

The probability for more than one grouting can be described with Boolean variables [0,1] (Thoft-Christiansen & Baker, 1982). In this application, the random variable can exist in one of two conditions: (1) where the “more than one grouting event *k* occurs”, or (2) where the “more than one grouting event *k* does not occur”. The “more than one grouting event” occurs with probability *p*, and does not occur with probability *1-p* according to the following:

$$\delta_{ik} = \begin{cases} 1\ event\ k\ occurs\ with\ probability\ p \\ 0\ event\ k\ occurs\ with\ probability\ (1-p) \end{cases}$$

The sealing time equation presented below, regards a first grouting and if necessary a re-grouting. If the purpose is to calculate the total sealing time after *n* number of re-groutings, the equation could be complemented with the additional terms of *“δ_{ik} $*t_{rs}$ (z_i)”* and with a similar manner for post-grouting. In the following evaluation the sealing time is only calculated for grouting and if necessary complemented with re-grouting.

The sealing time, with fulfilled requirements, could then be expressed as:

$$t_s = t_d(x_i) + t_g(y_i) + t_w(z_i) + t_p(v_i) + t_e(w_i) + \delta_{ik} \cdot t_{rs}(q_i) + \delta_{ik} \cdot t_{post}(u_i) + \lambda \qquad \text{Equation 4-23}$$

$$t_{rs} = t_{rd}(x_i) + t_{rg}(y_i) + t_{rw}(z_i) + t_{rp}(v_i) \qquad \text{Equation 4-24}$$

Where:	t_s	sealing time
	t_d	drilling time
	t_g	grouting time
	t_w	waiting time
	t_p	probing time
	t_e	establish time
	t_{rs}	re-sealing time
	t_{post}	post-grouting time
	t_{rd}	re-drilling time
	t_{rg}	re-grouting time
	t_{rw}	re-waiting time
	t_{rp}	re-probing time
	p	probability to fail with first round grouting/ or probability to fail with grouting, which calls for post-grouting
	δ_{ik}	$\begin{cases} 1 \text{ event } k \text{ occurs with probability } p \\ 0 \text{ event } k \text{ occurs with probability } (1-p) \end{cases}$
	λ	unknown parameter

<u>Drilling time, t_d</u>

The time for drilling t_d depends of the number on grouting holes, the hole length, the hole diameter and the rock mass conditions, as discussed in chapter 3. In addition holes may need to be cleaned from debris and dust before grouting and sometimes measured for divergence as well. Drilling in a rock mass can then be noted as a normal time and are to great extent controlled by the capacity of drilling. However, drilling in a rock mass is also affected by a random process of errors. The errors may originate from a jammed drill bit or broken equipment etc. This error time can then together with the normal time result in a non-normally distributed or multi-model distribution time for drilling. Within this work the randomness of the error time is not especially evaluated and the drilling time is assumed to be normal distributed. The drill time could then be noted as:

$$t_d = n \cdot (k_1 + k_2 + k_3) + k_4 \cdot (n-1) \qquad \text{Equation 4-25}$$

Where: n = number of holes
k_1= drill time per hole = hole length/drilling capacity
k_2= cleaning time per hole
k_3= divergence measurement per hole
k_4= time between holes

The drilling capacity depends on the drill rig, which means that the capacity depends on whether a single- double- or triple-boom drill rig is used, which allows simultaneously drilling of more than one hole. Based on experience, the drilling capacity of Φ 64 mm grout holes with a length of 20 meters is assumed to be between 60 and 80 meters per hour, with an average around 70 meters per hour using a twin-boom hydraulic drill rig (Manell, 2004) &

(Lundqvist, 2004). For calculation within this work, the time for establishment is regarded separately, wherefore the drilling capacity is chosen in the upper range of the distribution with a mean of 80 meters per hour, as shown in Figure 4-16.

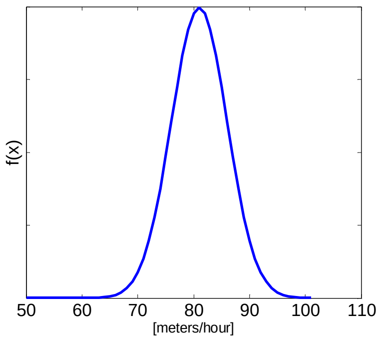


Figure 4-16 Assumed capacity for drilling of grout holes (Φ 64 mm) using a twin-boom hydraulic drill rig, based on experience.

<u>Grouting time, t_g</u>

The grouting time t_g depends on the activities specified in the grouting method. For each grouting method the grouting time is dependent on, for example, the rock mass properties, the grouting fan geometry, the grout mix properties, the grout volume, the grouting pressure and the grouting performance. The grouting time distribution for randomized calculated grouting fans as well as for real grouting fans has shown to be normally or log-normally distributed, see Figure 4-17. The following calculation of grouting time distribution is therefore assumed to be log-normally distributed.

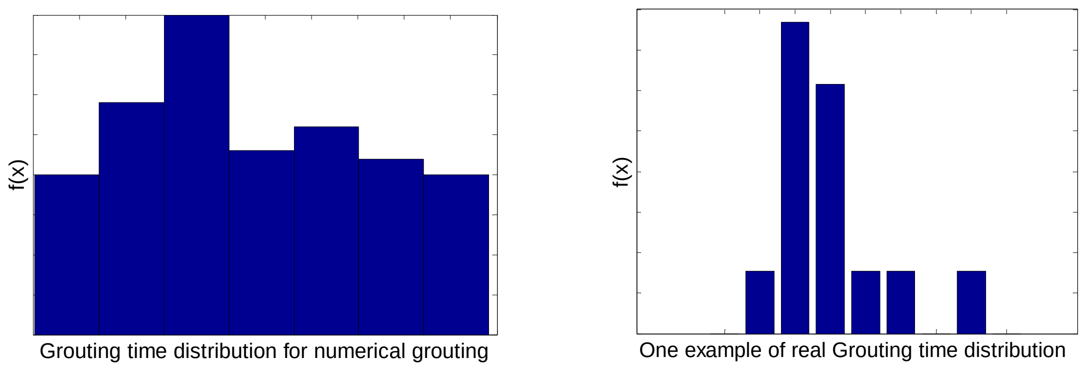


Figure 4-17 Examples of grouting time distributions. The left histogram is based on numerical grouting of log-normally distributed joint planes. The right histogram is based on a series of 13 grouting fans in similar rock mass properties from Arlandabanan.

<u>Waiting time, t_w</u>

The time of waiting t_w depends mainly on the setting time of the grout and the number of groutings. For different situations, different requirements on grout stability could be applicable. The resistance against flushing of grout by cooling water from drilling in joints has been modelled by Johansson (1997). The recommendation was then given that the grout should reach a shear resistance above 12 kPa to be able to resist the flushing water from the drilling process, measured at a drill water pressure of 2.0 MPa. In the following calculations the time of waiting is set equal to the time of reaching 12 kPa shear resistance of the grout *(x)* expressed in hours, times the number of groutings. The waiting time is then assumed to be a step function, with a fixed waiting time and no variation.

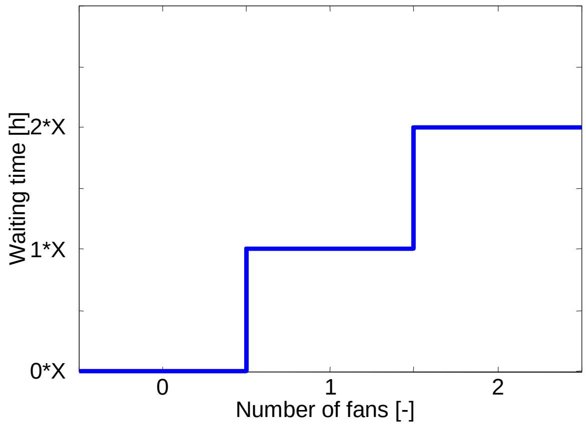


Figure 4-18 Waiting time for hardening of the grout as a function of the number of groutings [-] and the time [h] to reach 12 kPa shear resistance for the specific grout mix.

Setting time for the different grout mixes are noted in the appendix.

<u>Probing time, t_p</u>

The time for probing and other measurements to receive information about the rock mass to be grouted varies between projects. For some situations, with steady conditions, probing may be unnecessary. For other situations with rapidly changing rock mass conditions and high risks, probing may be very important and time consuming. The probing time for each specific location depends on a combination of “the time for knowing rock mass conditions” and “the risk for changing conditions” as noted:

t_p = f (time for knowing rock mass conditions, risk for changing conditions)

Probing can both be carried out in grout holes, which eliminates the specific drilling time for probing, or in probing holes, which has no other purpose than probing. Within this project probing for the different methods is equal to water loss measurements prior to grouting to decide the grout mix and after grouting to decide if re-grouting is necessary or not. The time for probing could then be noted as:

$$t_p = n_p \cdot t_d(x_i) + (n_p + n_{pg}) \cdot t_{wloss}(y_i) \qquad \text{Equation 4-26}$$

Where n_p is the number of probe holes, which are used to decide about re-grouting and n_{pg} is the number of grout holes, which are tested for water loss and t_{wloss} the time for each water loss measurement. Drilling of probe holes could be performed with a smaller diameter than for grout holes and thereby the capacity could increase to around 150 meters per hour for a twin-boom hydraulic drill rig. Commonly, for practical reasons, the same equipment is used for drilling of probe holes as for drilling of grout holes since probe holes have almost the same length as grouting holes. The capacity for drilling of probe holes is therefore assumed to be equals to the drilling capacity of grouting holes, as shown in Figure 4-16. The time for each water loss measurement is not further analysed here. Instead the time is based on experience and assumed to be normal distributed with an average time of 8 minutes and a standard deviation of 1.5 minutes for each water loss measurement as shown in Figure 4-19 (Stuge, 2004), (Manell, 2004). The assumed time represents the average time for a fan with many water loss measurements (~30/fan). If the number of water loss measurements is less, then the average time needs to be increased.

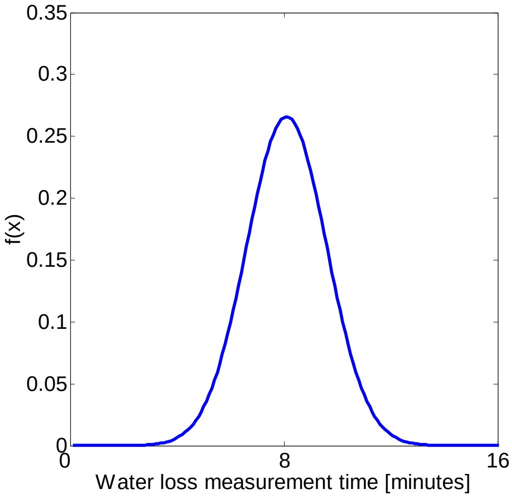


Figure 4-19   Time to carry out a water loss measurement in a 15 meter hole.

<u>Establishment time, t_e</u>

Within this thesis, the time for to preparation etc. at site, is called establishment time. Based on experience, the time to establish each drilling, grouting or water loss measurement operation has been assumed to be in the range of 33 minutes, as shown in Figure 4-20. The approximated normal distribution, which will be used to calculate the total sealing time is also shown, with a mean of 33 minutes and a standard deviation of 12 minutes.

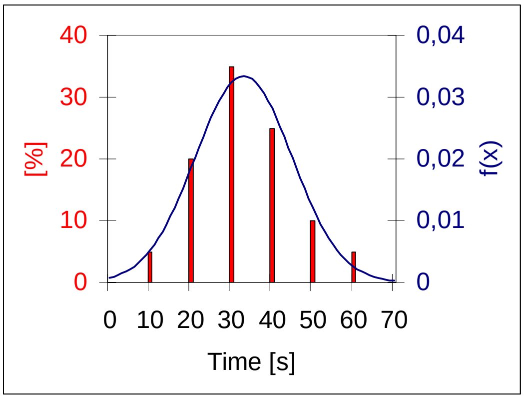


Figure 4-20 Time for establishment for each operation.

<u>Re-sealing time, t_{rs}</u>

The time for re-sealing t_{rs} is calculated in a similar manner as the time for sealing, but the occurrence on re-sealing depends of the probability p to not fulfil the requirement with the prior grouting.

The probability to not fulfil the requirement with the prior grouting can be low if the grouting effort is extensive in relation to the requirement or high if the grouting effort is moderate in relation to the requirement. The right level should be chosen based on various information as cost of grouting, cost of waiting, internal- and external influence of failing with grouting, negative publicity of failing with grouting etc. From place to place, the probability to fail with the first round of grouting, needs to be evaluated. In the following calculation of re-grouting time, the re-grouting time distribution is assumed to be normal distributed.

Postgrouting time t_{post}

Post grouting is commonly much more time consuming than pre-grouting to reach the same sealing effect due to, for example, the use of lower grouting pressures. In general, time for post-grouting is not a critical factor during an underground project, but if the post-grouting turns out to be extensive; it could be a critical factor. A grouting method with high sealing effect can fulfil the requirement and result in no post-grouting, while a grouting method with low sealing effect can result in extensive post-grouting. When comparing the sealing time for different grouting methods it is important to decide weather or not and to what extent post-grouting will be a critical factor. Post-grouting can therefore be regarded as a decision problem, see Figure 4-21. Within this project post-grouting is therefore left out of the sealing time calculations and instead regarded in the discussion section in chapter 7.2.1.

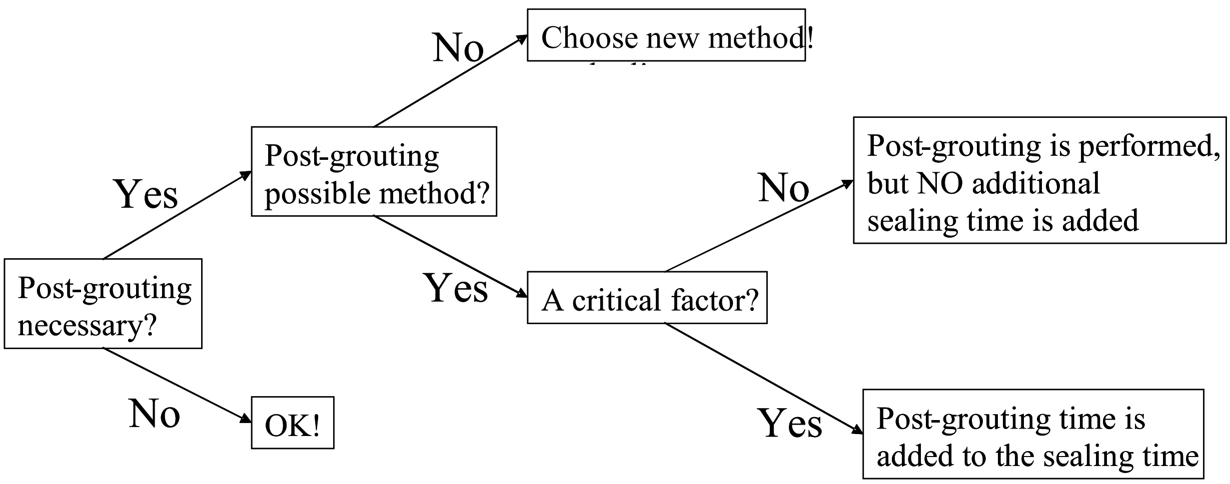


Figure 4-21 Illustration of post-grouting as a decision problem.

4.3.2 Calculating the sealing time

The sealing time for the different grouting methods, are calculated by adding the time for the different actions described by each grouting method.

The sealing time is calculated as the sum of the stochastic variables with different distributions for a specified tunnel length. The tunnel length should represent a studied length with equal rock mass properties. For the following calculations the length has been set to 500 meters, which for a fan length of 20 meters is equivalent to 25 fans. For a real grouting situation the number of fans should be larger, due to the fact that an overlap of the grouting fans is used to secure a continuous sealing. Within the numerical calculations the overlap is set to zero to avoid calculating the inflow from a specific joint twice.

For some of the calculated methods re-grouting is requested if water loss measurements in control holes is above the control hole requirement. Each re-grouting time is dependent on the prior grouting fan. Therefore a total grouting time distribution is calculated for the combination of the grouting time and the, if needed, additional re-grouting time.

The different distribution representing the grouting time, the drilling time, the waiting time, the probing time and the establishment time may to some degree be correlated, but within this project they are assumed independent. The distributions are further assumed to be within the same range with no dominating distribution. This could also be expressed based on

(Weisstein, 2004), as: if $X_1, X_2, \ldots X_N$ is a set of N independent random variables representing the different distributions (drilling time, grouting time, waiting time, probing time, establishment time) and each X_i have an arbitrary probability distribution $P(x_1, \quad , x_N)$ with mean μ_i and a finite variance σ_i^2. Then the normal form variable X_{norm} has a limiting cumulative distribution function which approaches a normal distribution.

The "fuzzy" central limit theorem, as described shortly above, says that data which are influenced by many small and unrelated random effects are approximately normally distributed (Weisstein, 2004).

The calculation of the sealing time t_s can by applying the central limit theorem be performed for the different distributions and will thus result in a normal distributed sealing time.

Based on the equation 4-23 presented in chapter 4.2.31 the expected sealing time for n fans can be calculated as the sum of the expected values of each element for a fan $E(t_i(x_j))$ times the number of fans n according to:

$$E(t_s) = n \cdot \left(E[t_d(x_i)] + E\lfloor t_{g+rg}(y_i) \rfloor + E[t_w(z_i)] + E\lfloor t_p(v_i) \rfloor + E[t_e(w_i)] \right) \quad \text{Equation 4-27}$$

The standard deviation σ can, based on the theory of variance reduction discussed in chapter 4.2.3, be calculated as the square root of the sum of the variances times the square root of the number of fans according to:

$$\sigma_s = \sqrt{n} \cdot \sqrt{\sigma_d^{\,2} + \sigma_{g+rg}^{\,2} + \sigma_w^{\,2} + \sigma_p^{\,2} + \sigma_e^{\,2}} \quad \text{Equation 4-28}$$

Each grouting method will then result in an expected sealing time and a corresponding standard deviation. To be able to compare the methods even the achieved sealing level noted as total inflow for the studied length must be calculated.

4.4 Suggested hypothetical rock masses

Three different rock masses denoted A, B and C are described below.

Rock mass A is a medium permeable rock with a low in situ water head (~20m). The open joints in A have an aperture between 0.0 and 0.5 mm and appear on average every fourth meter (5 open joints/20m.), shown in Figure 4-22. The joint planes have a small amount of contact, 10 %, and the aperture variability of a joint plane is found to be in the range of half the arithmetic mean aperture. Each joint plane is rather large in extension, around 30 meter, mainly vertical and striking an angle of 90 degrees in relation to the tunnel. The calculated hydraulic conductivity of the joint distribution shown below is in the range of 3.83 $*10^{-7}$ and the theoretical porosity is 0.031 ‰.

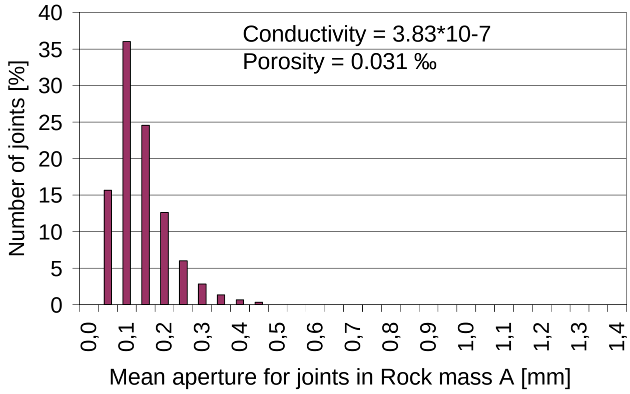


Figure 4-22 Joint aperture distribution for rock mass A.

Rock mass B is a highly permeable rock mass with a medium in situ water head (~100 m). The open joints in B have an aperture between 0.0 and 1.0 and appear on average every meter (20 open joints/20m.), shown in Figure 4-23. The joint planes have a contact of 30 %, which is larger than for rock mass A. The aperture variability of a joint plane is found to be in the range of half the arithmetic mean aperture. Each joint plane is rather large in extension, around 30 meters, mainly vertical and striking an angle of 90 degrees in relation to the tunnel. The calculated hydraulic conductivity for the joint distribution below is in the range of 1.83 $*10^{-5}$ and the theoretical porosity is 0.225 ‰.

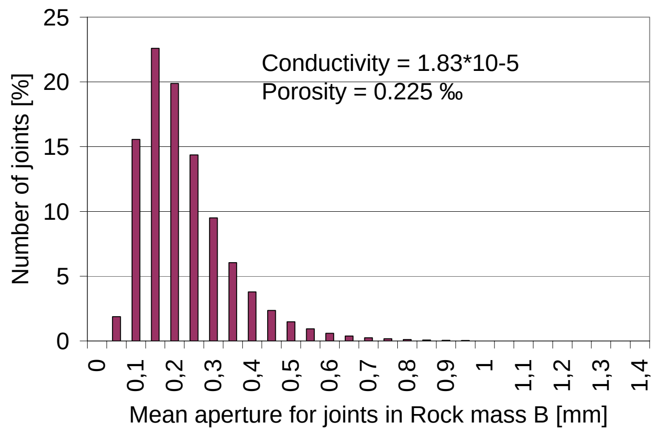


Figure 4-23 Joint aperture distribution for rock mass B.

Rock mass C is an extremely high permeable rock mass with a high in situ water head (~150 m). Rock mass C is most likely found for short sections of a tunnel. The open joints in C have an aperture between 0.0 and 1.4 mm and appear average on every 0.4 meters (50 open joints/20m.), shown in Figure 4-24. The joint planes have a small amount of contact, 10 %, and the aperture variability of a joint plane is found to be in the range of half the arithmetic mean aperture. Each joint plane is rather large in extension, around 30 meter, mainly vertical and striking an angle of 90 degrees in relation to the tunnel. The calculated hydraulic conductivity for the joint distribution below is in the range of 1.52 $*10^{-4}$ and the theoretical porosity is 0.844 ‰.

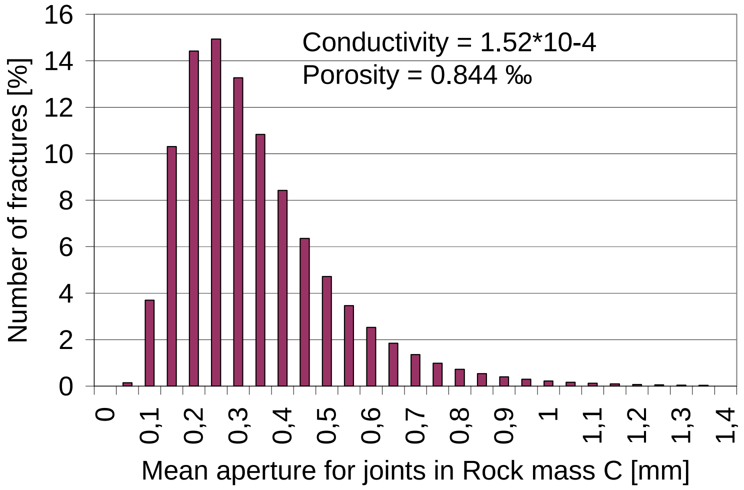


Figure 4-24 Joint aperture distribution for rock mass C.

Table 4-4 Joint distribution- and joint plane properties for rock mass A, B and C.

		A	B	C
The distribution of joints				
Hydraulic conductivity	[m/s]	3.83 E-7	1.83 E-5	1.52 E-4
Theoretical porosity	[$^0/_{00}$]	0.031	0.225	0.844
Mean aperture	[mm]	0.125	0.200	0.300
Standard deviation	[mm]	0.5	0.5	0.5
Aperture multiple	[mm]	0.05	0.05	0.05
Amount of open joints	[%]	25	25	25
Total number of joints/20 m.	[-]	20	80	200
Aperture distribution	[mm]	0.00 - 0.50	0.00 - 1.00	0.00 - 1.40
The individual joint planes				
Dimension	[m]	30 X 30	30 X 30	30 X 30
Aperture standard deviation	[mm]	0.5	0.5	0.5
Open part of each plane	[%]	10	30	10

4.5 Description of grouting methods

The three different grouting methods, which will be calculated and numerically grouted are described here. The methods have similarities with methods known from practice, but they also differ as they are adjusted to be numerically grouted. In the following, grouting time and grouting volume are calculated except for the filling of the grout hole.

In the numerical model, each joint is grouted individually, which means that criteria defined below are divided by the average number of joints crossing the grout hole and that the grouting time is taken as the highest value among the individual joints for each fan.

4.5.1 Description of grouting method MS

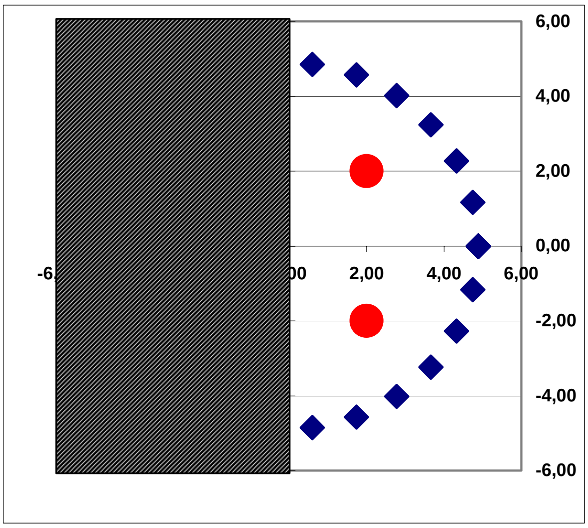


Figure 4-25 Grout fan layout for grouting method MS.

1) 26 grout holes (cc 1.18 m.) with a length of 20 meter and a radius of 32 mm are drilled as shown in Figure 4-25.

2) All grout holes are measured for water loss.

3) Grouting is started in the hole with the highest water loss value and so on.
Simultaneously grouting of different holes is not performed.
Stop pressure is set to 2.5 MPa above ground water pressure.
Grouting is completed when the stop pressure is reached and the grout flow is below 2 litres/minute during 5 minutes.

   Holes with LU < 1, are grouted with conventional grout cement and start with a W/C ratio of 2 and then thickened if no resistance is obtained according to the following limits:
   For a grout with W/C ratio 2.0 maximum 100 kg cement before thickening.For a grout with W/C ratio 1.0 maximum 200 kg cement before thickening.
   For a grout with W/C ratio 0.8 maximum 400 kg cement before thickening.
   For a grout with W/C ratio 0.5 grouting is continued until a stop is reached.

Holes with LU ≥ 1, are grouted with conventional grout cement and start with a W/C ratio of 1 and then thickened if no resistance is obtained according to the following limits:
For a grout with W/C ratio 1.0 maximum 200 kg cement before thickening.
For a grout with W/C ratio 0.8 maximum 400 kg cement before thickening.
For a grout with W/C ratio 0.5 grouting is continued until a stop is reached.

4) Four control holes are drilled according to Figure 4-25. The water loss is measured at a pressure of 0.5 MPa above ground water pressure. If the water loss in any hole is above 0.15 LU a second grout fan is drilled and grouted according to the description below, if not the grouting is completed.

5) The second grout fan is in general drilled and grouted as the first fan, but with some changes as described here. The grout holes are drilled in between the first grout holes.

   Holes with LU below < 0.3, are grouted with micro cement (d_{max95}=16 μm) and start with a W/C ratio of 2 and then thickened if no resistance is obtained according to the following limits:
   For a grout with W/C ratio 2.0 maximum 100 kg cement before thickening.
   For a grout with W/C ratio 1.0 maximum 200 kg cement before thickening.
   For a grout with W/C ratio 0.8 maximum 400 kg cement before thickening.
   For a grout with W/C ratio 0.5 grouting is continued until a stop is reached.

   Holes with LU between 0.3 and 1.0, are grouted with conventional grout cement (d_{max95}=30 μm) and start with a W/C of 2 and then thickened if no resistance is obtained according to the following limits:
   For a grout with W/C ratio 2.0 maximum 100 kg cement before thickening.
   For a grout with W/C ratio 1.0 maximum 200 kg cement before thickening.
   For a grout with W/C ratio 0.8 maximum 400 kg cement before thickening.
   For a grout with W/C ratio 0.5 grouting is continued until a stop is reached.

   Holes with LU ≥ 1, are grouted with conventional grout cement (d_{max95}=30 μm) and start with a W/C of 1 and then thickened if no resistance is obtained according to the following limits:
   For a grout with W/C ratio 1.0 maximum 200 kg cement before thickening.
   For a grout with W/C ratio 0.8 maximum 400 kg cement before thickening.
   For a grout with W/C ratio 0.5 grouting is continued until a stop is reached.

6) Four control holes are drilled according to Figure 4-25 with a length of 15 meters. The water loss is measured at a pressure of 0.5 MPa above ground water pressure. If the water loss in any hole is above 0.15 LU, a third grout fan is drilled and grouted according to the description in number 7, if not the grouting is completed.

7) The number of grouting rounds is maximised to one re-grouting.

4.5.2 Description of grouting method MS-1

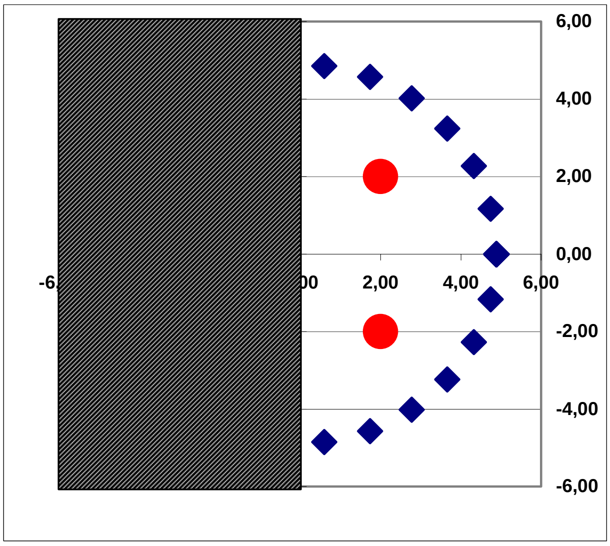


Figure 4-26 Grout fan layout for grouting method MS.

Method MS-1 is equivalent to method MS, except that the choice is based on Lugeon values on hole level instead of on joint level as for MS.

The result will be that grouting with method MS-1 is more often started with a micro cement and a W/C ratio of 2, and that grouting with method MS is more often started with a conventional grout cement and a W/C ration of 1.

See further the description of MS above.

4.5.3 Description of grouting method MA

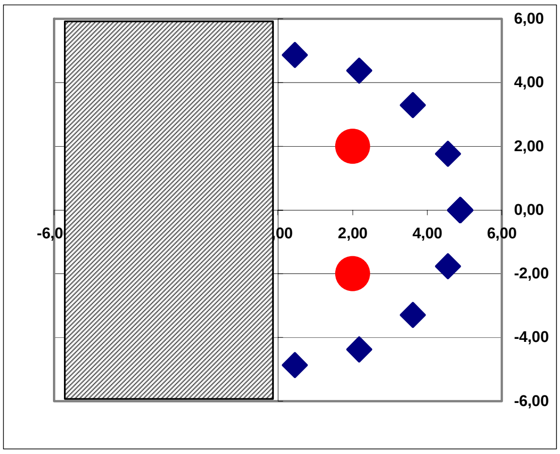


Figure 4-27 Grout fan layout for grouting method MA.

1) 17 grout holes (cc 1.8 m.) with a length of 20 meter and a radius of 32 mm are drilled as shown in Figure 4-27.

2) Grouting is started in the bottom and then continued upwards. Simultaneous grouting of different holes is not performed. Stop pressure is set to 1.5 M Pa above ground water pressure.
Grouting is completed when the stop pressure is reached and the grout flow is below 0.2 l/min. In the numerical model, each joint is grouted individually, which means that the above defined completion criteria is divided by the average number of joints crossing the grout hole and that the grout time is taken as the highest value among the individual joints.

   Grouting is started with micro cement (d_{max95}=16 μm), a W/C ratio of 0.8.
   Grouting is continued with the start grout until 713 kg cement has been grouted, thereafter the grout is changed to a micro cement (d_{max95}=16 μm), W/C ratio 0.6 for another 713 kg of cement.

Chapter Five

5 Results from evaluation of grouting methods

In this chapter the results from the numerical calculations of the different grouting methods are reported. It is essential to notice that grouting refers to the first performed grouting and re-grouting to the second performed grouting at the same location. Dots and lines in figures have in the digital version been colorized. Green colour is used to present results before grouting, blue is after grouting, red is after re-grouting and black refers to control holes. For comparison between methods the inflow criteria has been set to 5 litres/min/100 meters and re-grouting based on Lugeon values in control holes to 0.15 Lugeon. To enable extended comparison between methods, control holes have, in this chapter, been evaluated even for grouting methods without prescribed control holes. The different methods and the different rock masses used in this chapter for evaluation have been described in detail in chapter 4.

The numerical calculations have been performed for nine combinations of method and rock mass according to Table 5-1.

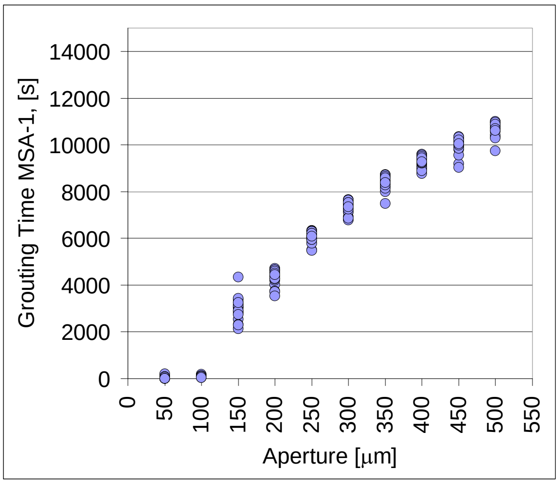
Table 5-1 The nine different combinations of method and rock mass.

Calculation	**Method Description**		**Rock mass Description**	
1	MS-1	Grout W/C 2.0-0.5, high amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	A	Few joints, aperture=0.125mm, C=3.8*10^{-7} m/s
2	MS	Grout W/C 2.0-0.5, Medium amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	A	Few joints, aperture=0.125mm, C=3.8*10
3	MA	Grout W/C 0.8-0.6, Hole cc 1.8 m., ΔP=1.5 MPa	A	Few joints, aperture=0.125mm, C=3.8*10
4	MS-1	Grout W/C 2.0-0.5, high amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	B	Medium joint number, aperture=0.200mm, C=1.8*10^{-5} m/s
5	MS	Grout W/C 2.0-0.5, Medium amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	B	Medium joint number, aperture=0.200mm, C=1.8*10^{-5} m/s
6	MA	Grout W/C 0.8-0.6, Hole cc 1.8 m., ΔP=1.5 MPa	B	Medium joint number, aperture=0.200mm, C=1.8*10^{-5} m/s
7	MS-1	Grout W/C 2.0-0.5, high amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	C	Large joint number, average aperture=0.300mm, C=1.5*10^{-4} m/s
8	MS	Grout W/C 2.0-0.5, Medium amounts of low W/C, Hole cc 1.18 m., ΔP=2.5 MPa	C	Large joint number, average aperture=0.300mm, C=1.5*10^{-4} m/s
9	MA	Grout W/C 0.8-0.6, Hole cc 1.8 m., ΔP=1.5 MPa	C	Large joint number, average aperture=0.300mm, C=1.5*10^{-4} m/s

5.1 Result Method MS-1, Rock mass A (MSA-1)

5.1.1 Comparison on joint level

The grouting method, MS-1, has been applied to rock mass A. The basis for the evaluation is the calculation of each possible joint in the interval 0 – 500 µm, which gives 200 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-1. As shown, both the time and the volume are dependent on the joint aperture.



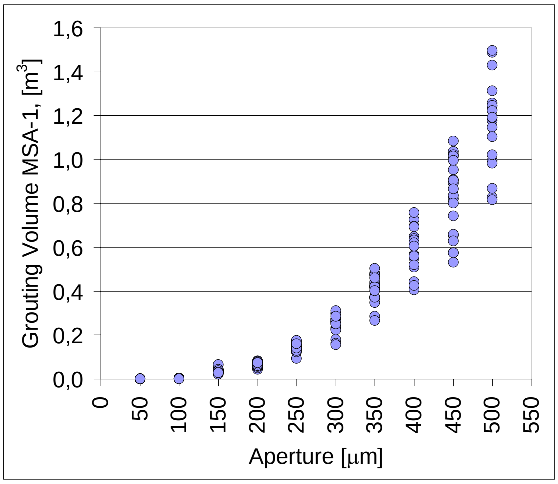
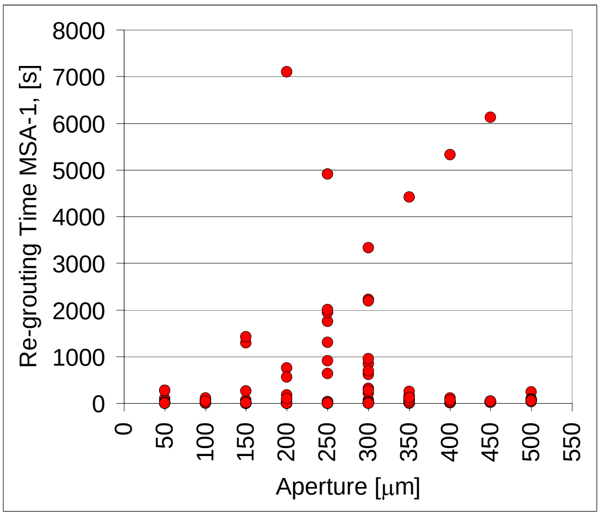


Figure 5-1 Grouting time and grouting volume for different apertures MSA-1.

All the joints have also been re-grouted, as shown in Figure 5-2. For all joints and apertures the re-grouting time is significantly shorter than the grouting. The re-grouting time is not increased by a larger aperture, which could be because that the initial grouting already sealed most of the wider joints to a high level. The variance, within an aperture multiple, is higher for the re-grouting than for the grouting, which could be because the joints are not fully reach by the hole spacing chosen for the grouting.



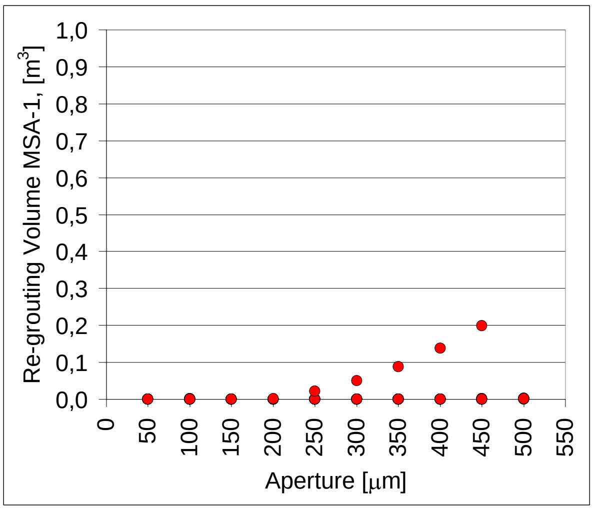


Figure 5-2 Re-grouting time and re-grouting volume for different apertures MSA-1.

If instead the re-grouting is plotted on a logarithmic scale, the difference between small values could be more clearly noted, as shown in Figure 5-3.

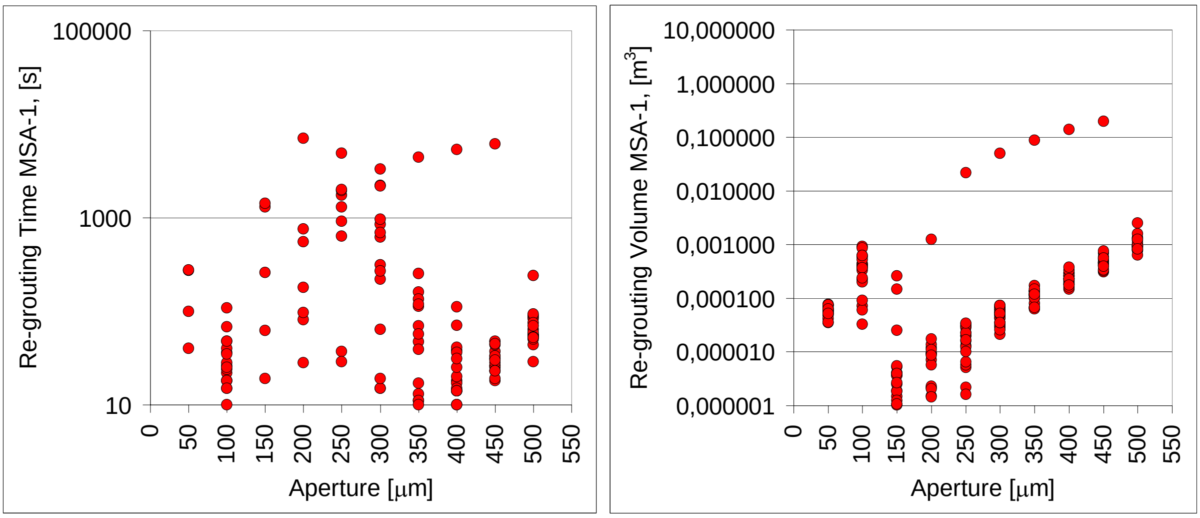


Figure 5-3 Re-grouting time and re-grouting volume for different apertures MSA-1 on a logarithmic scale (values equal to zero can not be plotted).

The sealing effect, as shown in Figure 5-4, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is high for apertures larger than 150 µm, but low for smaller apertures. For joints with an aperture of 50 µm the sealing time, as well as, the sealing effect was almost zero. The sealing effect from re-grouting was mostly improved for joints with an aperture in the range of 100 µm.

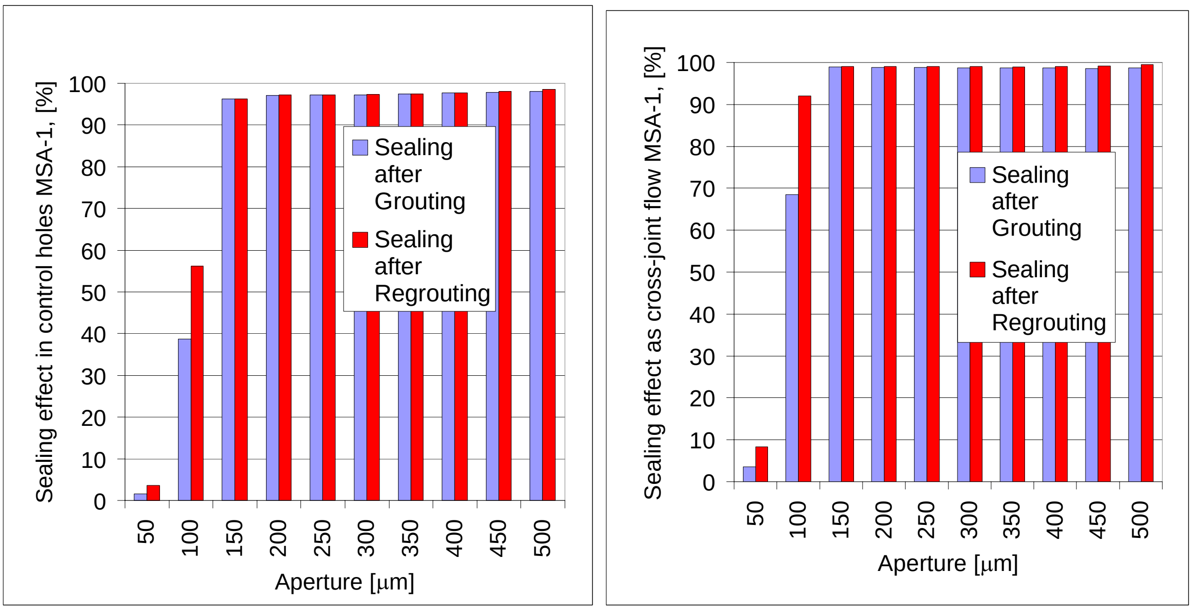


Figure 5-4 Sealing effect for different apertures MSA-1, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.1.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

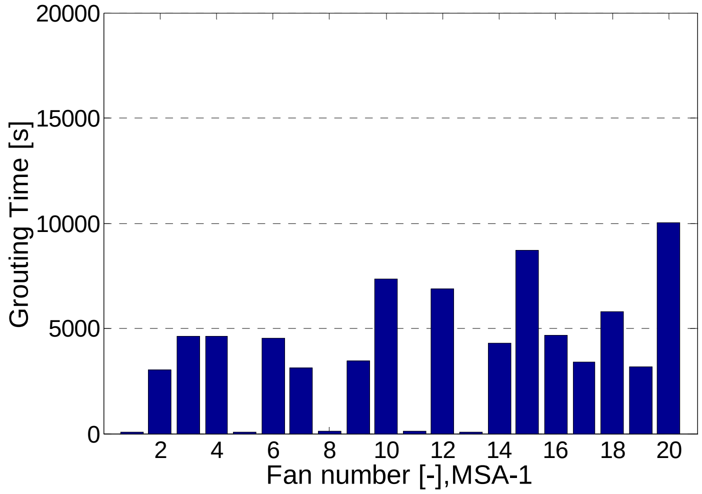


Figure 5-5 Grouting time for the individual grout fans MSA-1.

In Figure 5-5 the grouting time for the individual fans are shown. The average grouting time was 3901 seconds, with a standard deviation of 2922 seconds. For this method 25 % of the fans had a grouting time close to zero. The explanation is that those fans only consist of joints with small apertures, almost unreachable for this method.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-6 the inflow for method MS-1 in rock mass A is shown. From the figure the large difference before and after grouting could be noticed. Between first and second grouting the flow was further reduced, but of much less magnitude.

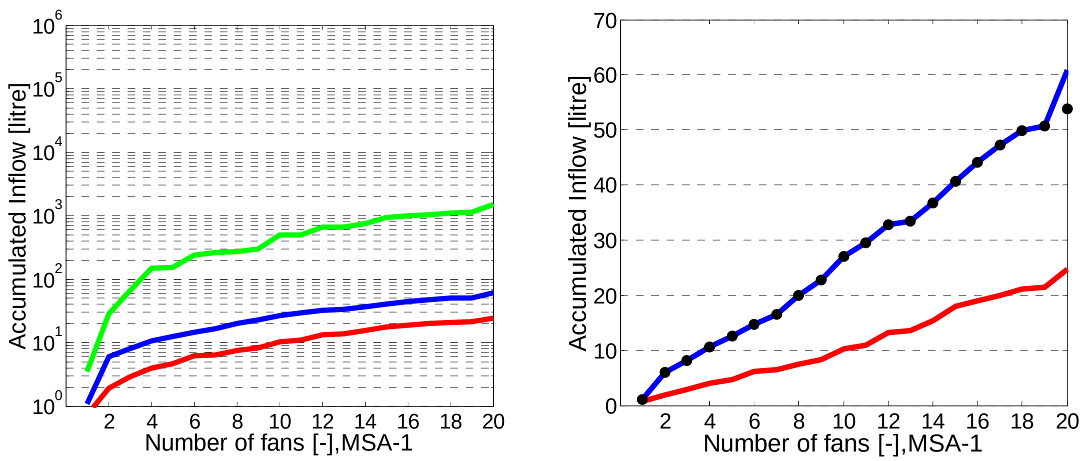


Figure 5-6 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

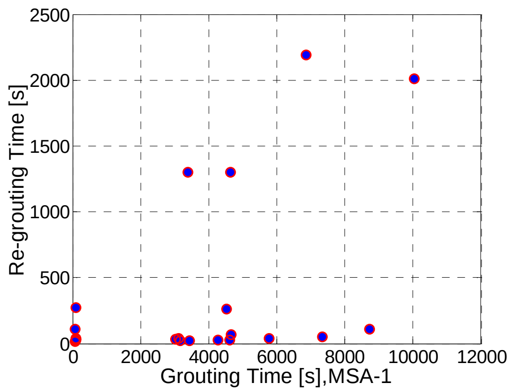


Figure 5-7 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-7 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and the re-grouting time for MSA-1.

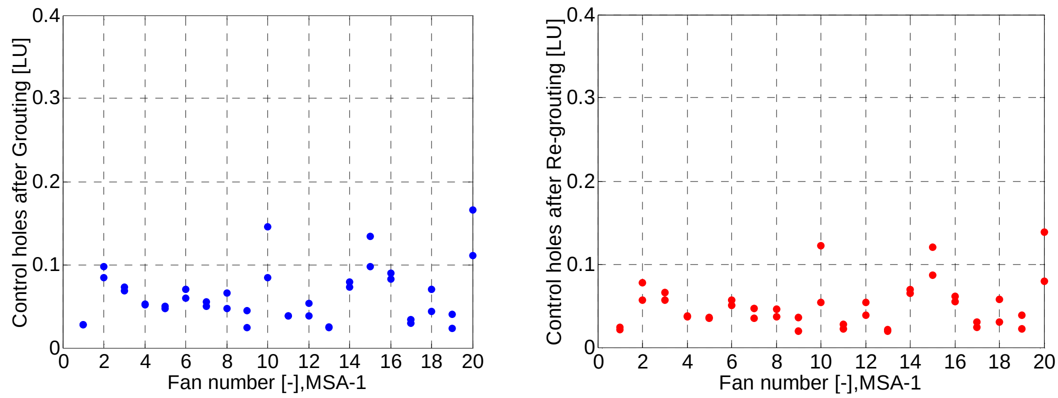


Figure 5-8 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 1 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-8 two control holes for each fan half after grouting and after re-grouting are shown. One of the control holes after grouting had a Lugeon value equal to or above the set criteria of 0.15 Lugeon. The control holes shown in Figure 5-8 resulted in 1 re-grouting (fan nr 20). The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. The added re-grouting time is 2009 seconds and the re-grouting volume 0.0235 m^3.

On the other hand, if re-grouting is carried out and tested, none of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently not be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-9 the probability to be below the inflow requirement is shown. The probability to not fulfil the inflow requirement is calculated as the number of fans, which has an inflow above each inflow requirement. If the inflow requirement after grouting is below 3 litres/min/100 meters for method MSA-1, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 51 litres/min/100 meters, the chance to not fulfil the requirement is 0 %. The inflow convergence is a prediction of the final inflow after 20 fans.

The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-9 the inflow convergence stabilizes after about 10 fans for grouting of MSA-1 and after about 2 fans for the re-grouting. The result shown below is highly dependent of one single joint of 0.45 mm, which affects the result in fan number 20.

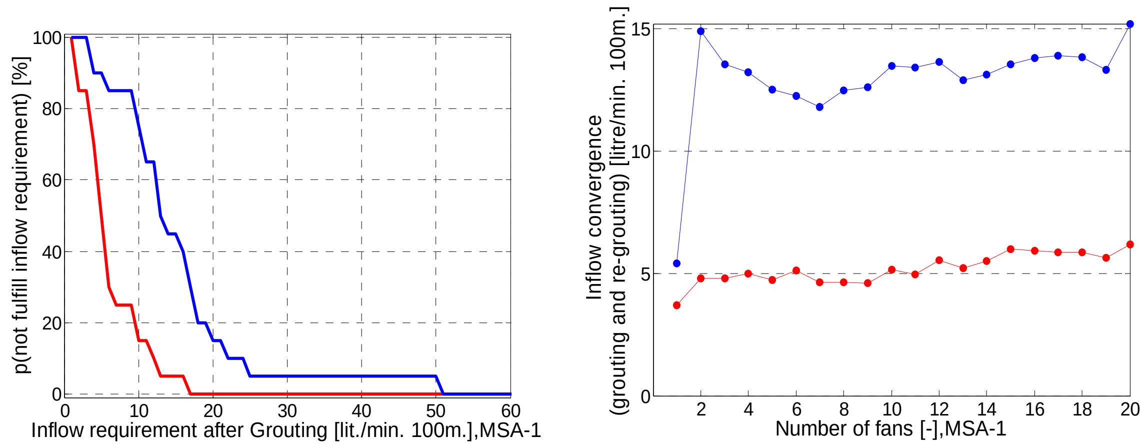


Figure 5-9 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSA-1 (left). Inflow convergence after n number of fans for MSA-1 (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-10. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level grouting reaches a high sealing effect and re-grouting a much lower. On the fan level the average value of all joints belonging to a fan are shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. Still the fan sealing effect after grouting for MSA-1 is quite low, which is because of a low joint intensity and a high amount of narrow joints for rock mass A.

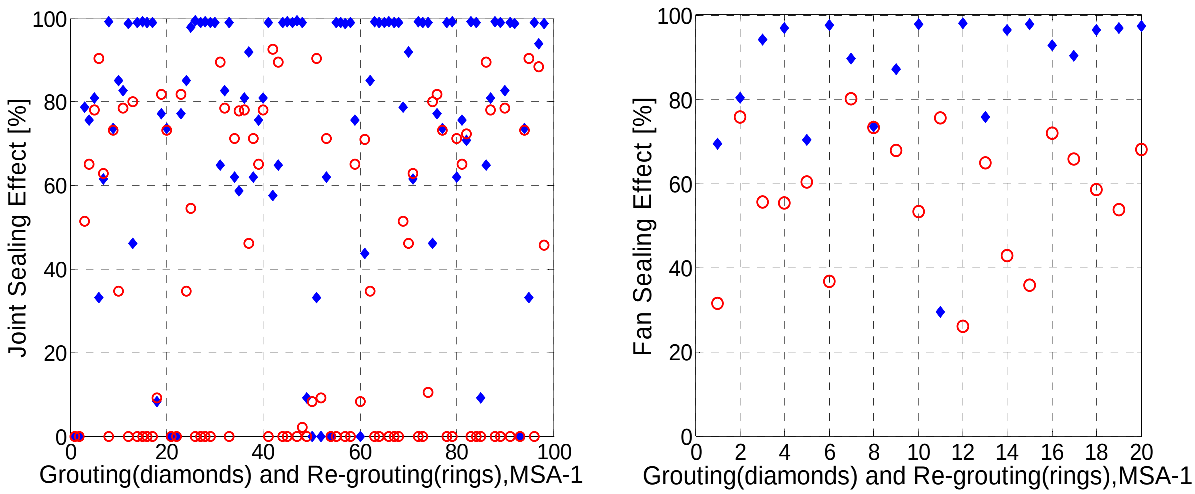


Figure 5-10 Joint- and fan sealing effect after grouting and re-grouting for MSA-1.

In Figure 5-11 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 2000 seconds implies a high sealing effect close to 100 %, shorter grouting implies both a lower, as well as, a medium sealing effect. For re-grouting a sealing effect of between 0 % and 90 % is achieved independent of the time taken.

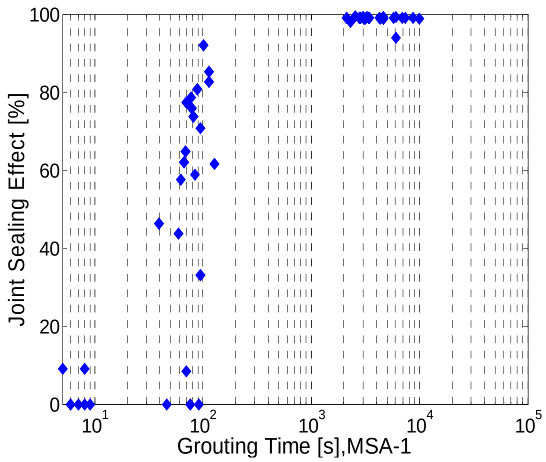


Figure 5-11 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-12 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 3000 seconds imply a sealing effect between 80 % and 100 %, shorter times imply lower sealing effect. Independent of time, the sealing effect was between 80 % and 100 %. Compared to other rock masses the fan sealing effect for rock mass A is low, which may be explained by the low joint intensity and the narrow joint apertures. Some of the randomized fans in rock mass A consisted only of a few very narrow joints.

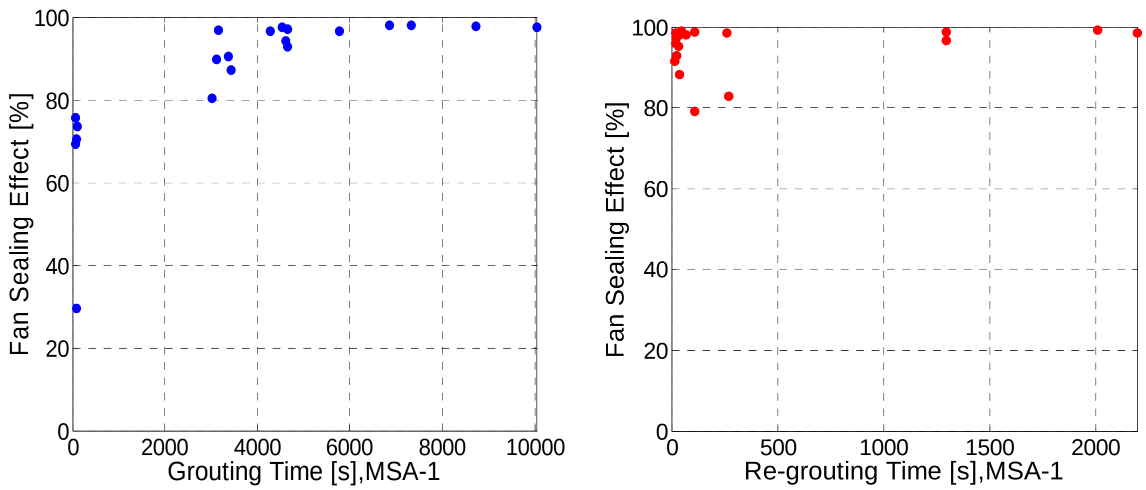


Figure 5-12 Accumulated sealing effect on a fan level after grouting and re-grouting.

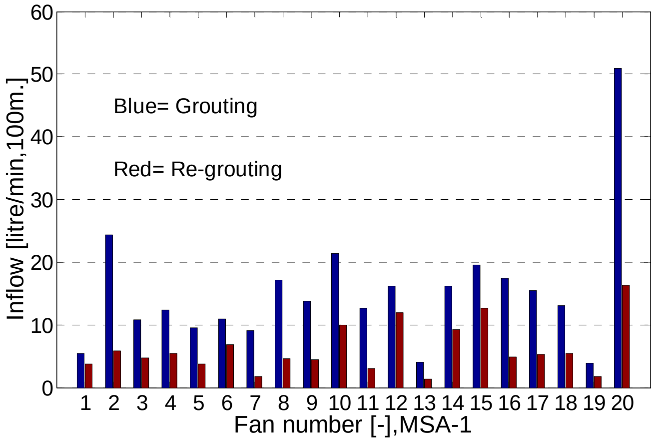
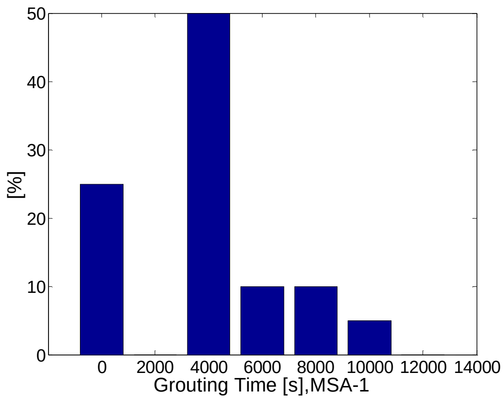


Figure 5-13 Inflow to the tunnel from the individual grout fans.

In Figure 5-13 the inflow from the individual fans after grouting and re-grouting are shown. The high inflow in fan 20, has been analysed, and is very dependent on one single joint with an aperture of 0.45 mm. If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 18 of 20 fans the inflow after grouting is above the requirement. In 10 of 20 of the holes requiring re-grouting, the inflow is above that required even after re-grouting. A third grouting or more would be necessary to fulfil the requirement for the remaining fans.

The calculation of the flow for the different fans gave re-grouting in 18 of 20 fans, which could be compared with the re-grouting criteria for the control holes that only gave one re-grouting. To compare them are difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-14 the grouting time distribution for the fans and the calculated inflow after grouting with MSA-1 are shown.



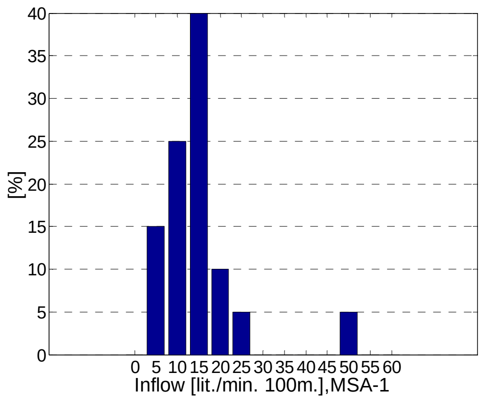


Figure 5-14 Fan grouting time- and fan inflow distribution for method MSA-1.

The average grouting time for a fan was calculated to 3901 seconds (1 hour and 5 minutes), with a standard deviation of 2922 seconds (49 min.) ,50 % of the fans have a grouting time close to 4000 seconds. The average inflow after grouting was calculated to 15.2 litres/min/100 meters, with a standard deviation of 10.0 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 7.41 m^3 or as an average of 371 litres per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-2. The control holes have not been able to localize the conductive structures, and thereby suggest re-grouting.

Table 5-2 Properties on a fan level of MSA-1.

	After Grouting	After grouting + control holes re-grouting	After full re-grouting
Sealing effect for a fan* [%]	μ=86.4 σ=16.7	μ=86.5 σ=16.8	μ=95.2 σ=5.7
Time for a fan [s]	μ=3901 σ=2922	μ=4002 σ=3168	μ=4298 σ=3309

*Average sealing effect for 20 fans.

5.1.3 Comparison on tunnel level

On a tunnel, level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grouting time to perform 20 fans with method MS-1 in rock mass A, was calculated to 78 027 seconds. To perform re-grouting for all fans was calculated to 7943 seconds, which is about 10 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans which need re-grouting if the inflow criteria is set to 5 litres/min. 100 meters, have been calculated and are shown in Table 5-3.

Table 5-3 Properties on a tunnel level of MSA-1.

	Before	After Grouting	After grouting + control holes re-grouting	After full re-grouting
Inflow to tunnel [l/min, 20 fans]	1523	61	52	25
Sealing effect for a tunnel* [%]		96.0	96.5	98.4
Time for a tunnel [s]		78 027	80 036	85 970
Number of fans above 5 litres/min/100 meters	20	18	18	10

*based on total inflow and total reduction for 20 fans.

After grouting the method MSA-1 in rock mass A fulfils an inflow requirement of 15 litres/min/100 meters and after full re-grouting an inflow requirement of 6 litres/min/100 meters is achieved.

5.2 Result Method MS, Rock mass A (MSA)

5.2.1 Comparison on joint level

The grouting method, MS, has been applied to rock mass A. The basis for the evaluation is the calculation of each possible joint in the interval 0 – 500 µm, which gives 200 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-15. As shown, both the time and the volume are dependent on the joint aperture.

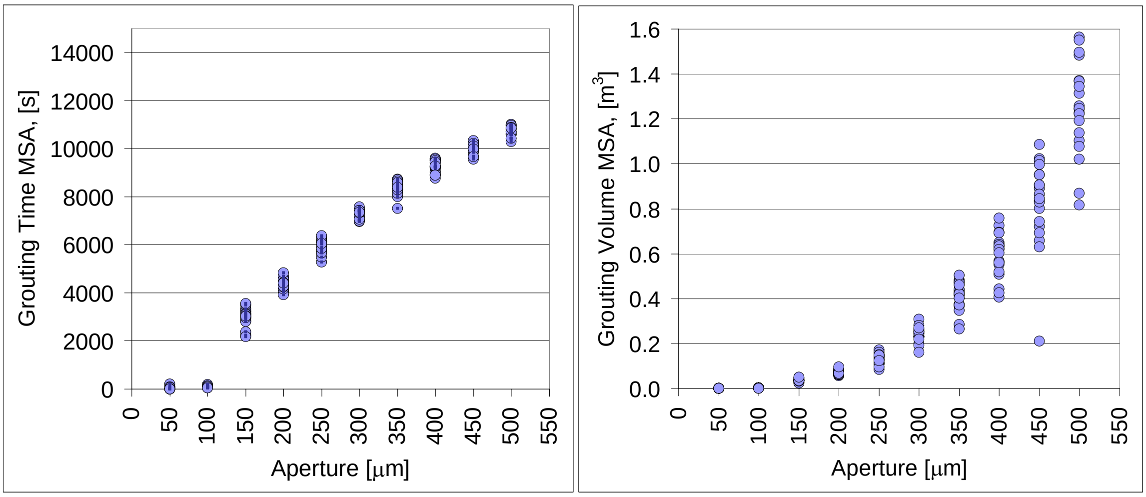


Figure 5-15 Grouting time and grouting volume for different apertures MSA.

All the joints have also been re-grouted, as shown in Figure 5-16. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting time is not increased by a larger aperture, which could be because the initial grouting already sealed most of the wider joints to a high level.

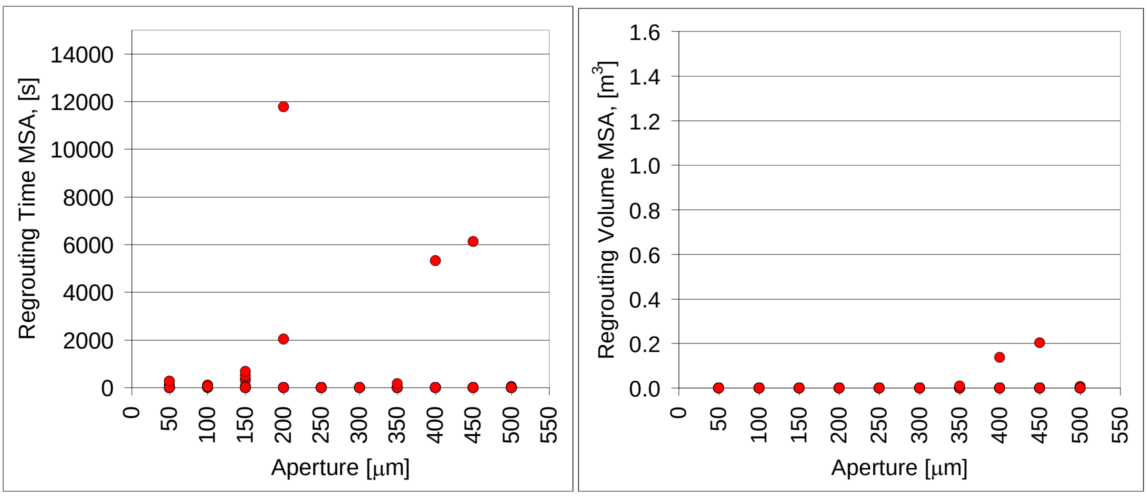


Figure 5-16 Re-grouting time and re-grouting volume for different apertures MSA.

If instead the re-grouting is plotted on a logarithmic scale, the difference between small values could be more clearly noted, as shown in Figure 5-17.

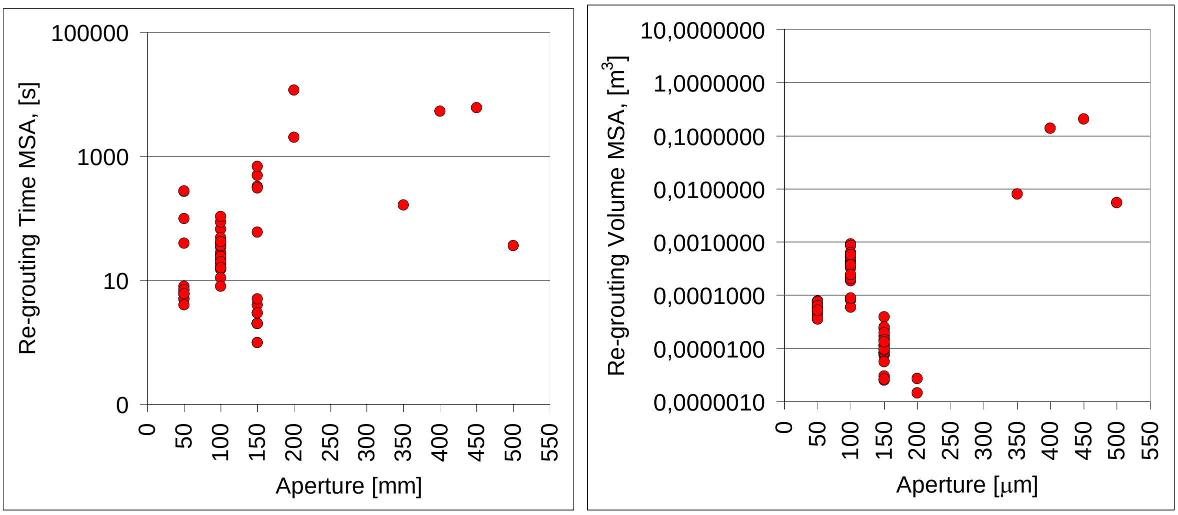


Figure 5-17 Re-grouting time and re-grouting volume for different apertures MSA on a logarithmic scale (values equal to zero can not be plotted).

The sealing effect, as shown in Figure 5-18, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is high for apertures larger than 150 µm, but low for smaller apertures. For joints with an aperture of 50 µm the sealing time, as well as, the sealing effect were almost zero. The re-grouting improved joints mostly with an aperture of 50 µm and 100 µm.

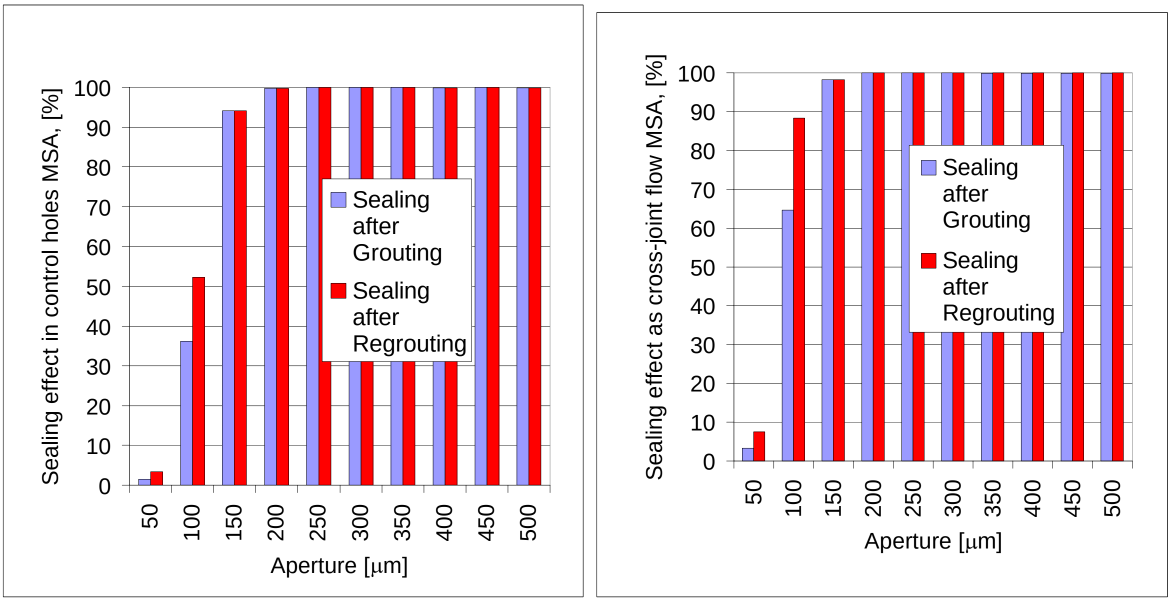


Figure 5-18 Sealing effect for different apertures MSA, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.2.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

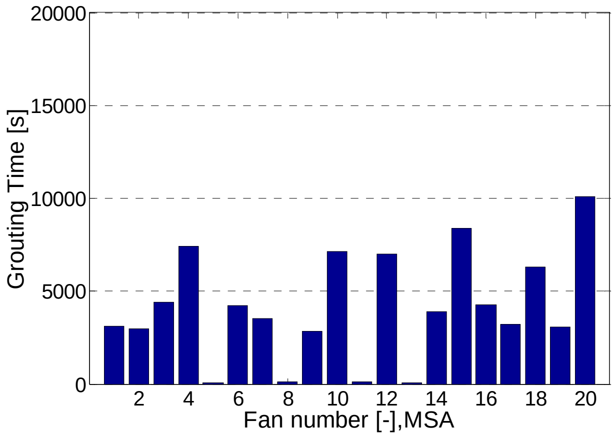


Figure 5-19 Grouting time for the individual grout fans MSA.

In Figure 5-19 the grouting time for the individual fans are shown. The average grout time was 4120 seconds, with a standard deviation of 2884 seconds. For this method 20 % of the fans had a grouting time close to zero, which is similar to method MSA-1. The explanation is that those fans only consist of joints with small apertures, almost unreachable for this method.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-20 the inflow for method MS in rock mass A is shown. From the figure, the large difference before and after grouting could be noticed. Between first and second grouting the flow was further reduced, but of much less magnitude.

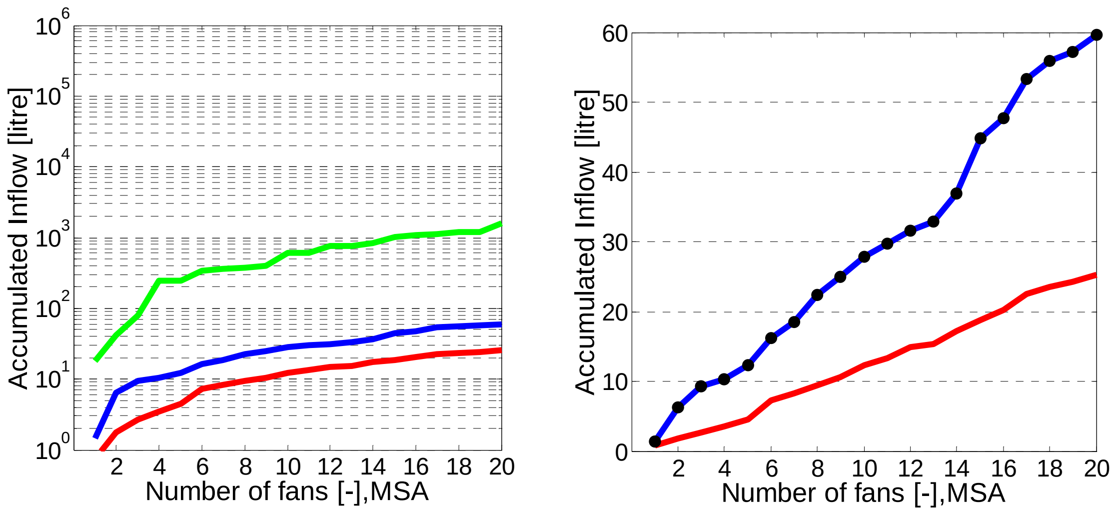


Figure 5-20 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

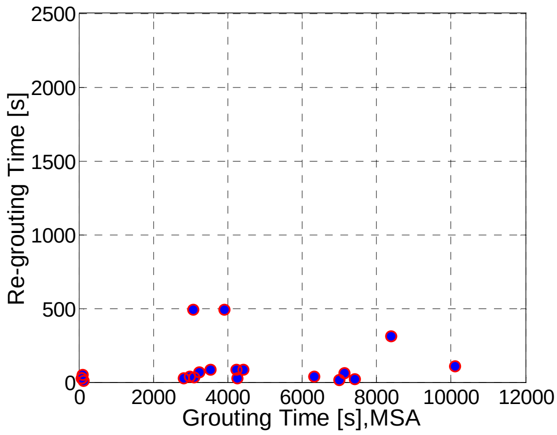
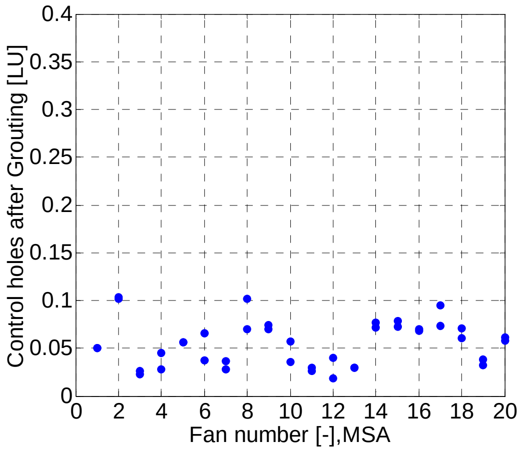


Figure 5-21 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-21 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and re-grouting time for MSA.



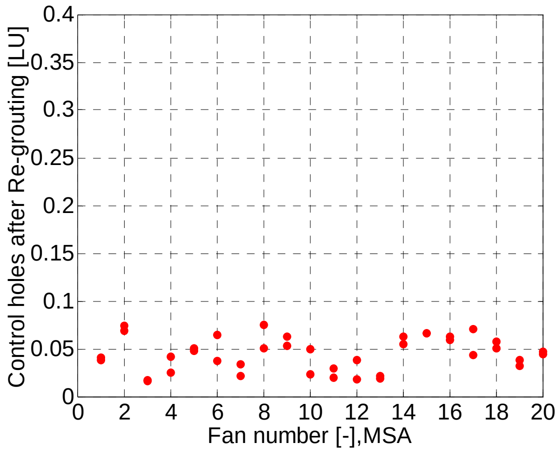


Figure 5-22 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For none of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-22 two control holes for each fan half after grouting and after re-grouting are shown. None of the control holes had a Lugeon value after grouting equal to or above the set criteria of 0.15 Lugeon. The control holes shown in Figure 5-22 didn't result in any re-groutings.

On the other hand, if re-grouting is carried out and tested, none of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently not be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-23 the probability to be below the inflow requirement is shown. The probability to not fulfil the inflow requirement is calculated as the number of fans, which has an inflow above each inflow requirement. If the inflow requirement after grouting is below 4 litres/min/100 meters for method MSA, the chance to not fulfil the requirement is 100%. If on the other hand the inflow criterion is above 39 lit/min/100 meters the chance to not fulfil the requirement is 0%.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-23 the inflow convergence stabilizes after about 15 fans for the grouting of MSA and after about 6 fans for the re-grouting.

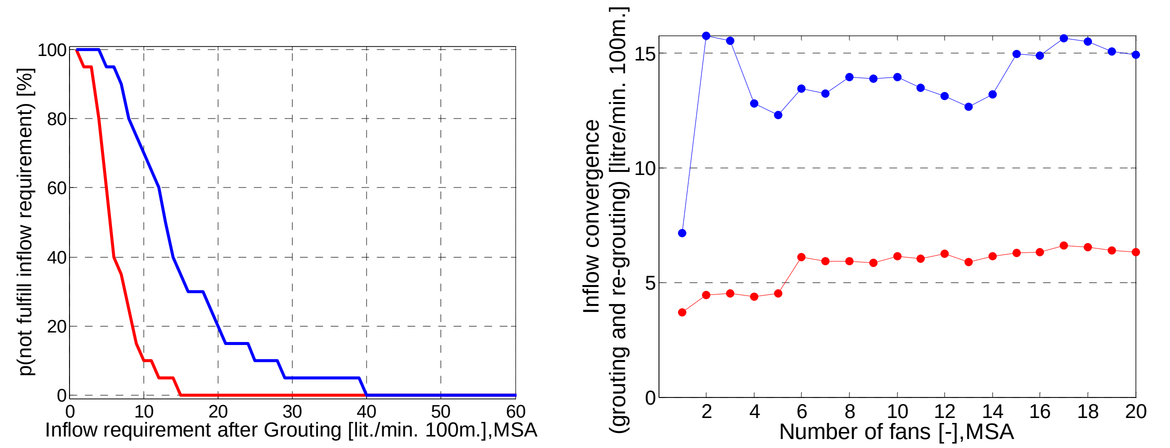


Figure 5-23 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSA (left). Inflow convergence after n number of fans for MSA (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level are shown in Figure 5-24. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level grouting reaches a high sealing effect and re-grouting a much lower. On the fan level the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. Still the fan sealing effect after grouting for MSA is quite low, which is because of a low joint intensity and a high amount of narrow joints for rock mass A.

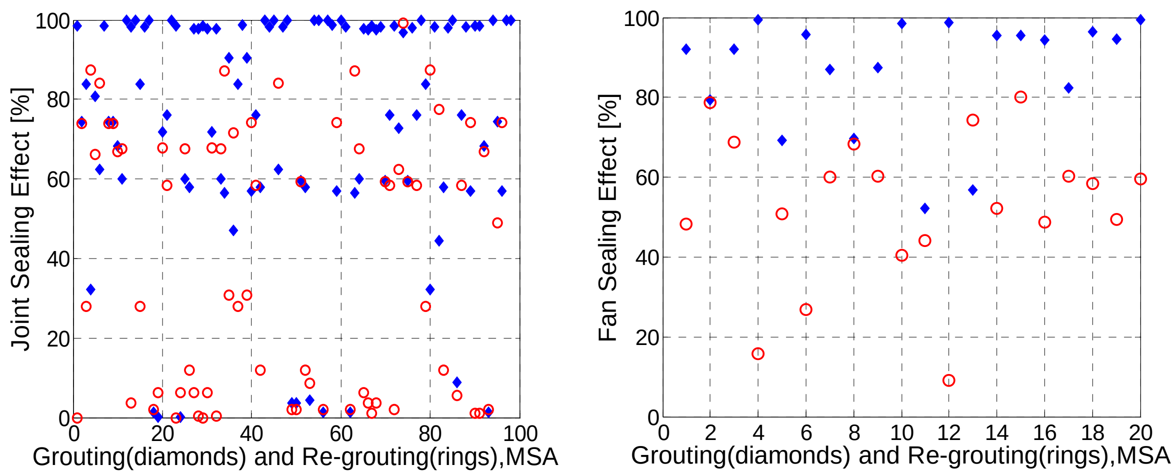


Figure 5-24 Joint and fan sealing effect after grouting and re-grouting for MSA.

In Figure 5-25 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level are shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 2500 seconds implies a sealing effect close to 100 %, shorter grouting time implies lower sealing effect. For re-grouting a sealing effect of between 0 % and 90 % is achieved independent of the time taken.

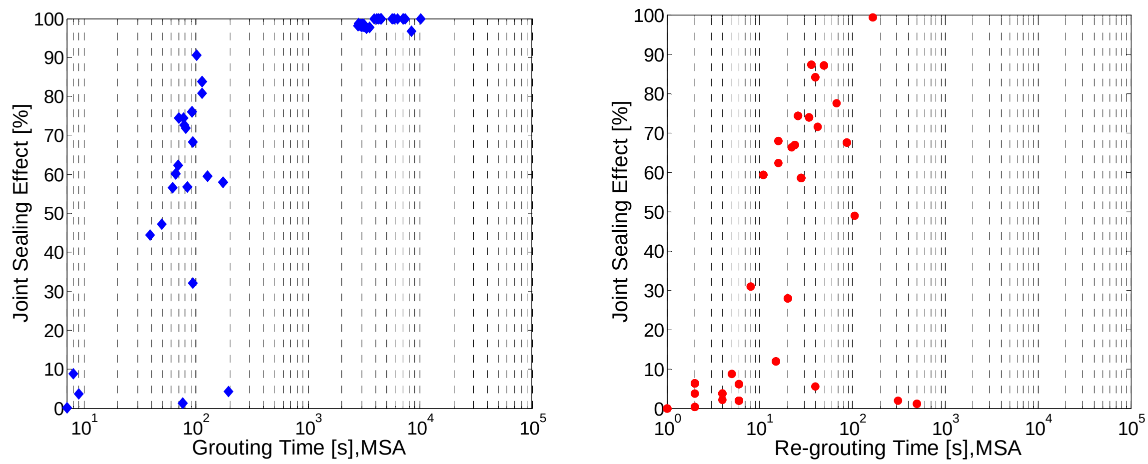


Figure 5-25 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-26 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 2500 seconds implies a sealing effect between 80 % and 100 %, shorter times implies lower sealing effect. Re-grouting for more than 100 seconds implies a high sealing effect, shorter times implies medium sealing effect. The sealing effect is similar between grouting and re-grouting, which imply that the method can not increase the sealing effect further in this rock mass.

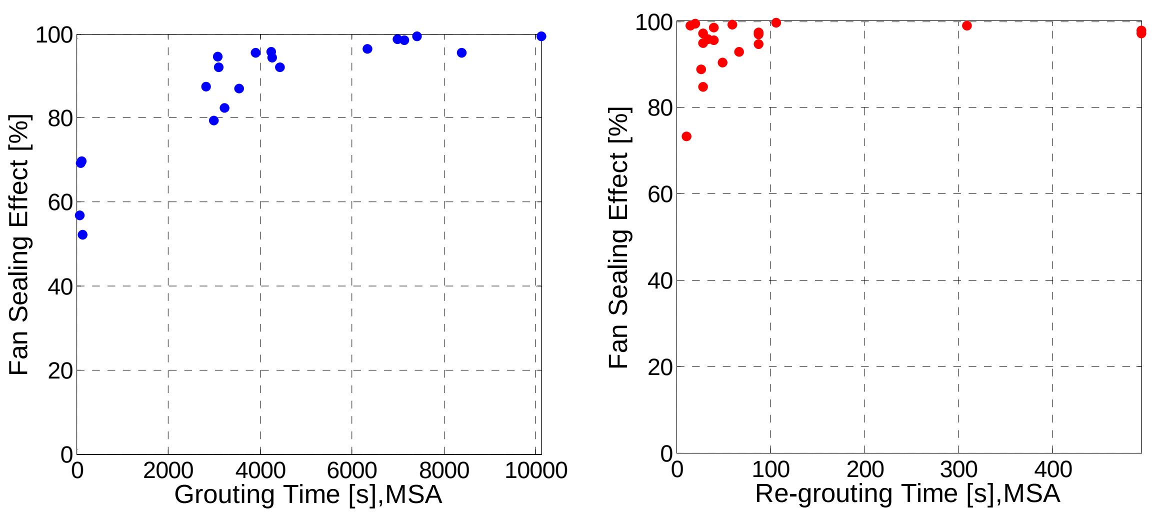


Figure 5-26 Accumulated sealing effect on a fan level after grouting and re-grouting.

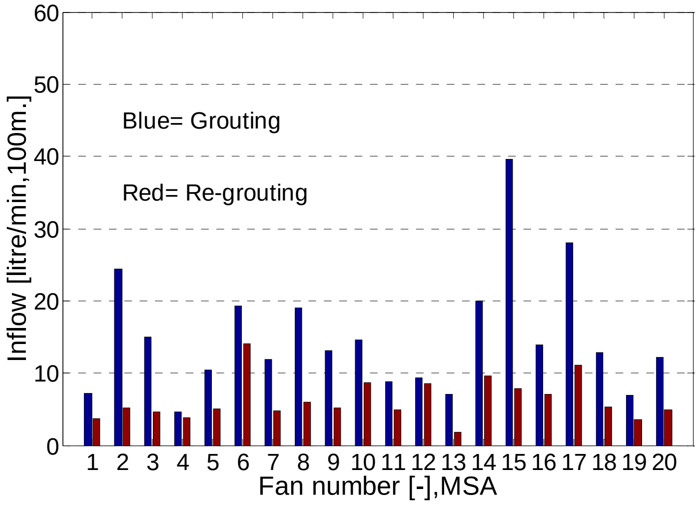


Figure 5-27 Inflow to the tunnel from the individual fans.

In Figure 5-27 the inflow from the individual fans after grouting and re-grouting are shown. If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 19 of 20 fans the inflow after grouting is above the requirement and in 12 of 20 fans the inflow is above the requirement, even after re-grouting. A third grouting or more would be necessary to fulfil the requirement for the remaining fans.

The calculation of the flow for the different fans gave re-grouting in 19 of 20 fans, which could be compared with the re-grouting criteria based on the control holes that gave no re-groutings. To compare them is difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-28 the grouting time distribution for the fans and the calculated inflow after grouting for MSA are shown.

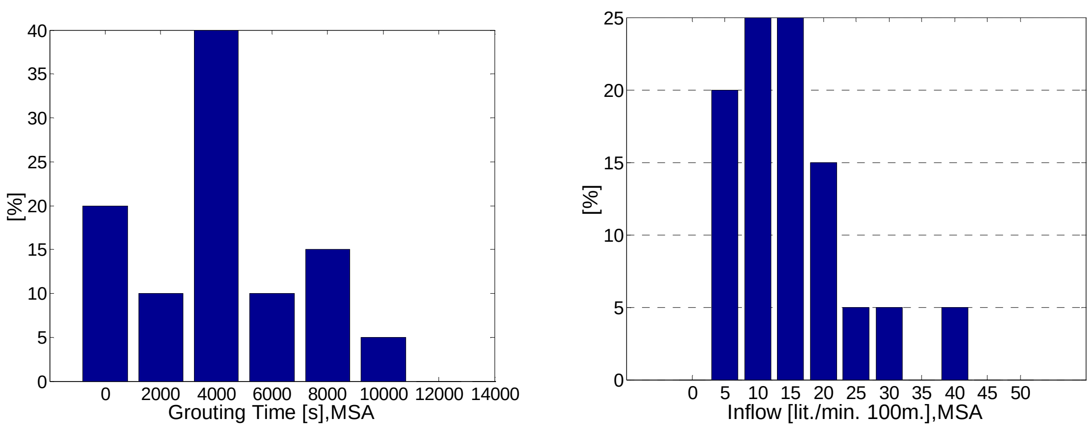


Figure 5-28 Fan grouting time- and fan inflow distribution for method MSA.

The average grouting time for a fan was calculated to 4120 seconds (1 hour and 8 minutes), with a standard deviation of 2884 seconds (48 min.), 50 % of the fans had a grouting time close to 4000 seconds. The average inflow after grouting was calculated to 14.9 litres/min/100 meters, with a standard deviation of 8.4 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 8.25 m^3 or as an average of 410 litres per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-4. As shown, the control holes have not been able to localize the conductive structures, and thereby suggest re-grouting.

Table 5-4 Properties on a tunnel level of MSA.

	After grouting	**After grouting + control holes re-grouting**	**After full re-grouting**
Fan Sealing Effect* [%]	μ=86.8 σ=14.2	μ=86.8 σ=14.2	μ=94.7 σ=6.3
Fan Time [s]	μ=4120 σ=2884	μ=4120 σ=2884	μ=4225 σ=2908

*Average sealing effect for 20 fans.

5.2.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grouting time to perform 20 fans with method MS in rock mass A, was calculated to 82 393 seconds. To perform re-grouting for all fans was calculated to 2110 seconds, which is about 2.5 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-5.

Table 5-5 Properties on a tunnel level of MSA.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After full re-grouting**
Inflow to tunnel [l/min, 20 fans]	1577	60	60	25
Sealing effect for a tunnel* [%]		96.2	96.2	98.4
Time for a tunnel [s]		82 393	82 393	84 503
Number of fans above 5 litres/min/100 meters	20	19	19	12

*based on total inflow and total reduction for 20 fans.

After grouting the method MSA in rock mass A fulfils an inflow requirement of 15 litres/min/100 meters and after full re-grouting an inflow requirement of 6.25 litres/min/100 meters is achieved.

5.3 Result Method MA, Rock mass A (MAA)

5.3.1 Comparison on joint level

The grouting method, MA, has been applied to rock mass A. The basis for the evaluation is the calculation of each possible joint in the interval 0 – 500 µm, which gives 200 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-29. As shown, both the time and the volume are dependent on the joint aperture.

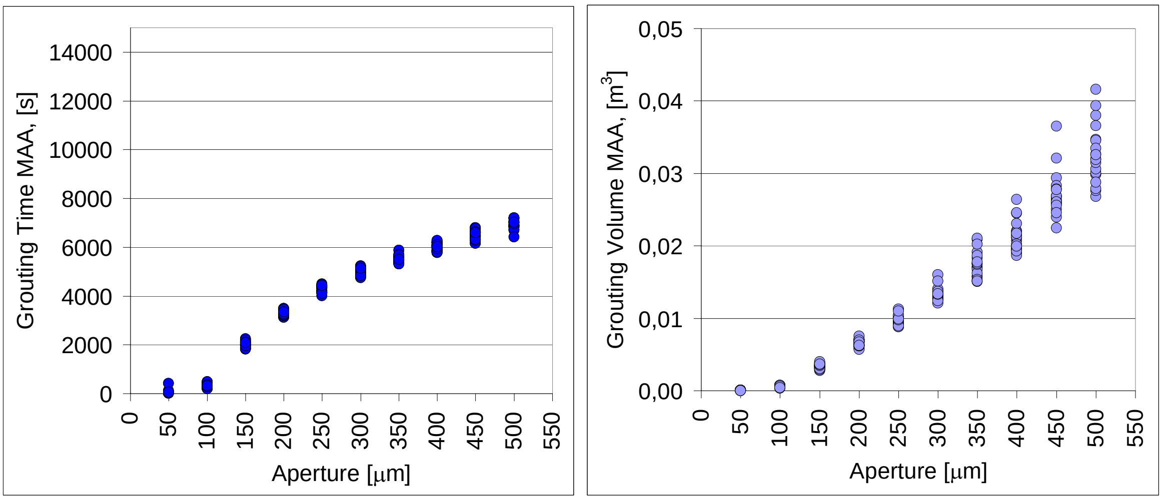


Figure 5-29 Grouting time and grouting volume for different apertures MAA.

All joints have been re-grouted, as shown in Figure 5-30. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting time is increased by a larger aperture, which indicates that the method needs a re-grouting or that the penetration length is too short for the chosen hole spacing. The variance, within an aperture multiple, is higher for the re-grouting than for the grouting, which also may be an indication of a too short penetration length.

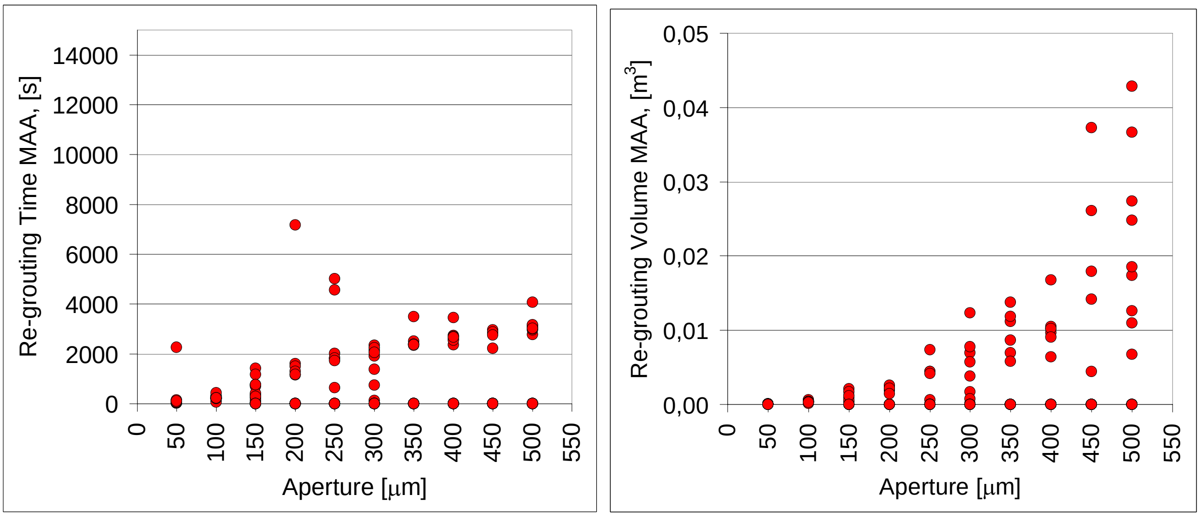


Figure 5-30 Re-grouting time and re-grouting volume for different apertures MAA.

The sealing effect, as shown in Figure 5-31, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is low for apertures between 50 µm and 100 µm. For apertures between 150 µm to 500 µm the sealing effect was around 90 %. A re-grouting improves all apertures, which also indicates that the penetration length may be too short for the chosen hole spacing.

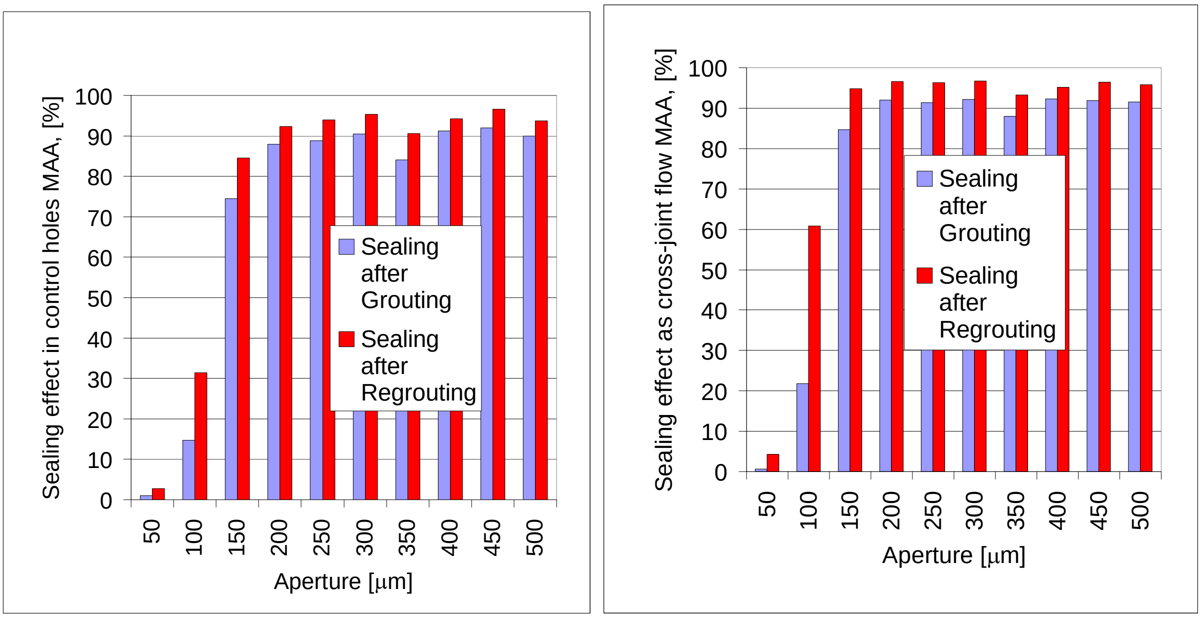


Figure 5-31 Sealing effect for different apertures MAA, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.3.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

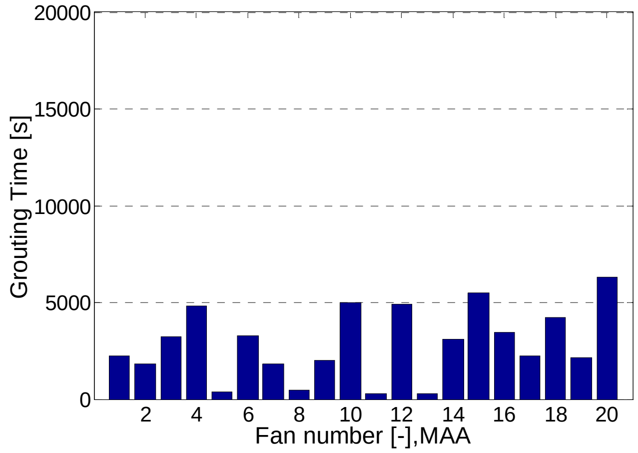


Figure 5-32 Grouting time for the individual grout fans MAA.

In Figure 5-32 the grouting time for the individual fans are shown. The average grout time was 2883 seconds, with a standard deviation of 1820 seconds. For this method 20 % of the fans had a grouting time close to zero. The explanation is that these fans only consist of joints with small apertures, which are almost unreachable for this method.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-33 the inflow for method MA in rock mass A is shown. From the figure the large difference before and after grouting could be noticed. Between grouting and re-grouting the flow was further reduced, but by a much lesser magnitude.

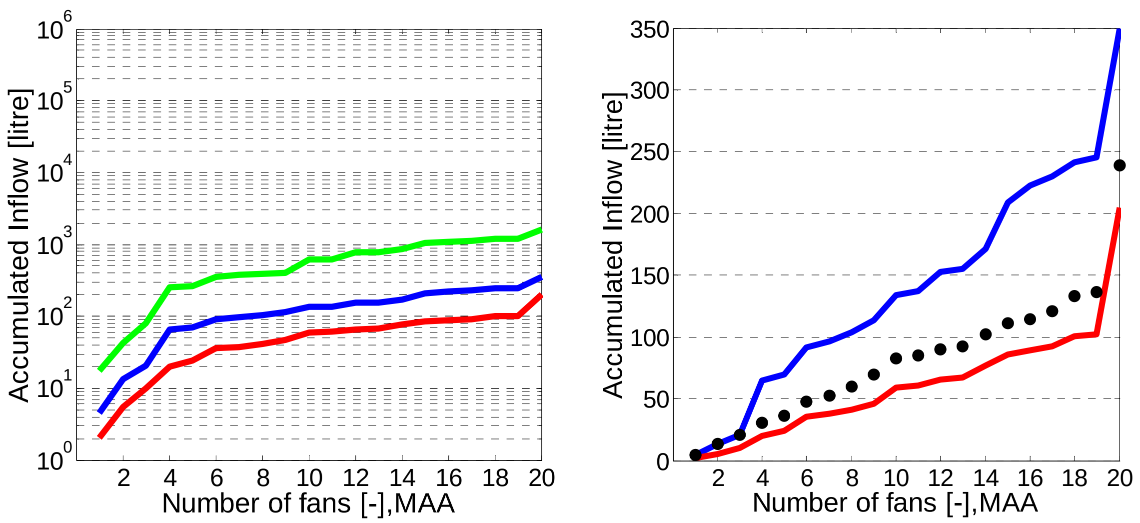


Figure 5-33 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

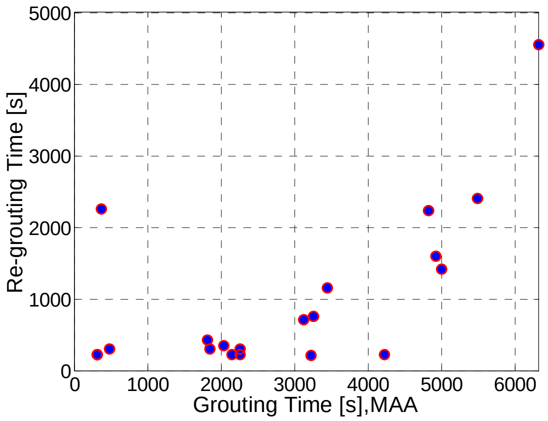


Figure 5-34 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. As shown in Figure 5-34 a higher grouting time has a small tendency to give a higher re-grouting time for method MA in rock mass A.

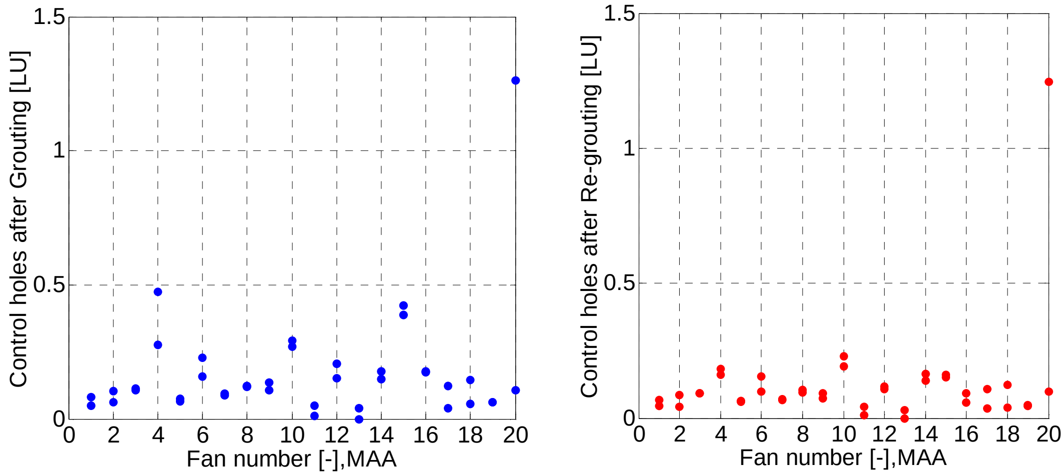


Figure 5-35 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 14 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-35 two control holes for each fan half, after grouting and after re-grouting, are shown. 14 of the control holes after grouting had a Lugeon value equal to or above the set criteria of 0.15 Lugeon. The control holes shown in Figure 5-35 resulted in 8 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. For grout fan 4, 6, 10, 12, 14, 15, 16, 20 the re-grouting time is 14 830 seconds.

On the other hand, if re-grouting is carried out and tested, 6 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For 6 fans the control holes have a Lugeon value above 0.15 LU and should, if the method suggested, consequently be re-grouted a second time, which not is performed within this project.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-36 the probability to be below the inflow requirement is shown. The probability to not fulfil the

inflow requirement is calculated as the number of fans, which has an inflow above each inflow requirement. If the inflow requirement after grouting is below 12 litres/min/100 meters for method MAA, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 524 litres/min/100 meters the chance to not fulfil the requirement is 0 %.
The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-9 the inflow convergence initially stabilizes after about 7 fans for the grouting of MAA and after about 5 fans for the re-grouting. Due to one very wide joint the inflow increases, as shown, in fan number 20.

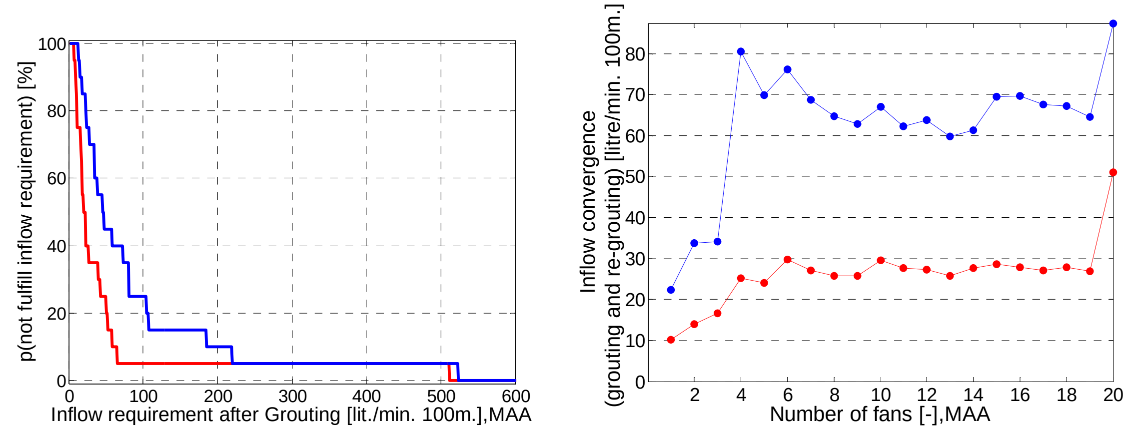


Figure 5-36 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MAA (left). Inflow convergence after n number of fans for MAA (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-37. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level neither grouting nor re-grouting reaches a high sealing effect. On a fan level the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. Still the fan sealing effect after grouting for MAA is low, which is because of a low joint intensity and a high amount of narrow joints for rock mass A. The two figures below are an indication that the grouting method is not suitable for the rock mass.

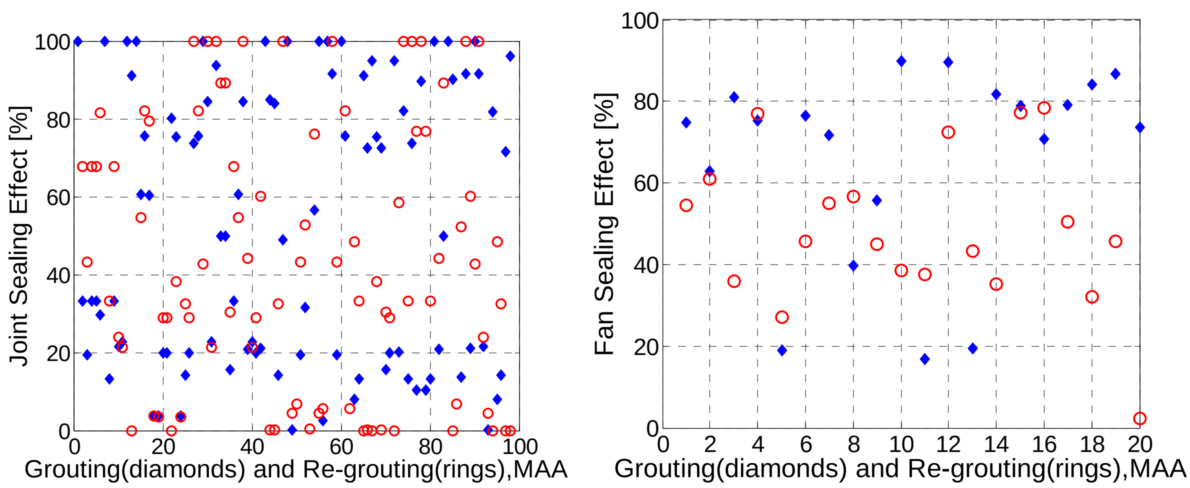


Figure 5-37 Joint and fan sealing effect after grouting and re-grouting for MAA.

In Figure 5-38 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level are shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 2000 seconds imply a sealing effect between 50 % and 100 %, shorter grouting time implies lower sealing effect between 0 % and 50 %. Re-grouting performed for more than 200 seconds imply a sealing effect of between 20 % and 100 %, shorter re-grouting time implies lower sealing effect.

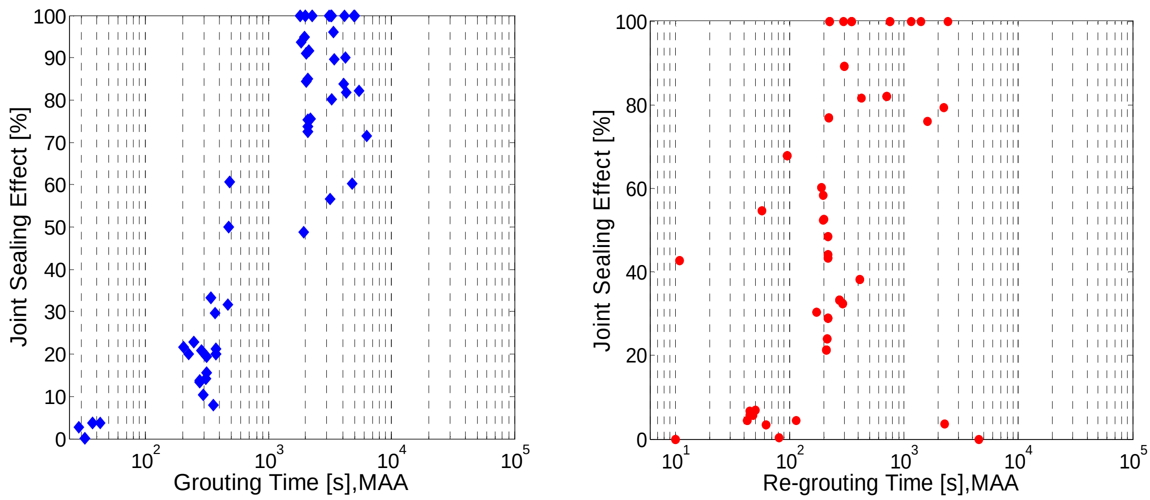


Figure 5-38   Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-39 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 2000 seconds implies a sealing effect between 60 % and 90 %, shorter times implies lower sealing effect. Independent of time, re-grouting implies a sealing effect between 40 % and 100 %. The re-grouting further implies a small increase of the sealing effect, as the lower values were raised from 20 % to 40 % and as the median was raised from ~80 % to ~90 %.

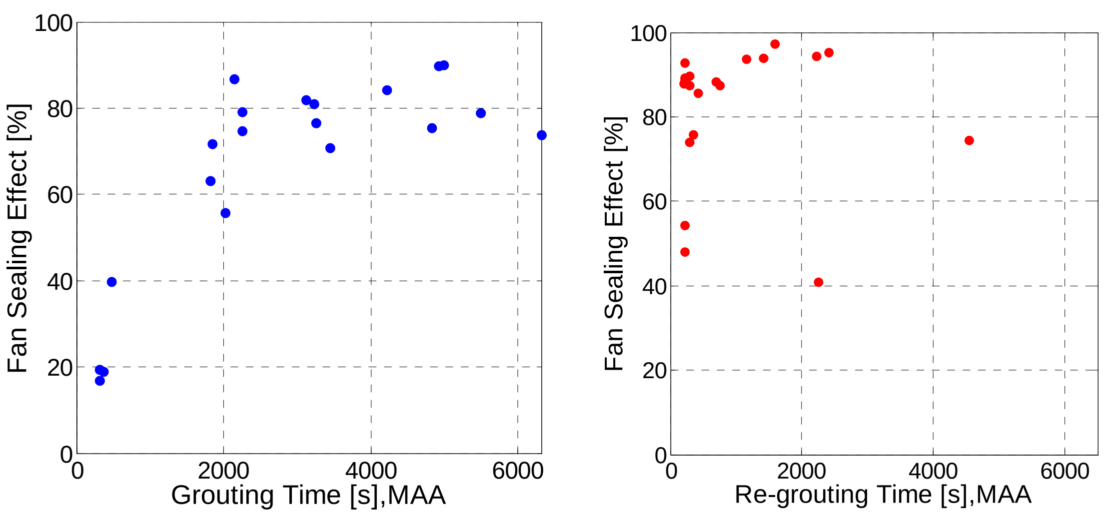


Figure 5-39   Accumulated sealing effect on a fan level after grouting and re-grouting.

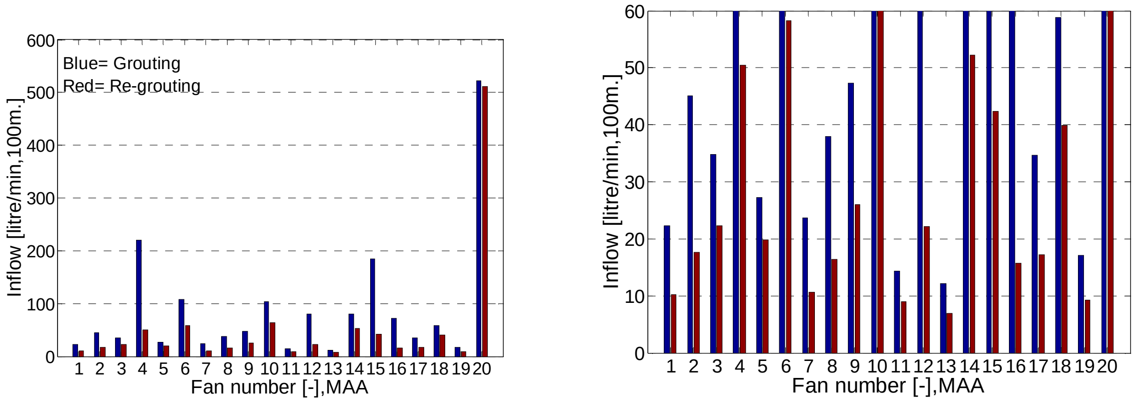


Figure 5-40 Inflow to the tunnel from the individual grout fans shown in two figures.

In Figure 5-40 the inflow from the individual fans after grouting and after re-grouting are shown. If the inflow requirement of 5 litres/min/100 meters is applied on a fan length of 20 meters and substituted to 1 litre/min/20 meters then in 20 of 20 fans the inflow after grouting and re-grouting is above the criteria. As shown the re-grouting will reduce the inflow, but to fulfil the requirement this method may not be optimal.

The calculation of the flow for the different fans gave re-grouting in 20 of 20 fans, which could be compared with the re-grouting criteria for the control holes that only gave 8 re-grouting. To compare them are difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-41 the grouting time distribution for the fans and the calculated inflow after grouting for MAA are shown.

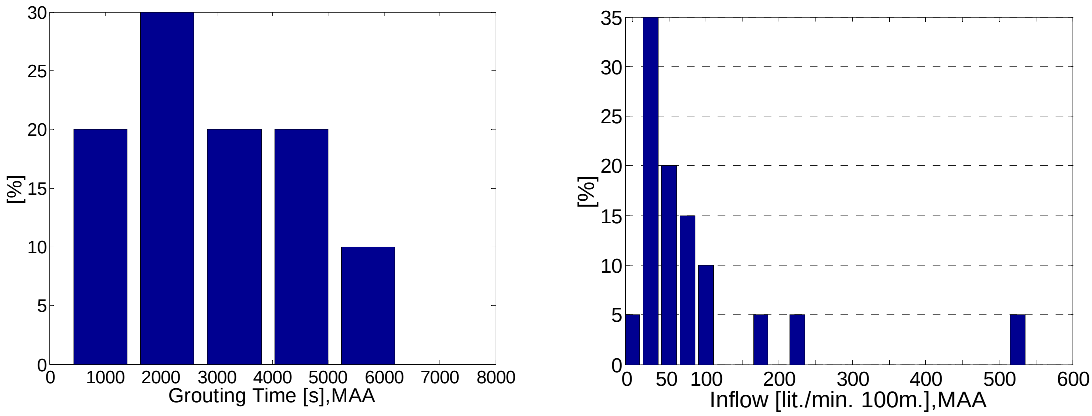


Figure 5-41 Fan grouting time- and fan inflow distribution for method MAA

The average grouting time for a fan was calculated to 2883 seconds (48 minutes), with a standard deviation of 1820 seconds (30 min.), 30 % of the fans had a grouting time close to 2000 seconds. The average inflow after grouting was calculated to 87 litres/min/100 meters, with a standard deviation of 116 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 0.60 m^3 or as an average of 30 litres per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-6. As shown, the control holes have to some extent been able to localize the conductive structures, and thereby suggest re-grouting. The control holes can for this situation, depending on the requirements, be regarded as a good parameter for judging the need for re-grouting.

Table 5-6 Properties on a fan level of MAA.

	After grouting	After grouting + control holes re-grouting	After full re-grouting
Sealing Effect for a fan* [%]	μ=66.4 σ=23.7	μ=70.7 σ=26.7	μ=81.9 σ=16.3
Time for a fan [s]	μ=2883 σ=1820	μ=3625 σ=2880	μ=3887 σ=2686

*Average sealing effect for 20 fans.

5.3.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grouting time to perform 20 fans with method MA in rock mass A, was calculated to 57 666 seconds. To perform re-grouting for all fans was calculated to 14 830 seconds, which is about 25 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-7.

Table 5-7 Properties on a tunnel level of MAA.

	Before	After grouting	After grouting + control holes re-grouting	After full re-grouting
Inflow to tunnel [l/min, 20 fans]	1617	350	238	204
Sealing effect for a tunnel* [%]		78.4	85.2	87.4
Time for a tunnel [s]		57 666	72 496	77 741
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MAA in rock mass A fulfils an inflow requirement of 87 litres/min/100 meters and after full re-grouting an inflow requirement of 51 litres/min/100 meters is achieved.

5.4 Result Method MS-1, Rock mass B (MSB-1)

5.4.1 Comparison on joint level

The grouting method, MS-1, has been applied to rock mass B. The basis for the evaluation is the calculation of each possible joint in the interval 0-1000 µm, which gives 400 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-42. As shown, both the time and the volume are dependent on the joint aperture.

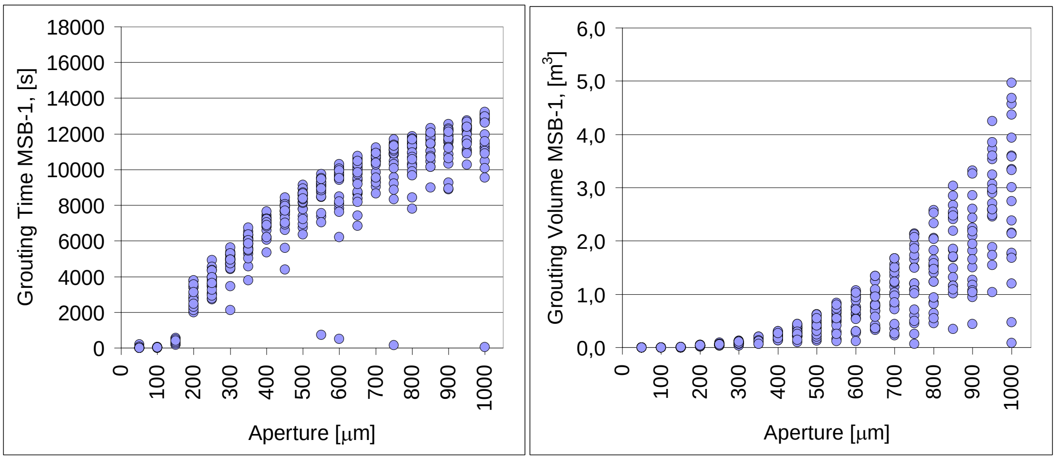


Figure 5-42 Grouting time and grouting volume for different apertures MSB-1.

All the joints have also been re-grouted, as shown in Figure 5-43. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting time is not increased by a larger aperture for apertures up to around 800 µm, which could be because the initial grouting already sealed most of those joints to a high level. For the joint above 800 µm the re-grouting time is increased by larger aperture, which could, for example, be because grout separation in wide joints creates conductive channels, which then are sealed by the re-grouting. The variance, within an aperture multiple, is higher for the re-grouting than for the grouting, which could be because the joints are not fully reached by the hole spacing chosen for the grouting.

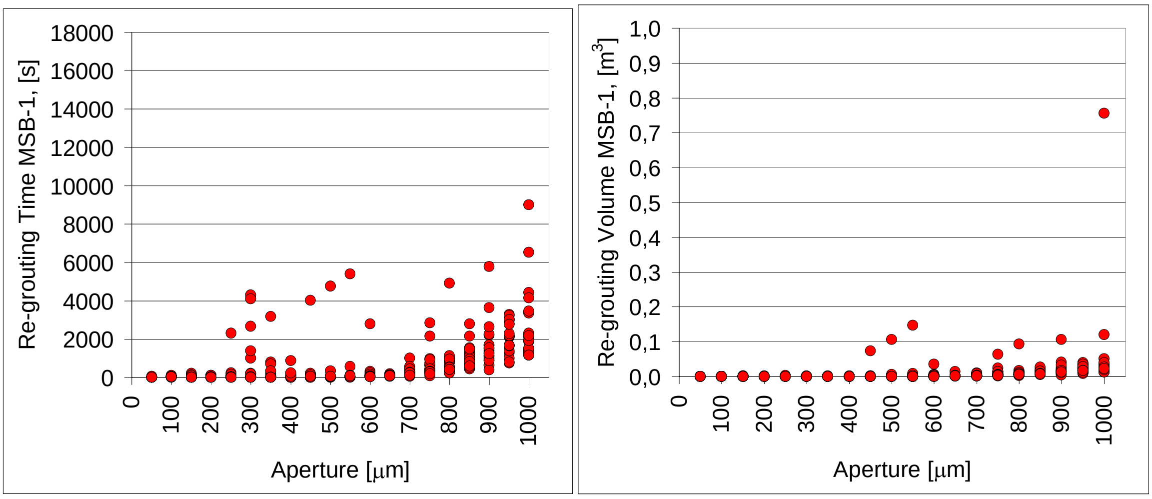


Figure 5-43 Re-grouting time and re-grouting volume for different apertures MSB-1.

The sealing effect, as shown in Figure 5-44, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is high for apertures larger than 150 μm, but low for smaller apertures. For joints with an aperture of 50 μm the sealing time as well as the sealing effect was almost zero. Joints with apertures in the range of 100 μm were mostly improved by re-grouting.

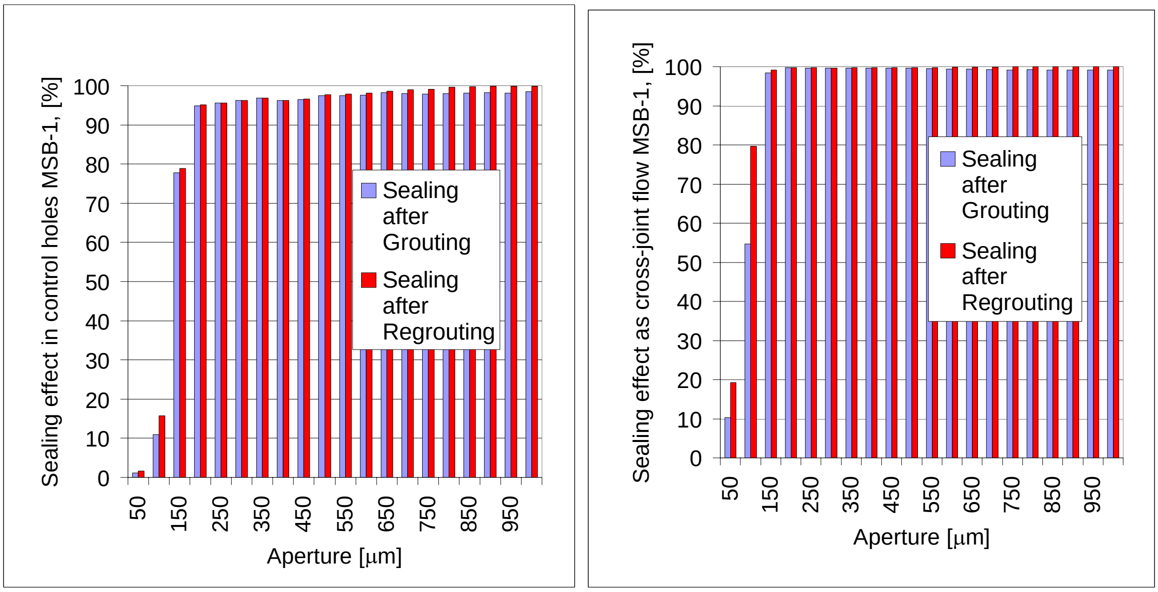


Figure 5-44 Sealing effect for different apertures MSB-1, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.4.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

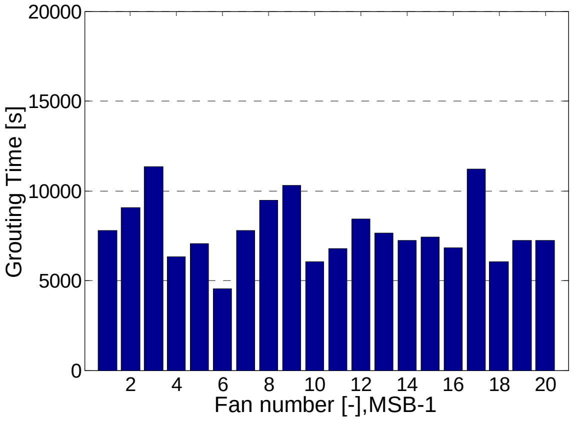


Figure 5-45 Grouting time for the individual grout fans (MSB-1).

In Figure 5-45 the grouting time for the individual fans are shown. The average grouting time was 7789 seconds, with a standard deviation of 1744 seconds. The grouting time for this method is more evenly distributed here than for this method in rock mass A, which is because of a higher joint intensity in rock mass B. The average time is higher and the standard deviation is lower than in rock mass A, which is also because of a higher joint intensity. In rock mass A it was possible to get fans with just a few narrow apertures, which resulted in 25 % of the fans which had a grouting time close to zero. As shown in Figure 5-45 none of the fans have a grouting time close to zero.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-46 the inflow for method MS-1 in rock mass B is shown. From the figure a large difference before and after grouting could be noticed. Between grouting and re-grouting, the flow was further reduced, but by a much lesser magnitude.

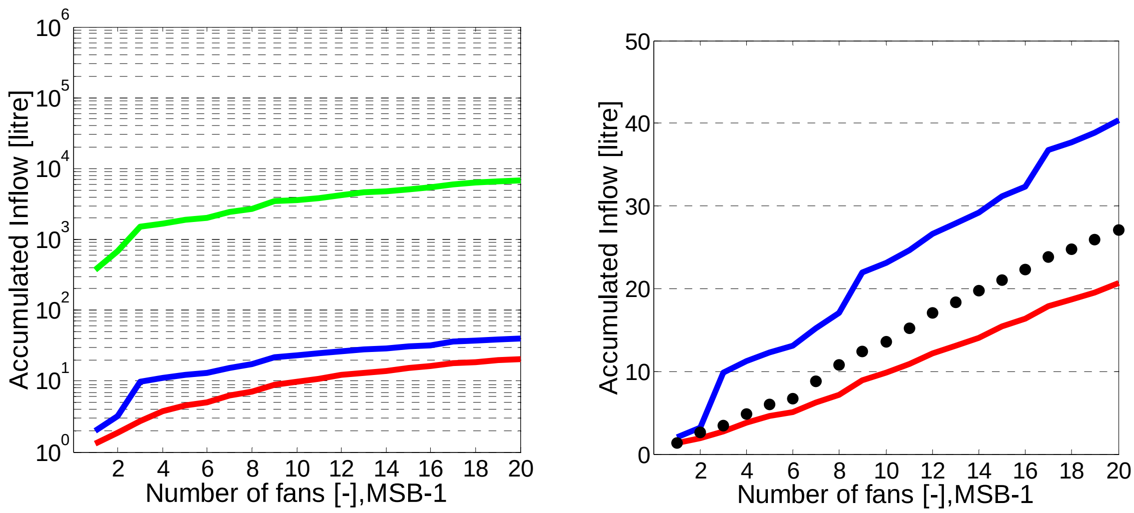


Figure 5-46 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

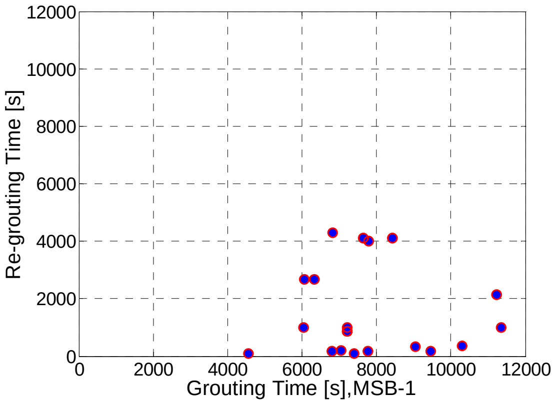
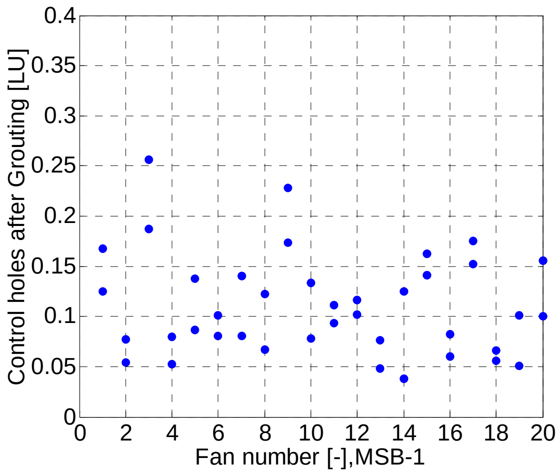


Figure 5-47 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-7 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and re-grouting time for MSB-1.



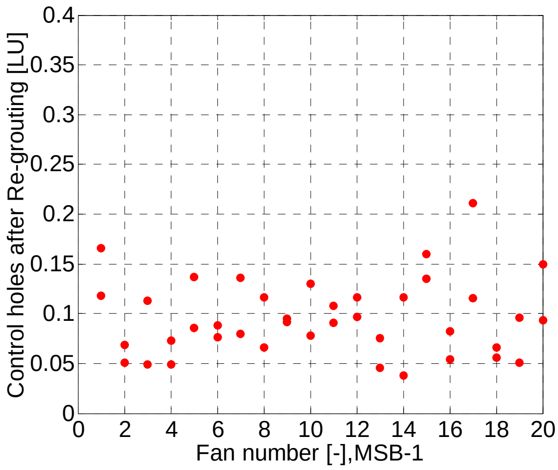


Figure 5-48 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 8 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-48 two control holes for each fan half after grouting and after re-grouting are shown. One of the control holes had a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-48 resulted in 6 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. For grout fans 1, 3, 9, 15, 17, and 20 the re-grouting time is 4806 seconds.

On the other hand, if re-grouting is carried out and tested, 3 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-49 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting is below 3 litres/min/100 meters for method MSB-1, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 33 litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-49 the inflow convergence stabilizes after about 10 fans for the grouting of MSB-1 and after about 2 fans for the re-grouting.

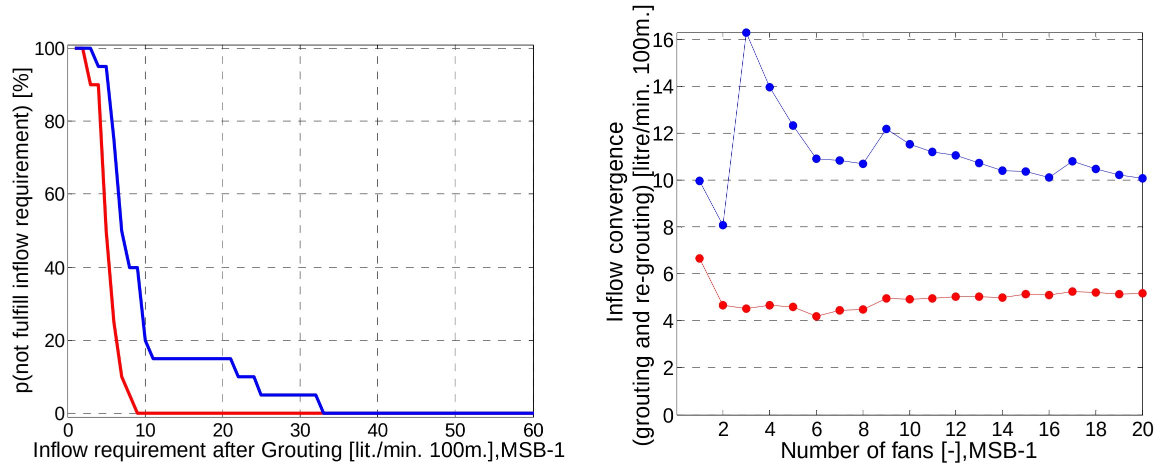


Figure 5-49 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSB-1 (left). Inflow convergence after n number of fans for MSB-1 (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-50. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level, grouting reaches a high sealing effect and re-grouting a much lower one. On the fan level the average value of all joints belonging to a fan is shown. The sealing effect will then be consequently higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. The fan sealing effect after grouting for MSB-1 is much higher than for MSA-1, which is explained by a more open rock mass.

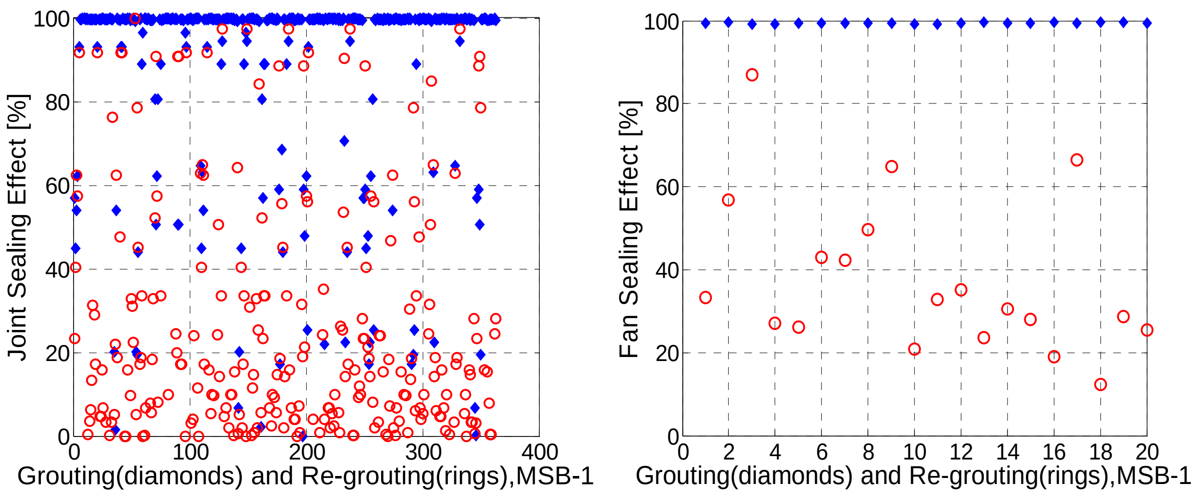


Figure 5-50 Joint and fan sealing effect after grouting and re-grouting for MSB-1.

In Figure 5-51 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 2000 seconds imply a high sealing effect close to 100 %, shorter grouting time imply lower sealing effect. For re-grouting a sealing effect of between 0 % and 90 % is achieved independent of the time taken.

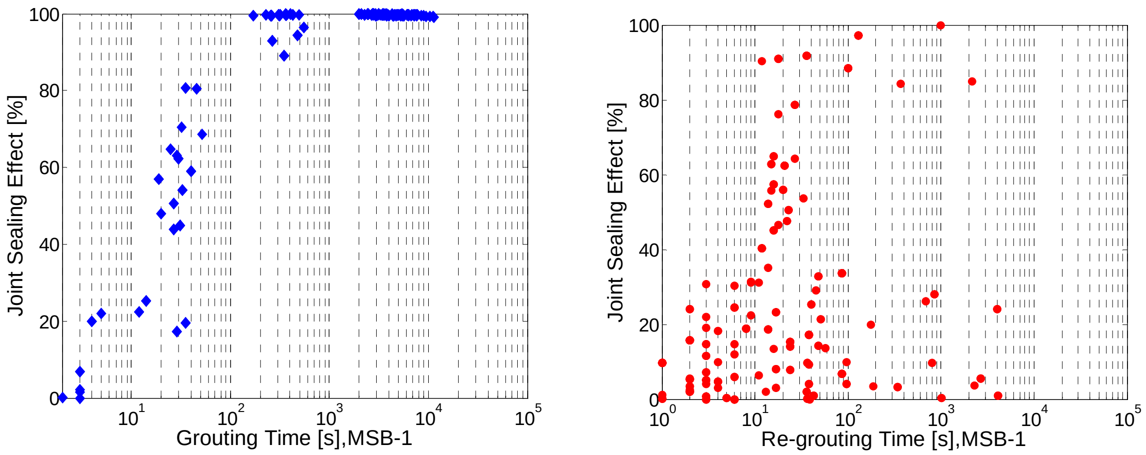


Figure 5-51 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-52 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 6000 seconds implies a high sealing effect of ~99.4 %. A very high sealing effect of ~99.6 % is achieved in the case of re-grouting, independent of the time taken.

As a comparison, the sealing effect for this method in rock mass A was between 80 % and 100 %. A more open rock mass implies a higher sealing effect. This accentuates that the sealing effect, as a parameter, needs to be related to the initial rock mass conductivity.

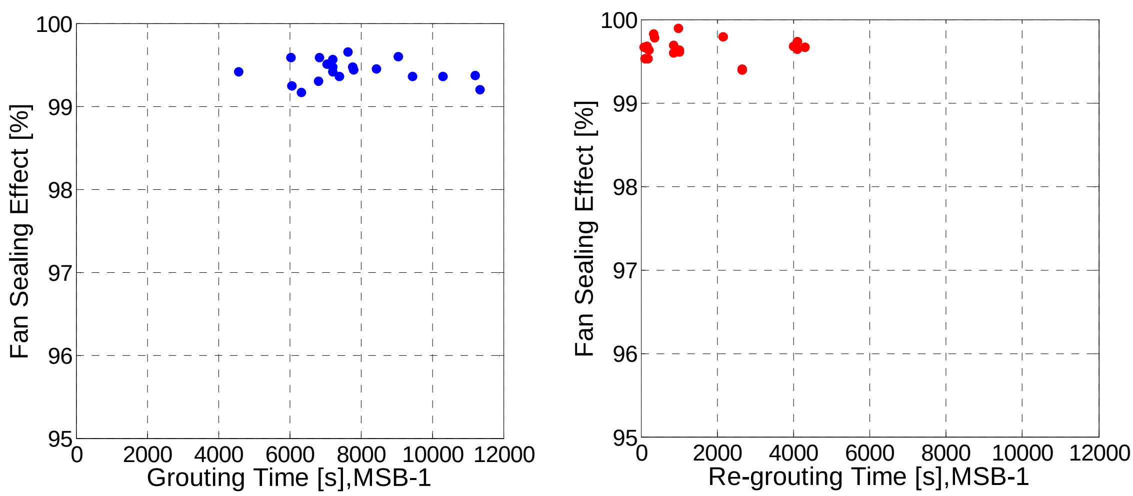


Figure 5-52 Accumulated sealing effect on a fan level after grouting and re-grouting.

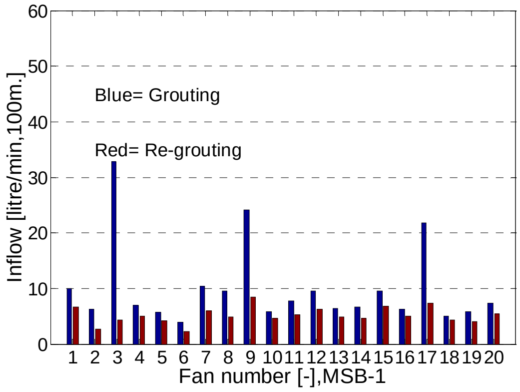


Figure 5-53 Inflow to the tunnel from the individual grout fans.

In Figure 5-53 the inflow from the individual fans after grouting and re-grouting is shown. In fan number 3, 9 and 17 the inflow after grouting is higher than average. After re-grouting the fans with higher inflow have achieved an inflow equal to the surrounding fans. For this method re-grouting is therefore necessary to reach an even sealing level for all fans. An alternative could also be one grouting round, but with decreased hole spacing.

If the inflow requirement of 5 litres/min/100 meters is applied on a fan length of 20 meters and substituted to 1 litre/min/20 meters then in 19 of 20 fans, the inflow after grouting is above the requirement. In 10 of 20 of the holes requiring re-grouting, the inflow is above that required even after re-grouting. To reach the inflow requirement, this method requires a third grouting round or more. It may also be necessary to change method if the inflow requirement should be reached.

The calculation of the flow for the different fans gave re-grouting in 19 of 20 fans, which could be compared with the re-grouting criteria for the control holes that only gave 6 re-groutings. To compare them is difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-54 the grouting time distribution for the fans and the calculated inflow after grouting for MSB-1 are shown.

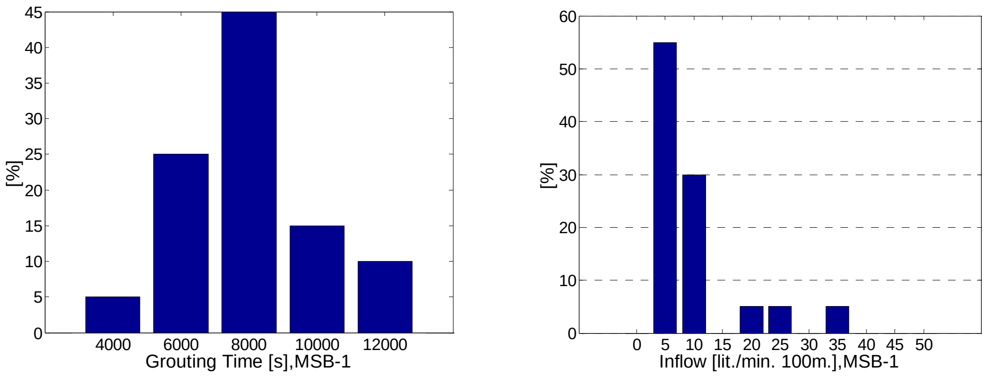


Figure 5-54 Fan grouting time- and fan inflow distribution for method MSB-1.

The average grouting time for a fan was calculated to 7789 seconds (2 hour and 9 minutes), with a standard deviation of 1743 seconds (29 min.), 45 % of the fans had a grouting time close to 8000 seconds.. The average inflow after grouting was calculated to 10.1 litres/min/100 meters, with a standard deviation of 7.4 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 51 m^3 or as an average to 2.6 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-8. As shown the sealing effect is already high after grouting. A small increase is received after the re-grouting based on control holes and a further small increase is achieved after a full re-grouting. It is not possible to judge the usefulness of the control holes, based on these results.

Table 5-8 Properties on a fan level of MSB-1.

	After grouting	**After grouting + control holes re-grouting**	**After re-grouting**
Fan Sealing Effect* [%]	μ=99.4 σ=0.1	μ=99.5 σ=0.2	μ=99.6 σ=0.1
Fan Time [s]	μ=7789 σ=1744	μ=8029 σ=2126	μ=9307 σ=2290

*Average sealing effect for 20 fans.

5.4.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grout time to perform the grouting of 20 fans with method MS-1 in rock mass B, was calculated to 155 778 seconds. To perform re-grouting for all fans was calculated to 30 369 seconds, which is about 20 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-9.

Table 5-9 Properties on a tunnel level of MSB-1.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After re-grouting**
Inflow to tunnel [l/min, 20 fans]	6 849	40	27	21
Sealing effect for a tunnel* [%]		99.4	99.6	99.7
Time for a tunnel [s]		155778	160584	186147
Number of fans above 5 litres/min/100 meters	20	19	18	10

*based on total inflow and total reduction for 20 fans.

After grouting the method MSB-1 in rock mass B fulfils an inflow requirement of 10 litres/min/100 meters and after full re-grouting an inflow requirement of 5.25 litres/min/100 meters is achieved.

5.5 Result Method MS, Rock mass B (MSB)

5.5.1 Comparison on joint level

The grouting method, MS, has been applied to rock mass B. The basis for the evaluation is the calculation of each possible joint in the interval 0-1000 µm, which gives 400 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-55. As shown, both the time and the volume are dependent on the joint aperture.

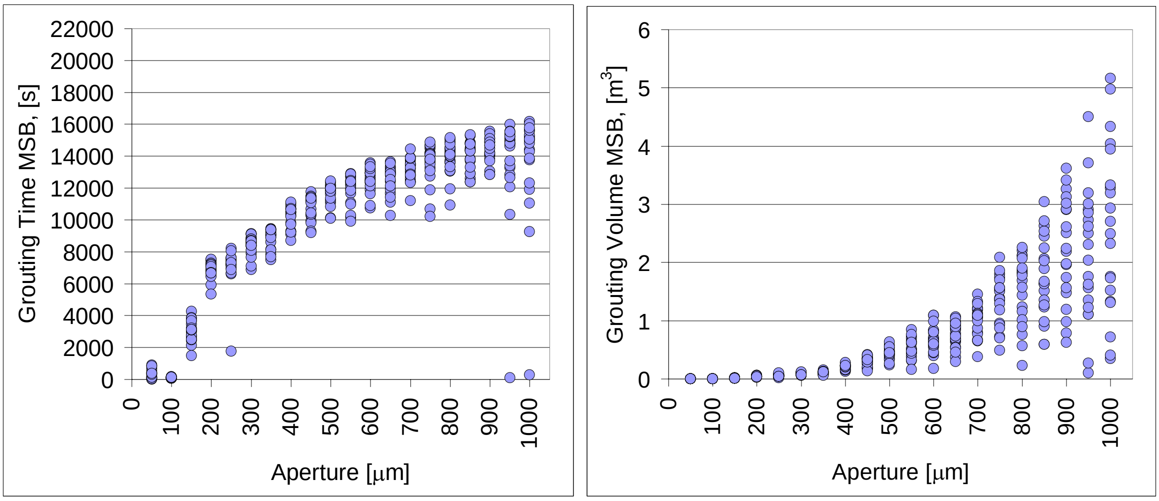


Figure 5-55 Grouting time and grouting volume for different apertures MSB.

All the joints have also been re-grouted, as shown in Figure 5-56. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting time is not increased by a larger aperture, which could be because the initial grouting already sealed most of the wider joints to a high level. The variance, within an aperture multiple, is higher for the re-grouting than for the grouting, which could be because the joints are not fully reached by the hole spacing chosen for the grouting.

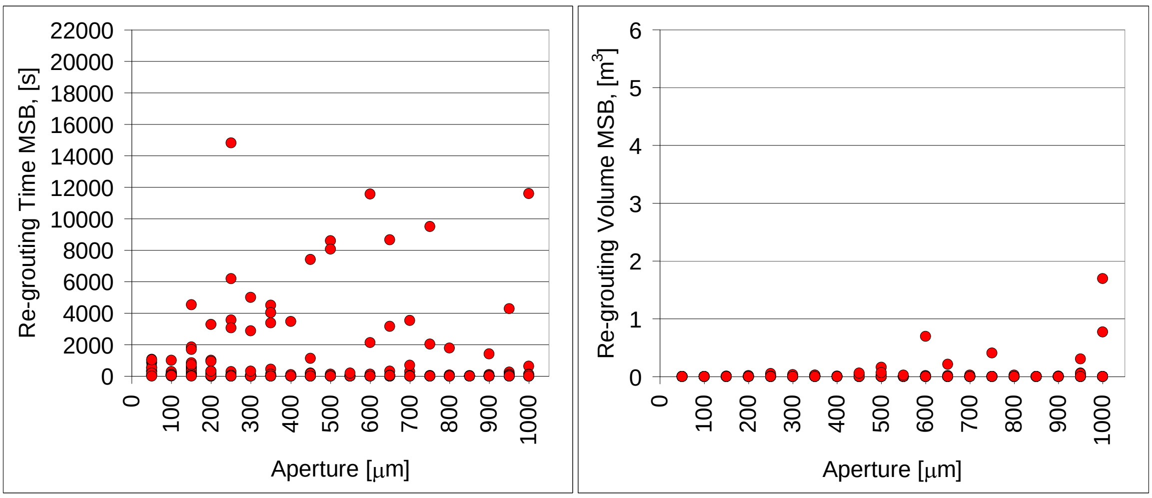


Figure 5-56 Re-grouting time and re-grouting volume for different apertures MSB.

If instead the re-grouting is plotted on a logarithmic scale, the difference between small values could be more clearly noted, as shown in Figure 5-57. From both the time and the volume it is noted that most of the fine joints are sealed during the re-grouting and that fewer of the larger joints are sealed during the re-grouting. It should be noted that out of 20 joints for each aperture, only a few large joints could be noted on the logarithmic scale, as values equal to zero are not plotted.

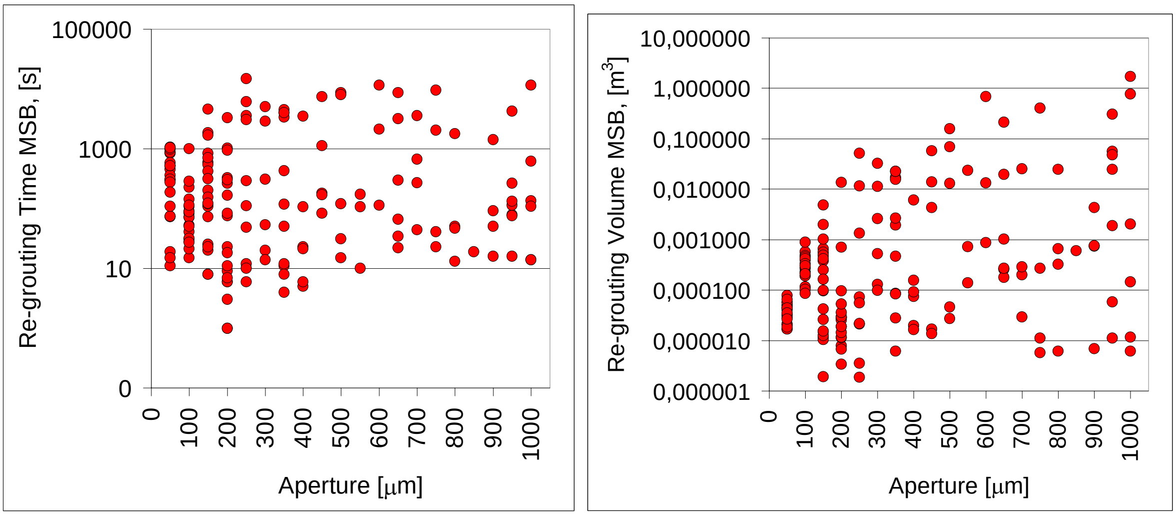


Figure 5-57 Re-grouting time and re-grouting volume for different apertures MSB on a logarithmic scale (values equal to zero can not be plotted).

The sealing effect, as shown in Figure 5-58, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is high for apertures larger than 200 μm, but low for smaller apertures. For joints with an aperture of 50 μm, the sealing effect is in the range of 10 %, which is a little higher than for rock mass A and C.

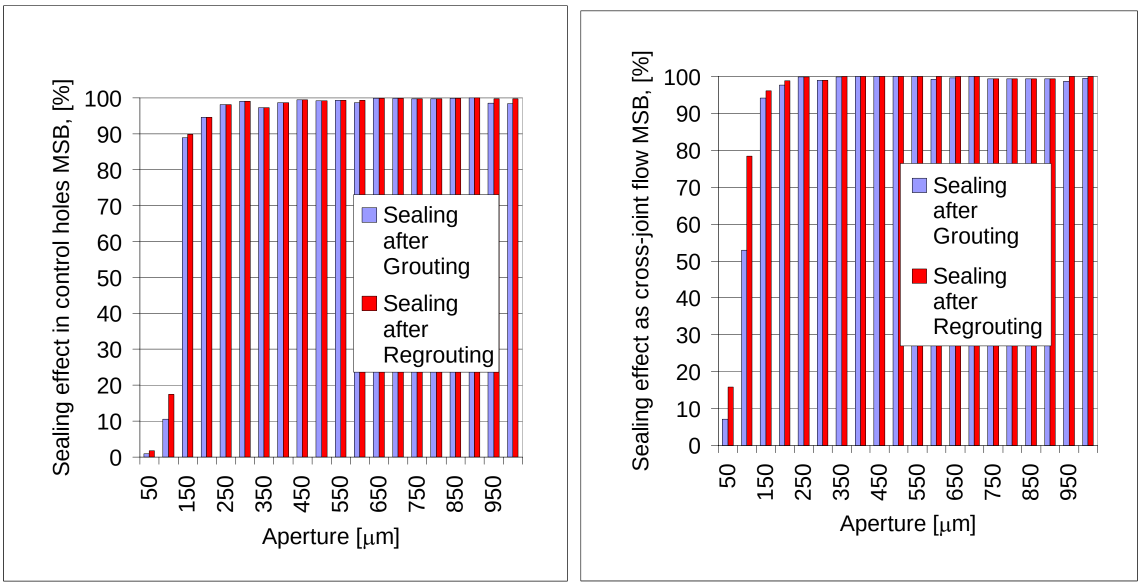


Figure 5-58 Sealing effect for different apertures MSB, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.5.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

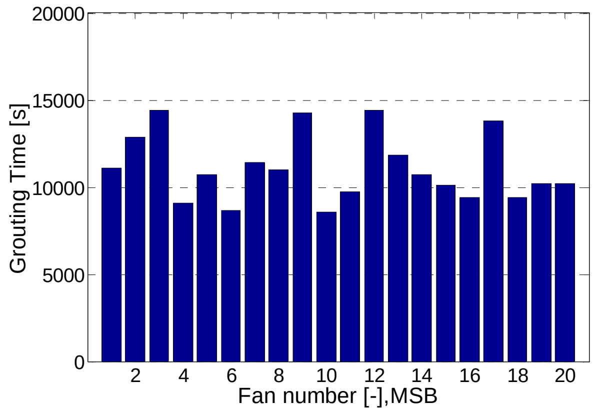


Figure 5-59 Grouting time for the individual grout fans (MSB).

In Figure 5-59 the grouting time for the individual fans are shown. The average grouting time was 11098 seconds, with a standard deviation of 1921 seconds. The grouting time is more evenly distributed than for MSA, which is because of a higher joint intensity in rock mass B. The average time is higher and the standard deviation is lower than in rock mass A, which also is because of a higher joint intensity. In rock mass A it was possible to get fans with just a few narrow apertures, which resulted in 20 % of the fans had a grouting time close to zero. As shown in Figure 5-59 none of the fans have a grouting time close to zero in rock mass B.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-60 the inflow for method MSB is shown. From the figure a large difference before and after grouting could be noticed. Between first and second grouting the flow was further reduced, but by a much lesser magnitude.

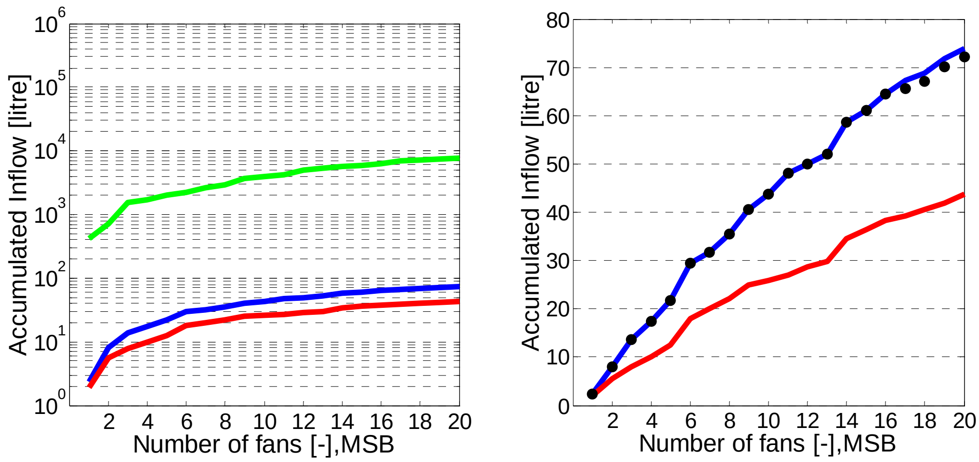


Figure 5-60 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

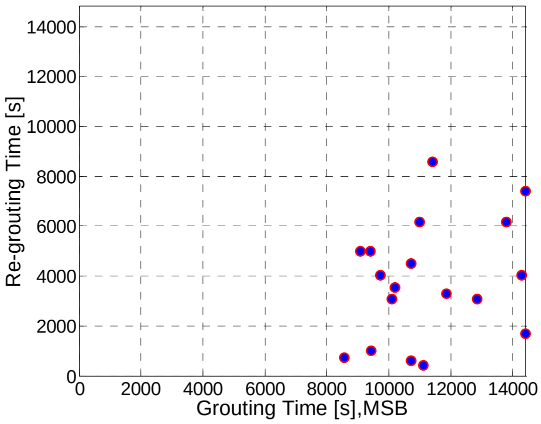


Figure 5-61 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-61 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and the re-grouting time for MSB.

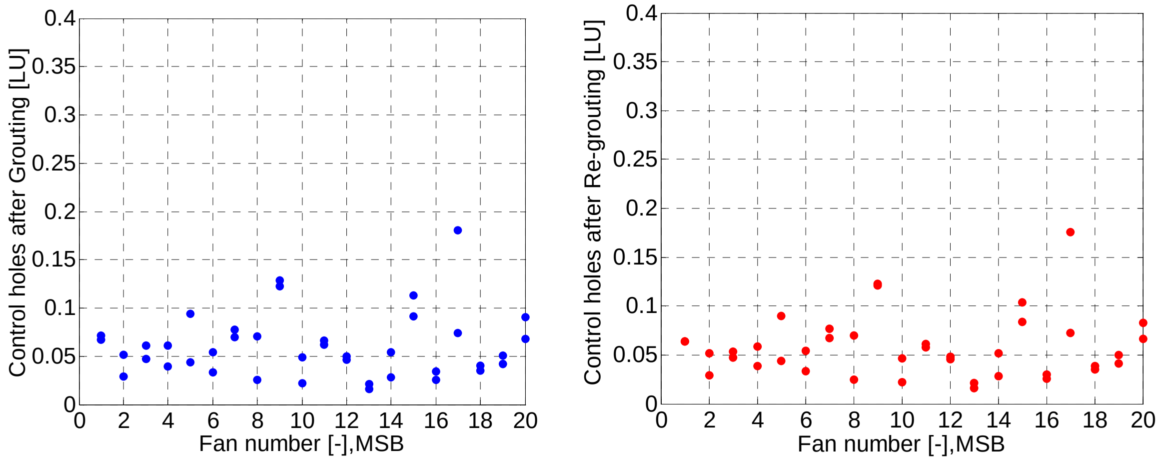


Figure 5-62 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 1 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-62 two control holes for each fan half after grouting and after re-grouting are shown. One of the control holes have a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-48 resulted in 1 re-grouting. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. For grout fan 17 the re-grouting time is 6176 seconds.

On the other hand, if re-grouting is carried out and tested, 1 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-63 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting is below 7 litres/min/100 meters for method MSB, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow requirement is above 38 litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-63 it is difficult to decide if the inflow convergence stabilizes or not for both the grouting and the re-grouting of MSB.

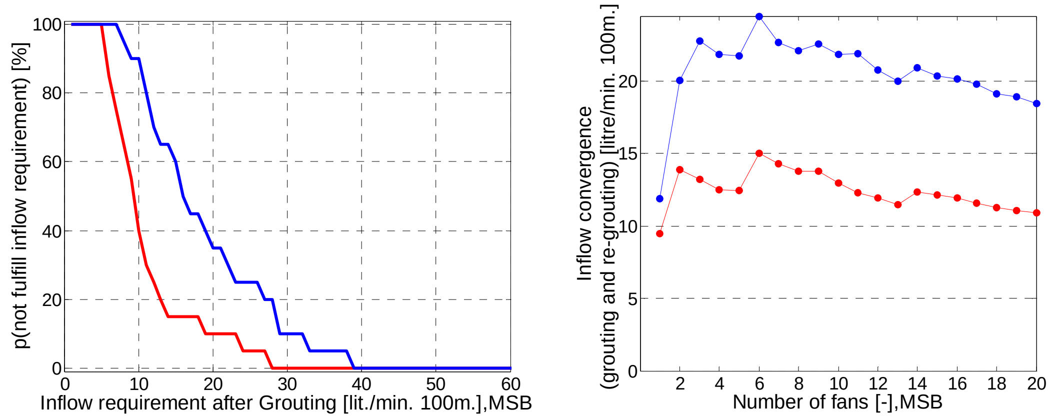


Figure 5-63 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSB (left). Inflow convergence after n number of fans for MSB (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-64. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On a joint level, grouting reaches a high sealing effect and re-grouting a much lower. On a fan level the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. The fan sealing effect after grouting for MSB is much higher than for MSA, which is explained by a more open rock mass.

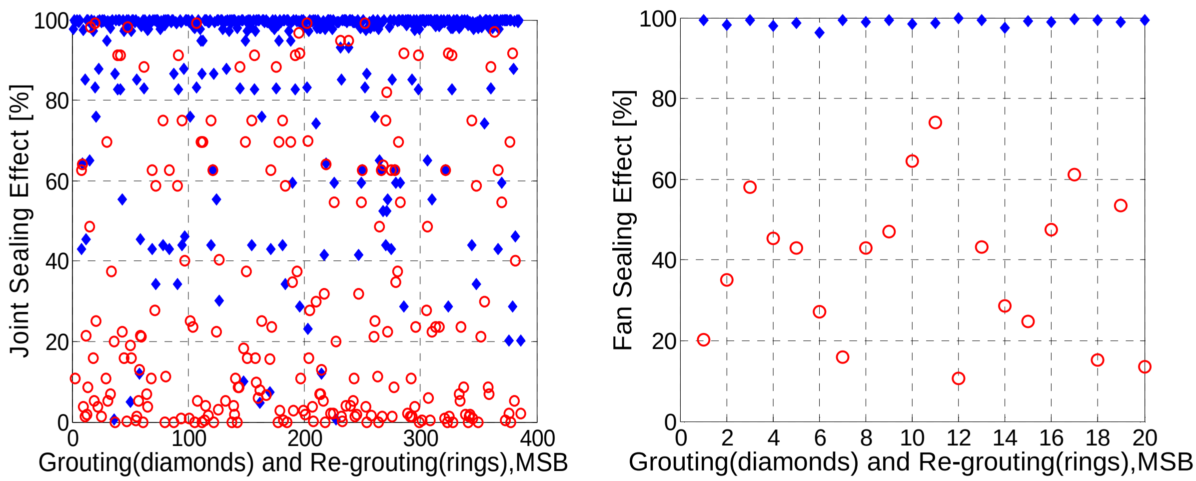


Figure 5-64 Joint and fan sealing effect after grouting and re-grouting for MSB.

In Figure 5-65 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 2000 seconds imply a high sealing effect between 90 % and 100 %, shorter grouting time imply lower sealing effect. For re-grouting a sealing effect of between 0 % and 99 % is achieved independent of the time taken.

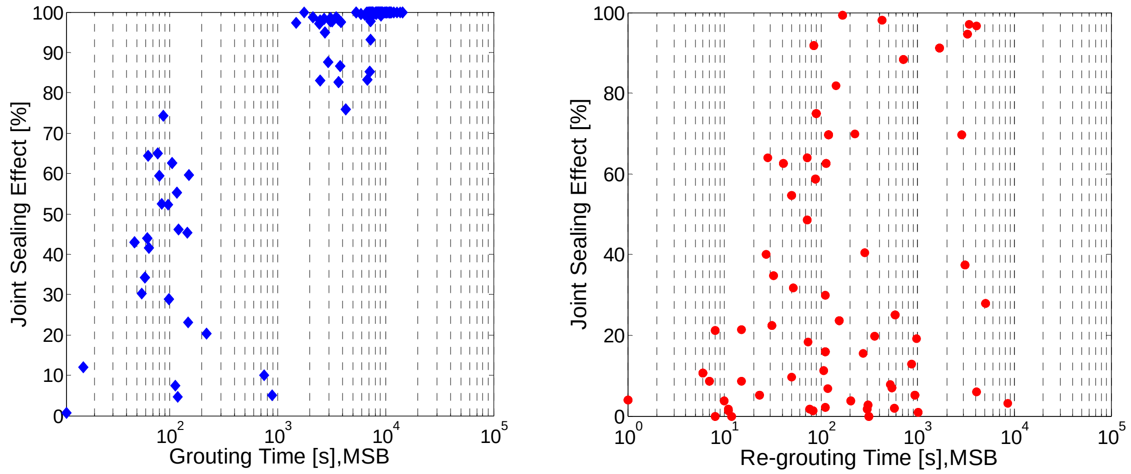


Figure 5-65 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-66 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 8000 seconds imply a sealing effect of around 99 %. An increased sealing effect is achieved in the case of re-grouting, independent of the time taken.

Compared to method MSB-1 the sealing effect for this method is lower, which may be explained by an extended use of grout mixes with a higher viscosity.

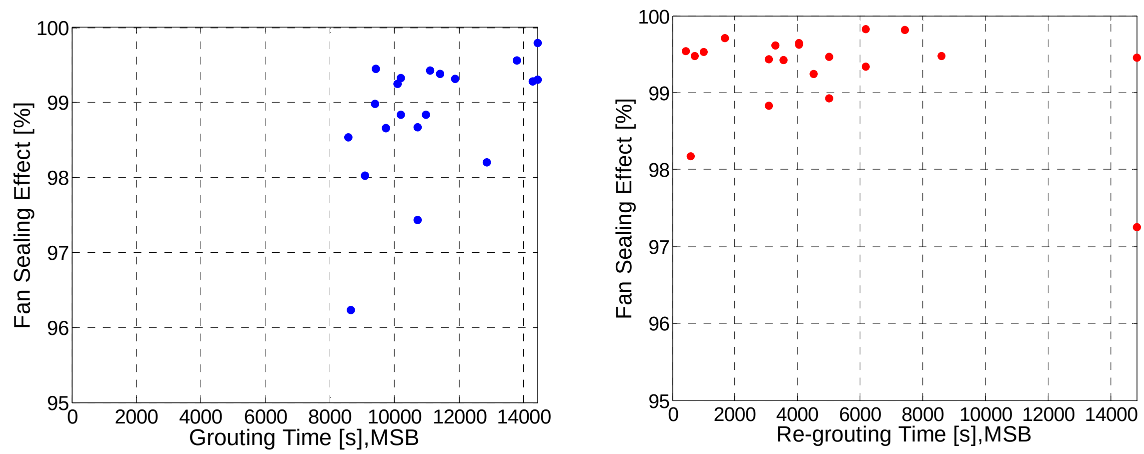


Figure 5-66 Accumulated sealing effect on a fan level after grouting and re-grouting.

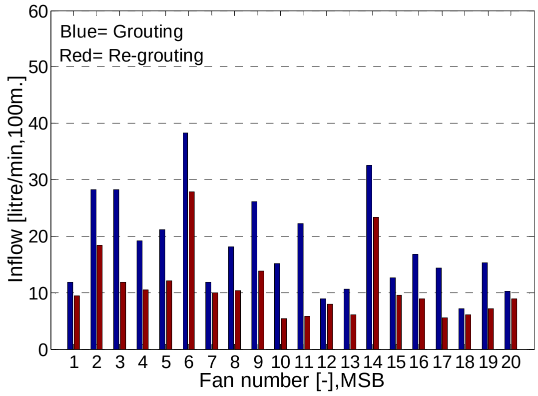


Figure 5-67 Inflow to the tunnel from the individual grout fans.

In Figure 5-67 the inflow from the individual fans after grouting and re-grouting is shown. Both after grouting and re-grouting the inflow variance is quite high, which indicates that a third grouting round with the same method may be useful. An alternative could also be to decrease the hole spacing.

If the inflow requirement of 5 litres/min/100 meters is applied on a fan length of 20 meter and substituted to 1 litres/min/20 meters, then the inflow after both grouting and re-grouting are above the requirement for all fans. If the sealing level is set to 5 litres/min/100 meters, this method requires a third grouting round or more. It may also be necessary to change the method if the sealing level is to be reached.

The calculation of the flow for the different fans gave re-grouting in all fans, which could be compared with the re-grouting criteria for the control holes that only gave one re-grouting. To compare them is difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-68 the grouting time distribution for the fans and the calculated inflow after grouting are shown.

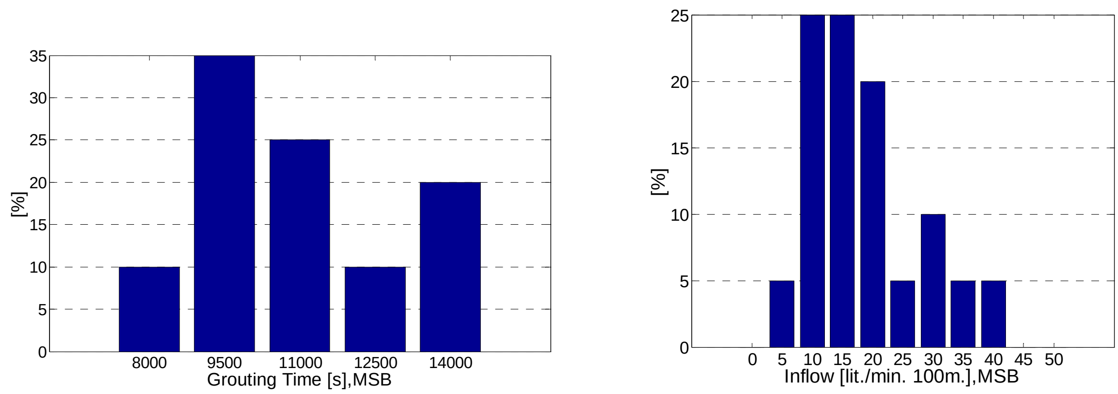


Figure 5-68 Fan grouting time- and fan inflow distribution for method MSB.

The average grouting time for a fan was calculated to 11 098 seconds (3 hour and 5 minutes), with a standard deviation of 1921 seconds (32 min.), 35 % of the fans had a grouting time close to 9600 seconds.. The average inflow after grouting was calculated to 18.5 litres/min/100 meters, with a standard deviation of 8.5 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 59 m^3 or as an average to 2.9 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-10. As shown the sealing effect is already high after grouting. No increase is received after re-grouting based on the control holes and a small increase is received after a full re-grouting. It is not possible to judge about the usefulness of the control holes, based on these results.

Table 5-10 Properties on a fan level of MSB.

	After grouting	After grouting + control holes re-grouting	After re-grouting
Sealing Effect for a fan* [%]	μ=98.8 σ=0.8	μ=98.8 σ=0.9	μ=99.3 σ=0.6
Time for a fan [s]	μ=11 098 σ=1921	μ=11 406 σ=2710	μ=15 996 σ=4310

*Average sealing effect for 20 fans.

5.5.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grout time to perform the grouting of 20 fans with method MS in rock mass B was calculated to 221 952 seconds. To perform re-grouting for all fans was calculated to 97 966 seconds, which is about 45 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-11.

Table 5-11 Properties on a tunnel level of MSB.

	Before	After grouting	After grouting + control holes re-grouting	After re-grouting
Inflow to tunnel [l/min, 20 fans]	7782	74	72	44
Sealing effect for a tunnel* [%]		99.0	99.1	99.4
Time for a tunnel [s]		155778	160584	186147
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MS in rock mass B fulfils an inflow requirement of 18.5 litres/min/100 meters and after full re-grouting an inflow requirement of 11.0 litres/min/100 meters is achieved.

5.6 Result Method MA, Rock mass B (MAB)

5.6.1 Comparison on joint level

The grouting method, MA, has been applied to rock mass B. The basis for the evaluation is the calculation of each possible joint in the interval 0-1000 µm, which gives 400 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-69. As shown, both the time and the volume are dependent on the joint aperture.

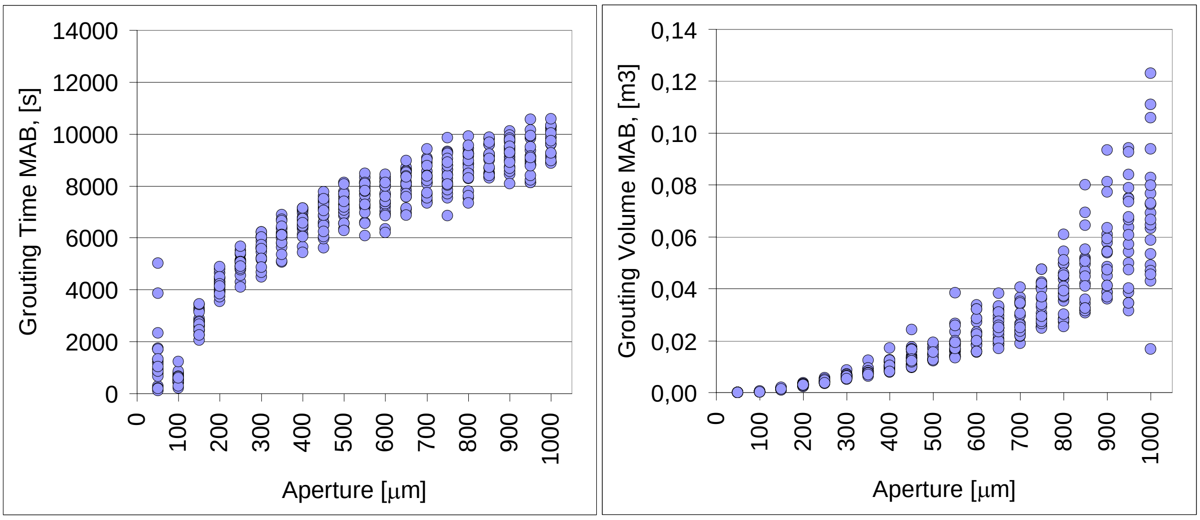


Figure 5-69 Grouting time and grouting volume for different apertures MAB.

All the joints have also been re-grouted, as shown in Figure 5-70. For all joints and apertures the re-grouting time is shorter than the grouting time, but not significantly shorter as for method MS and MS-1. The re-grouting time and volume is similar to the grouting time and volume. This is an indication that the grouting method does not, after grouting, result in a grout spread that affects the area of the re-grouting holes. The accumulated distance between grout holes was 1.8 meters after grouting and 0.9 meters after re-grouting.

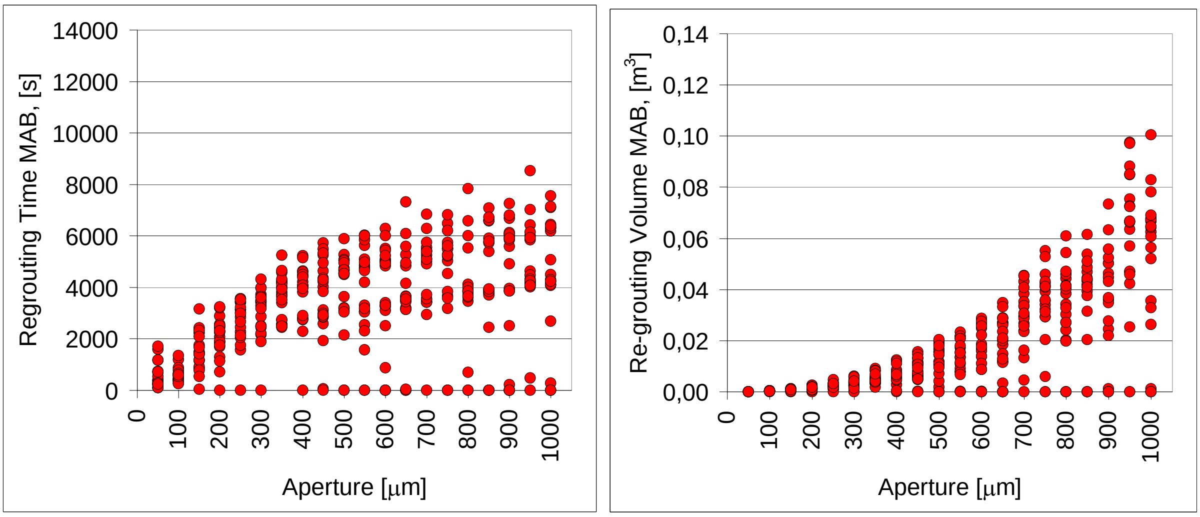


Figure 5-70 Re-grouting time and re-grouting volume for different apertures MAB.

The sealing effect, as shown in Figure 5-71, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect after grouting is around 70 % and after re-grouting around 90 %, which is regarded as very low.

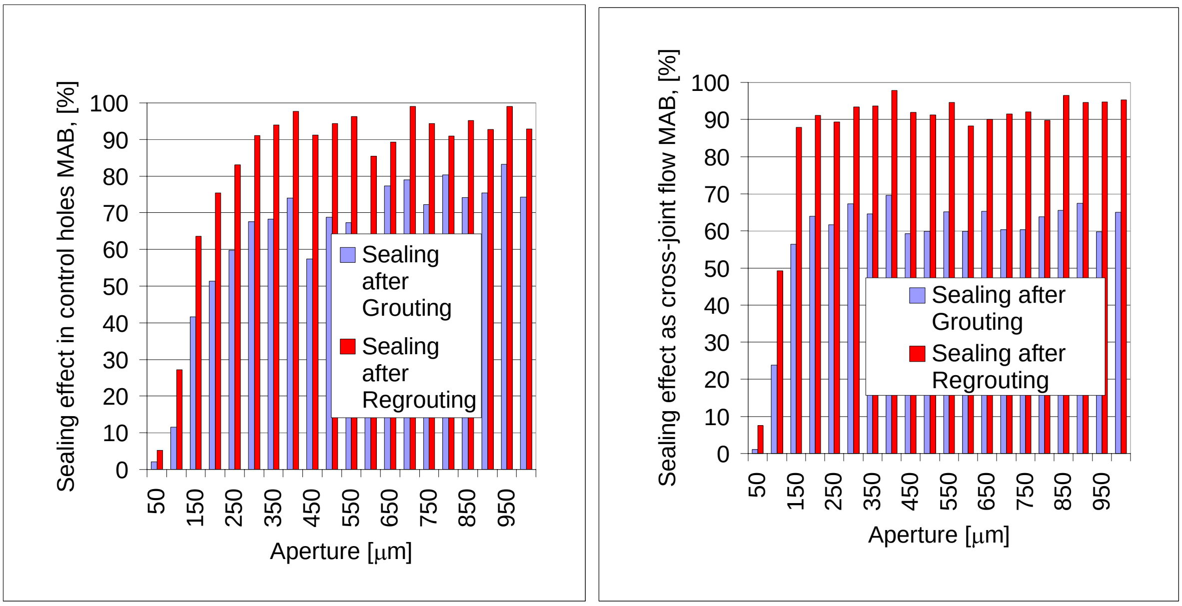


Figure 5-71 Sealing effect for different apertures MAB, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.6.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

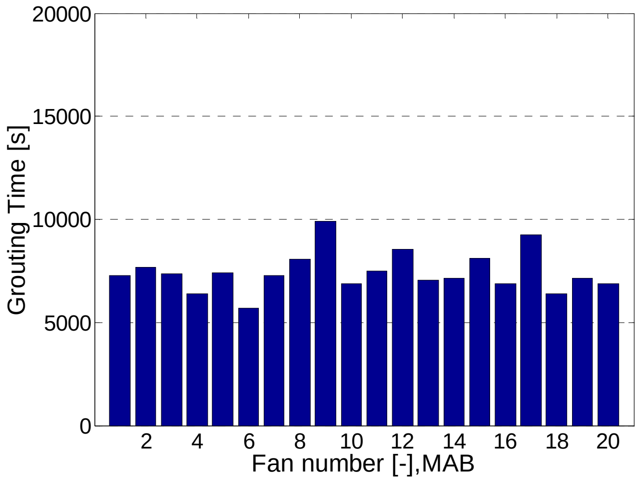


Figure 5-72 Grouting time for the individual grout fans (MAB).

In Figure 5-72 the grouting time for the individual fans are shown. The average grouting time was 2883 seconds, with a standard deviation of 1819 seconds. The grouting time is higher than in rock mass A, but the standard deviation is similar. A more jointed rock mass could be expected to give a more evenly distributed grouting time, which was not noted when comparing MAA and MAB.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-73 the inflow for method MA in rock mass B is shown. From the figure the large difference before and after grouting could be noticed, but still the inflow after grouting and re-grouting is very high, which is due to a very conductive rock mass (hydraulic conductivity of 1.83 $*10^{-5}$). Between grouting and re-grouting the flow was further reduced.

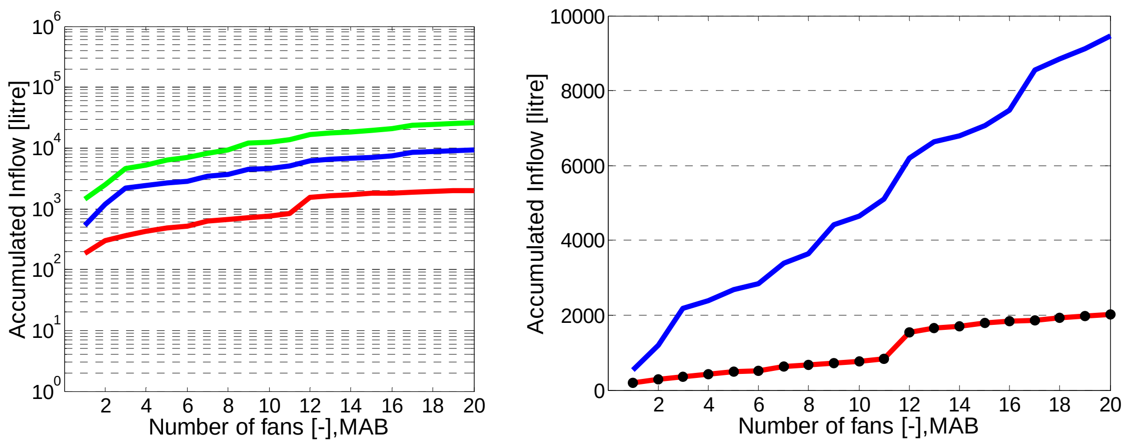


Figure 5-73 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

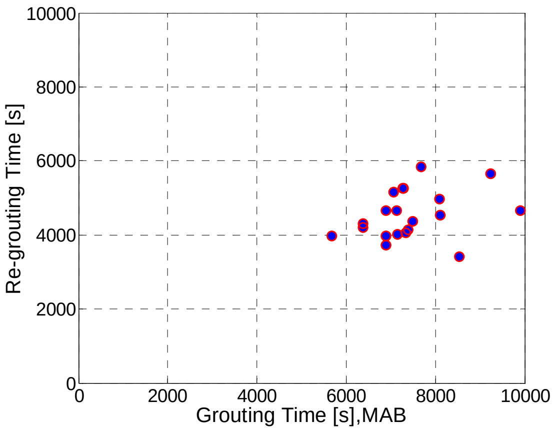


Figure 5-74 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-74 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and the re-grouting time for MAB.

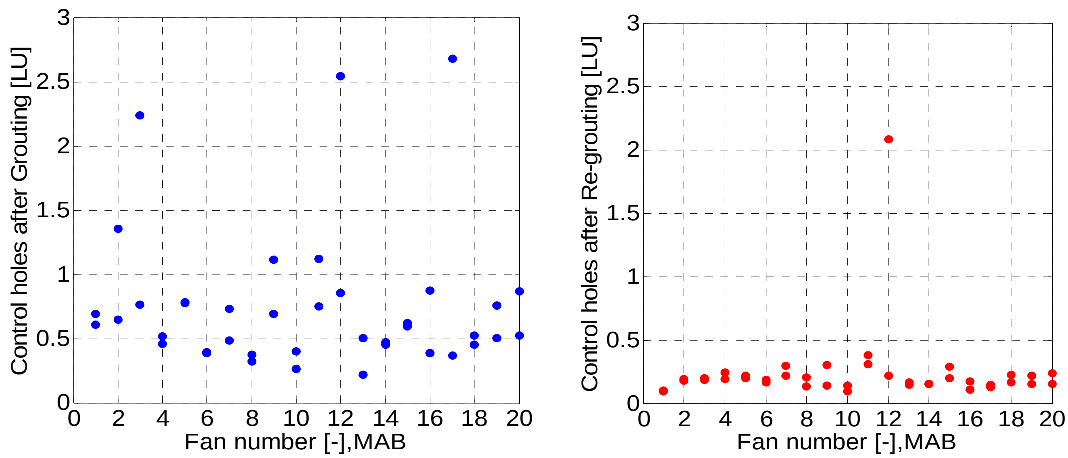


Figure 5-75 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 40 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-75 two control holes for each fan half after grouting and after re-grouting are shown. All of the control holes have a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-75 resulted in 20 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. For grout fans 1 to 20 the re-grouting time is 90 793 seconds.

On the other hand, if re-grouting is carried out and tested, 17 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-76 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting is below 764 litres/min/100 meters for method MAB, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 5400 litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-76 the inflow convergence stabilizes after about 6 fans for the grouting of MAB and after about 3 fans for the re-grouting.

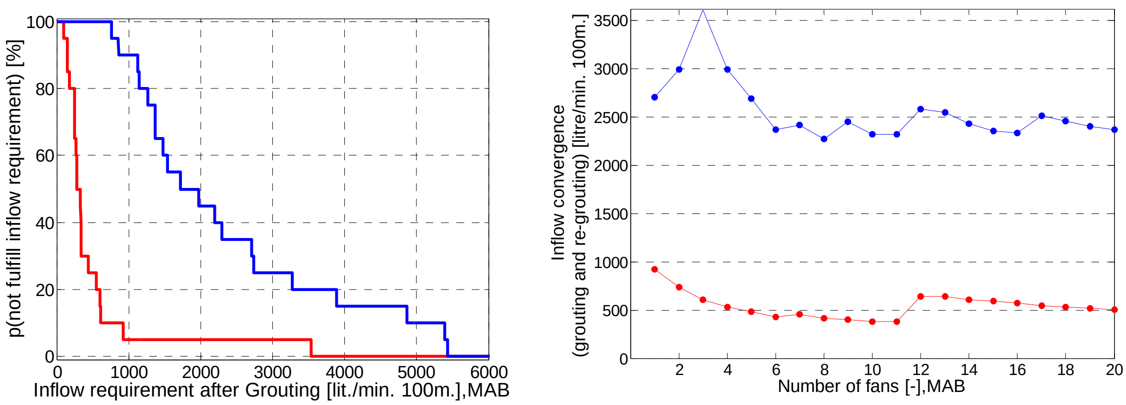


Figure 5-76 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MAB (left). Inflow convergence after n number of fans for MAB (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-77. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level re-grouting reaches a high sealing effect and grouting a much lower, which is the opposite to methods MS and MS-1. On a fan level, the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. The fan sealing effect after grouting of MAB is higher than for MAA, which is explained by a more open rock mass.

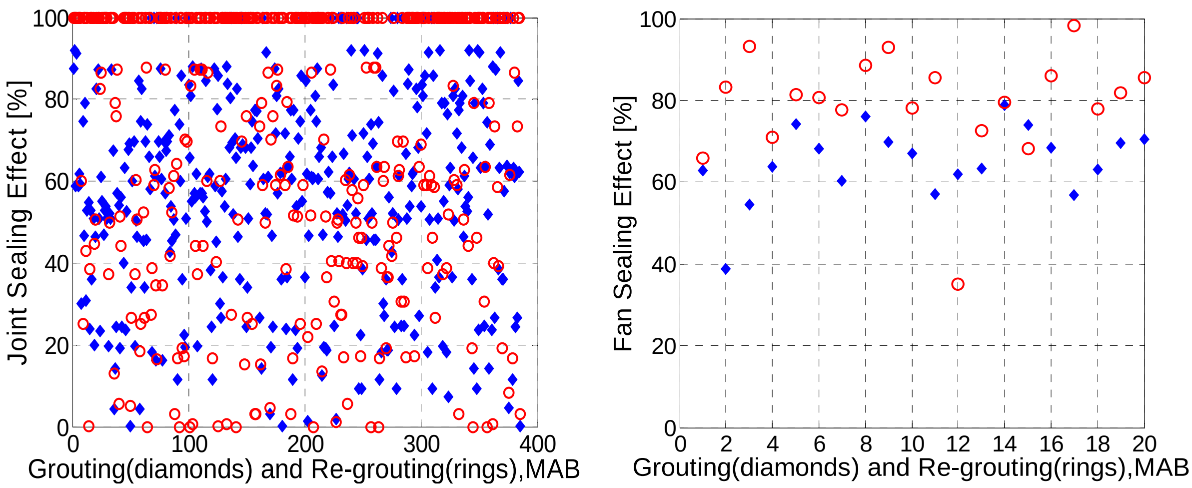


Figure 5-77 Joint and fan sealing effect after grouting and re-grouting for MAB.

In Figure 5-78 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 4000 seconds implies a sealing effect between 20 % and 100 %, shorter grouting time implies lower sealing effect. Re-grouting performed more than 700 seconds implies both a high and a low sealing effect; the shorter time implying lower sealing effect.

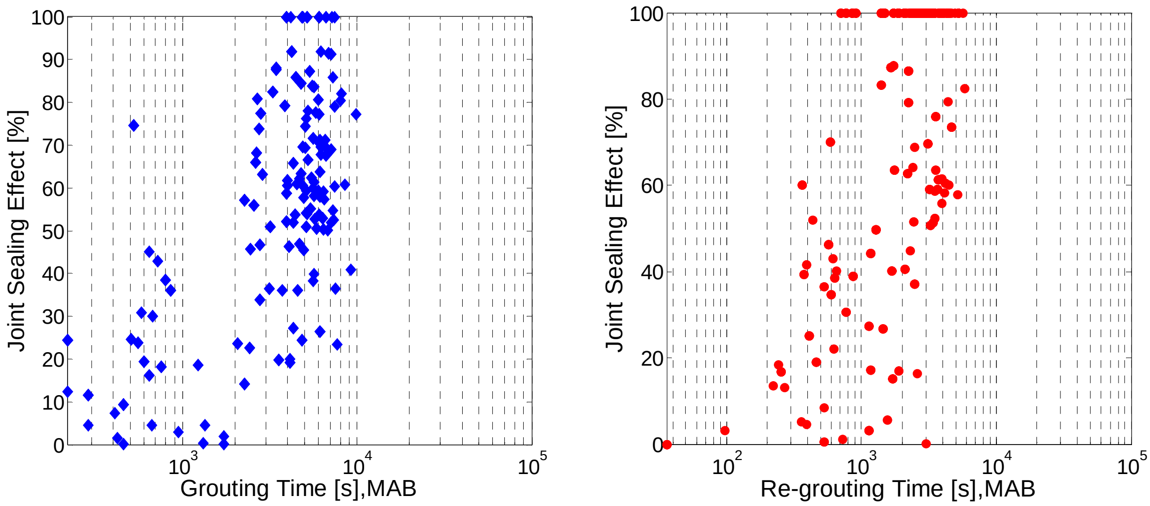


Figure 5-78 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-79 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 7000 seconds implies a sealing effect between 50 % and 80 %. Re-grouting for more than 4000 seconds implies an increased sealing effect to around 90 %.

Compared to method MSB-1 and MSB the sealing effect for this method is much lower, which may be explained by e.g. the extended use of grout mixes with a higher viscosity and a wider hole spacing.

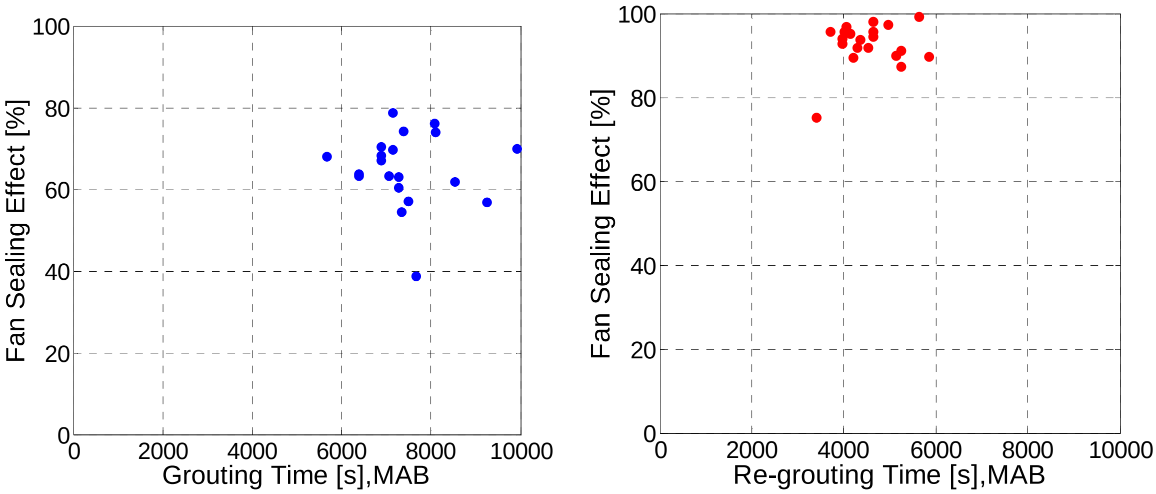


Figure 5-79 Accumulated sealing effect on a fan level after grouting and re-grouting.

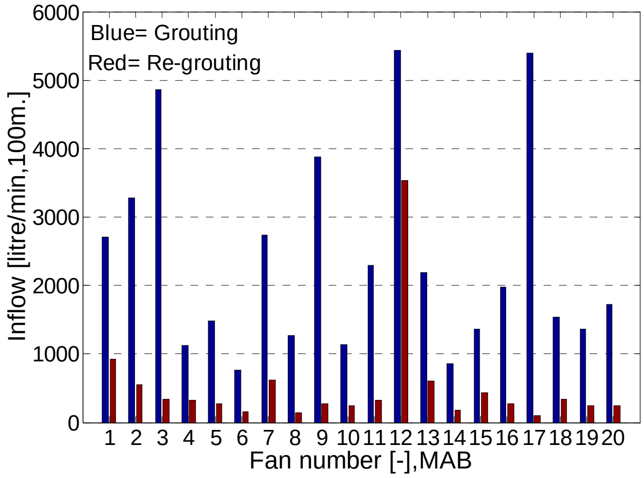


Figure 5-80 Inflow to the tunnel from the individual grout fans.

In Figure 5-80 the inflow from the individual fans after grouting and re-grouting is shown. In fan number 3, 12 and 17 the inflow after grouting and re-grouting is higher than average. Still after re-grouting some fans have high inflows, which indicate that the decreased hole spacing from cc 1.8 meters to cc 0.9 meters, is not enough for this method.

If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 20 of 20 fans both after grouting and re-grouting the inflow is above the requirement.

The calculation of the flow for the different fans gave re-grouting in 20 of 20 fans, which could be compared with the re-grouting criteria for the control holes that only gave 17 re-groutings. To compare them is difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-81 the grouting time distribution for the fans and the calculated inflow after grouting for MAB are shown.

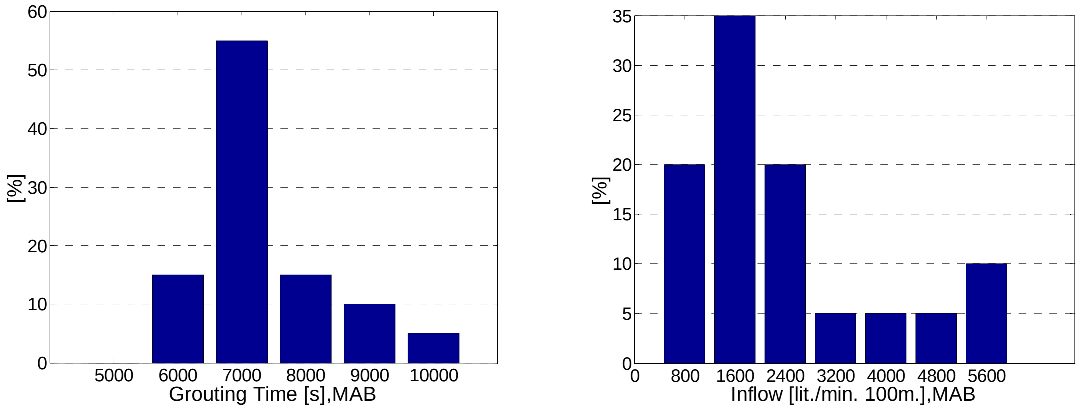


Figure 5-81 Fan grouting time- and fan inflow distribution for method MAB

The average grouting time for a fan was calculated to 7439 seconds (2 hour and 3 minutes), with a standard deviation of 977 seconds (32 min.), 55 % of the fans had a grouting time close to 7000 seconds. The average inflow after grouting was calculated to 2368 litres/min/100 meters, with a standard deviation of 1475 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 3.15 m^3 or as an average to 158 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-12. As shown the sealing effect is increased after re-grouting based on the control holes. The sealing effect is not further increased by a full re-grouting. The control holes, can for this situation, be regarded as a good parameter for judging the need for re-grouting.

Table 5-12 Properties on a fan level of MAB.

	After grouting	**After grouting + control holes re-grouting**	**After re-grouting**
Sealing Effect for a fan* [%]	μ=64.9 σ=9.0	μ=92.7 σ=5.2	μ=92.7 σ=5.2
Time for a fan [s]	μ=7439 σ=977	μ=11 979 σ=1326	μ=11 979 σ=1326

*Average sealing effect for 20 fans.

5.6.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grout time to perform the grouting of 20 fans with method MA in rock mass B was calculated to 148 749 seconds.

The average grouting time for a fan was calculated to 7439 seconds (2 hour and 3 minutes) per fan, with a standard deviation of 977 seconds (16 min.). To perform re-grouting for all fans was calculated to 90793 seconds, which is about 61 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-13.

Table 5-13 Properties on a tunnel level of MAB.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After re-grouting**
Inflow to tunnel [l/min, 20 fans]	26 207	9474	2025	2025
Sealing effect for a tunnel* [%]		63.8	92.3	92.3
Time for a tunnel [s]		148 789	239 582	239 582
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MAB fulfils an inflow requirement of 2368 litres/min/100 meters and after full re-grouting an inflow requirement of 506 litres/min/100 meters is achieved.

5.7 Result Method MS-1, Rock mass C (MSC-1)

5.7.1 Comparison on joint level

The grouting method, MS-1, has been applied to rock mass C. The basis for the evaluation is the calculation of each possible joint in the interval 0-1400 μm, which gives 560 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-82. As shown, both the time and the volume are dependent on the joint aperture. For apertures larger than 800 μm, the grouting time occasionally and increasingly shows lower values. The explanation is that for the larger apertures, the Lugeon value before grouting sometimes can be above 1.0 LU, which means that the start grout is changed from W/C 2.0 to a W/C 1.0 grout. The lower W/C ratio gives a higher viscosity and thereby a shorter grouting time. This effect was only rarely visible for rock mass B, due to a lower hydraulic pressure.

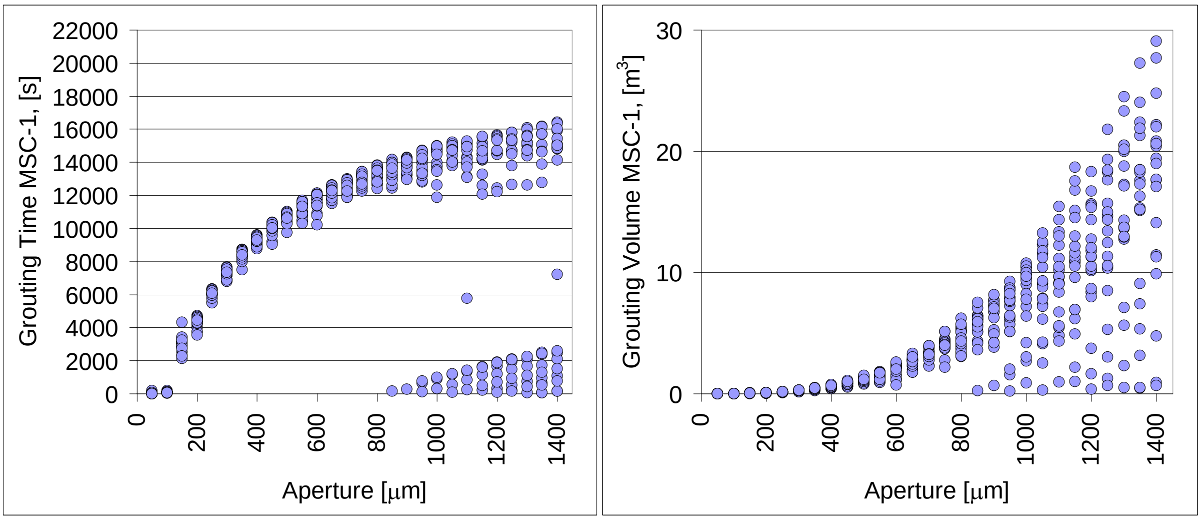


Figure 5-82 Grouting time and grouting volume for different apertures MSC-1.

All joints have also been re-grouted, as shown in Figure 5-83. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting time is not increased by a larger aperture for apertures up to around 600 μm, which could be because the initial grouting already sealed most of those joints to a high level. For the joints above 600 μm the re-grouting time was increased by larger aperture, which could be due to grout separation in wide joints which create conductive channels, and are then sealed by the re-grouting. The separation in the wide joints is higher due to the increased joint height and in this case also due to the choice of grout mix. During the initial grouting of the less conductive joints a grout mix with lower separation was used and for the more conductive joints a grout mix with higher separation was used.

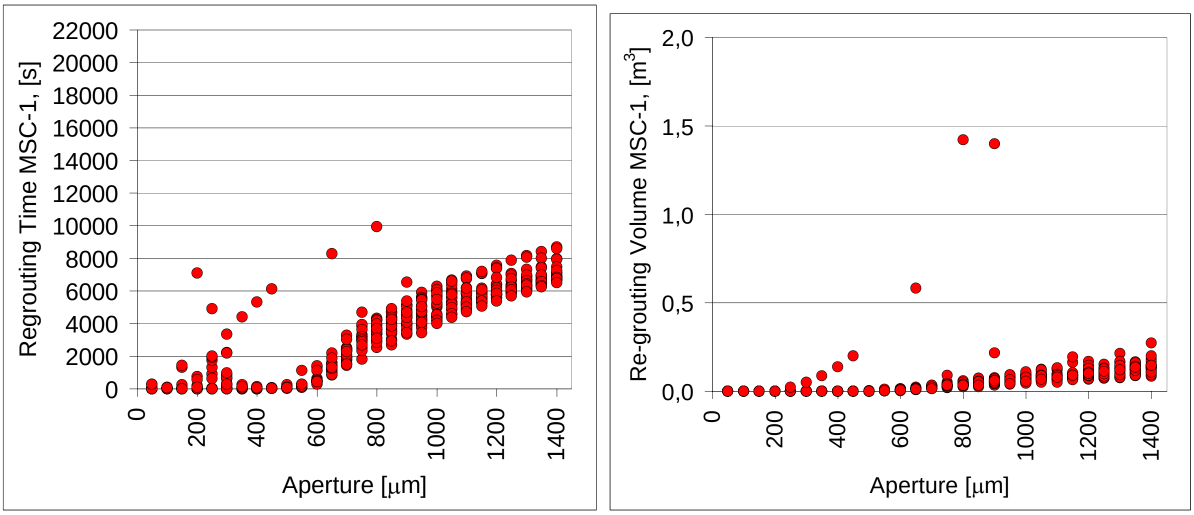


Figure 5-83 Re-grouting time and re-grouting volume for different apertures MSC-1.

The sealing effect, as shown in Figure 5-84, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect is in general higher when measured as cross joint flow rather than in control holes. The sealing effect after grouting is high for apertures larger than 150 µm, but low for smaller apertures. For joints with an aperture of 50 µm, the sealing times, as well as, the sealing effect were almost zero. A re-grouting improves all apertures, but for the apertures ranging between 150 µm and 500 µm the improvement by re-sealing is higher, which may be explained by the Lugeon based grout selection as discussed above.

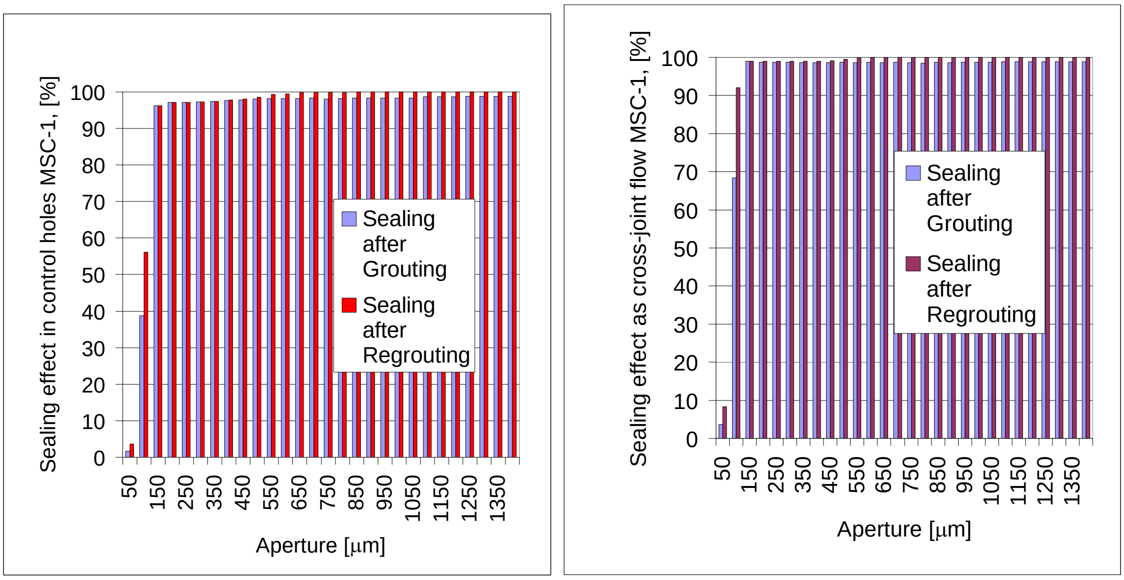


Figure 5-84 Sealing effect for different apertures MSC-1, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.7.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

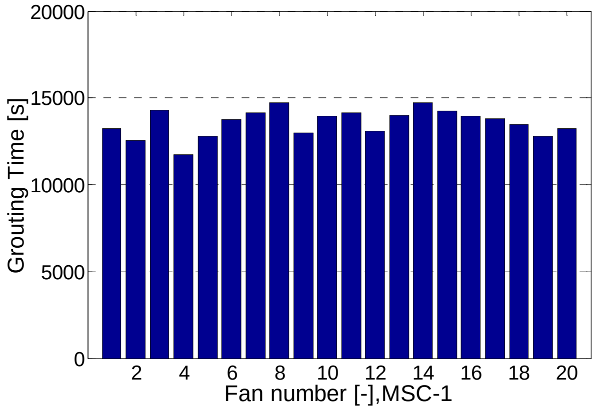


Figure 5-85 Grouting time for the individual grout fans (MSC-1).

In Figure 5-85 the grouting time for the individual fans are shown. The average grouting time was 13581 seconds, with a standard deviation of 774 seconds. The grouting time is more evenly distributed than for MSA-1 and MSB-1, which is because of a higher joint intensity in rock mass C. The average time is higher and the standard deviation is lower than in rock mass A and B, which also is because of a higher joint intensity. In rock mass A it was possible to get fans with just a few narrow apertures, which resulted in 25 % of the fans that had a grouting time close to zero. As shown in Figure 5-85 none of the fans have a grouting time close to zero.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-46 the inflow for method MS-1 in rock mass C is shown. From the figure, the large difference before and after grouting could be noticed. Between first and second grouting the flow was further reduced, but by a much lesser magnitude.

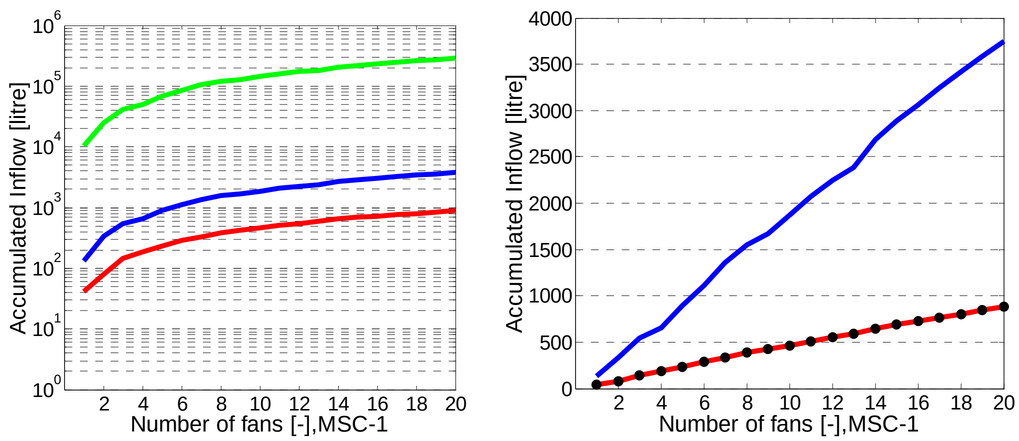


Figure 5-86 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

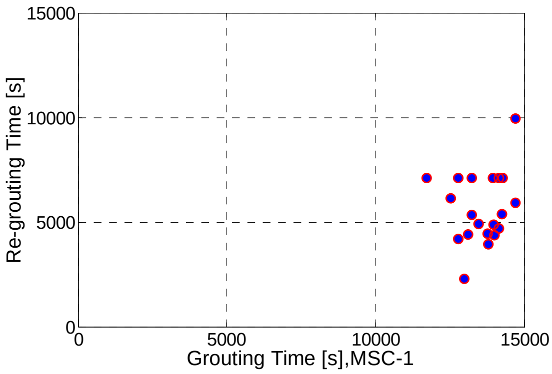


Figure 5-87 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-87 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and the re-grouting time for MSC-1.

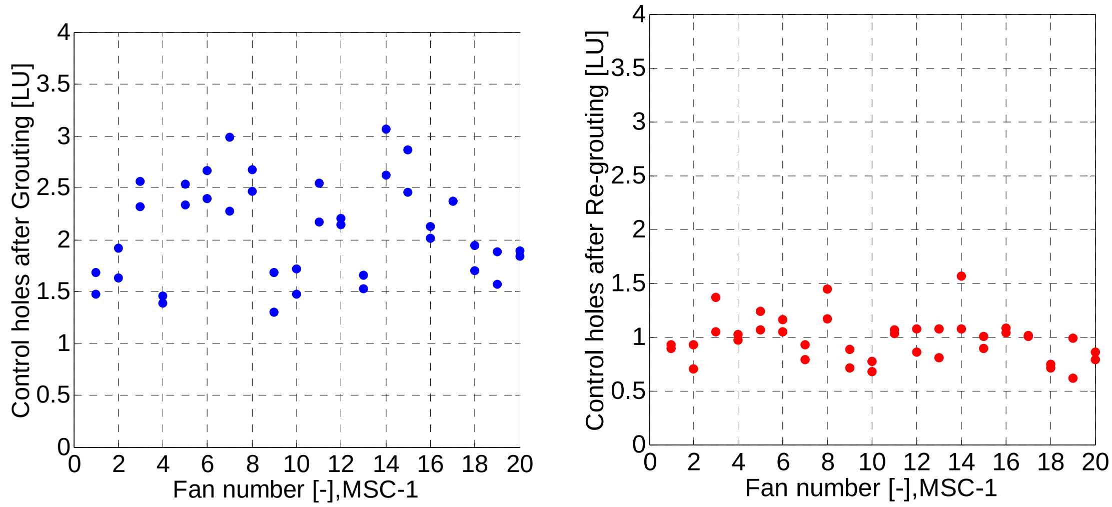


Figure 5-88 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 8 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-88 two control holes for each fan half after grouting and after re-grouting are shown. All of the control holes had a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-88 resulted in 20 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. The total re-grouting time is 113428 seconds.

On the other hand, if re-grouting is carried out and tested, 20 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-89 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting is below 600 litres/min/100 meters for method MSC-1, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 1500 litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-89 the inflow convergence stabilizes after about 6 fans for the grouting of MSC-1 and after about 3 fans for the re-grouting.

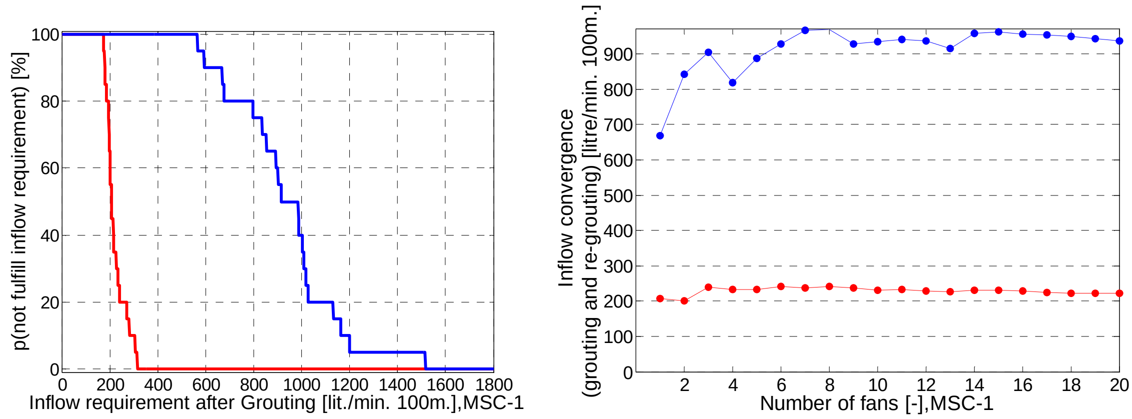


Figure 5-89 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSC-1 (left). Inflow convergence after n number of fans for MSC-1 (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-90. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level grouting reaches a high sealing effect and re-grouting a much lower effect. On the fan level the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. The fan sealing effect after re-grouting for MSC-1 is much higher than for MSB-1, which is explained by a more open rock mass.

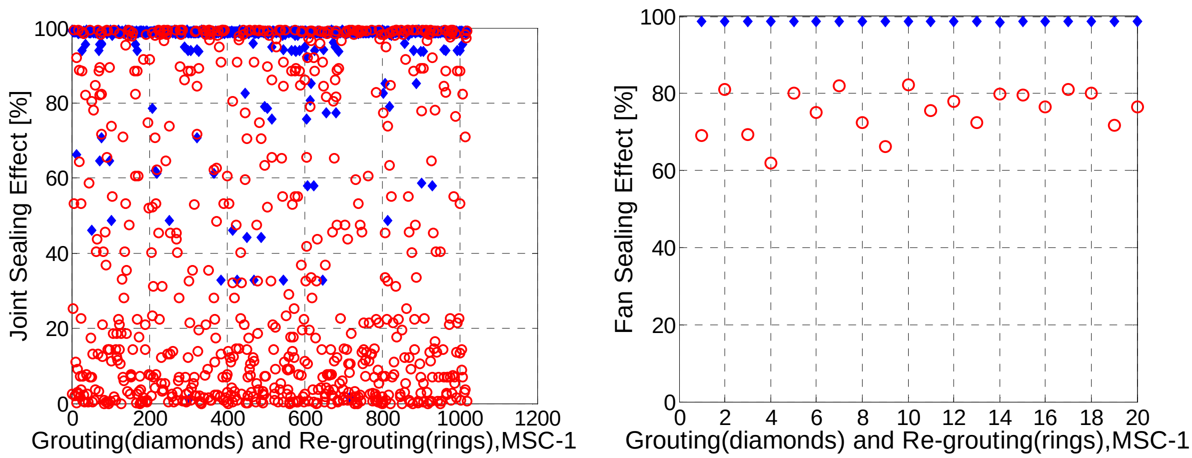


Figure 5-90 Joint and fan sealing effect after grouting and re-grouting for MSC-1.

In Figure 5-91 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 800 seconds implies a high sealing effect; shorter grouting time can both implies lower, as well as, a medium sealing effect. Re-grouting performed more than 200 seconds also implies a high sealing effect, shorter re-grouting times can both imply lower, as well as, a medium sealing effect.

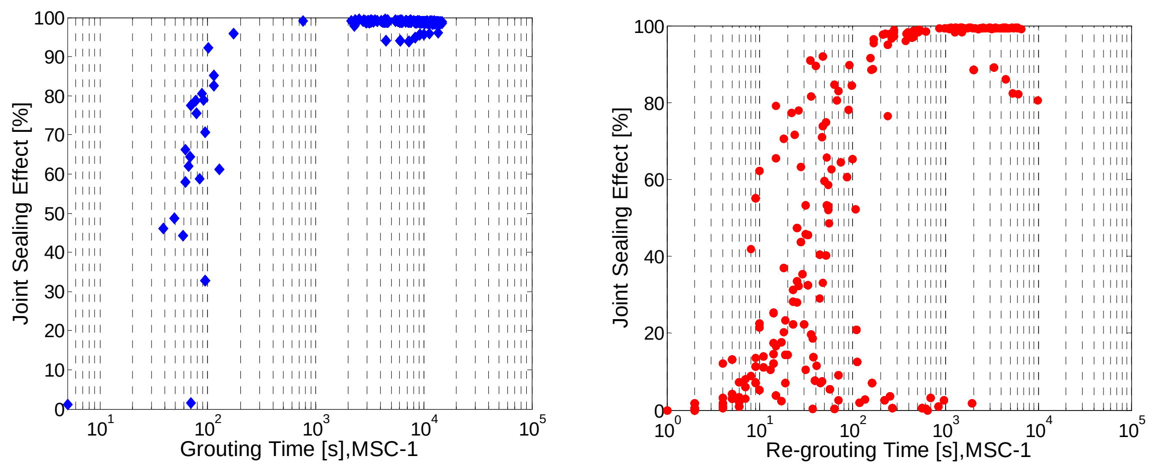


Figure 5-91   Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-92 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 12 000 seconds implies a high sealing effect around 98.7 %. Re-grouting for more than 4000 seconds implies an increased sealing effect to around 99.7 %.

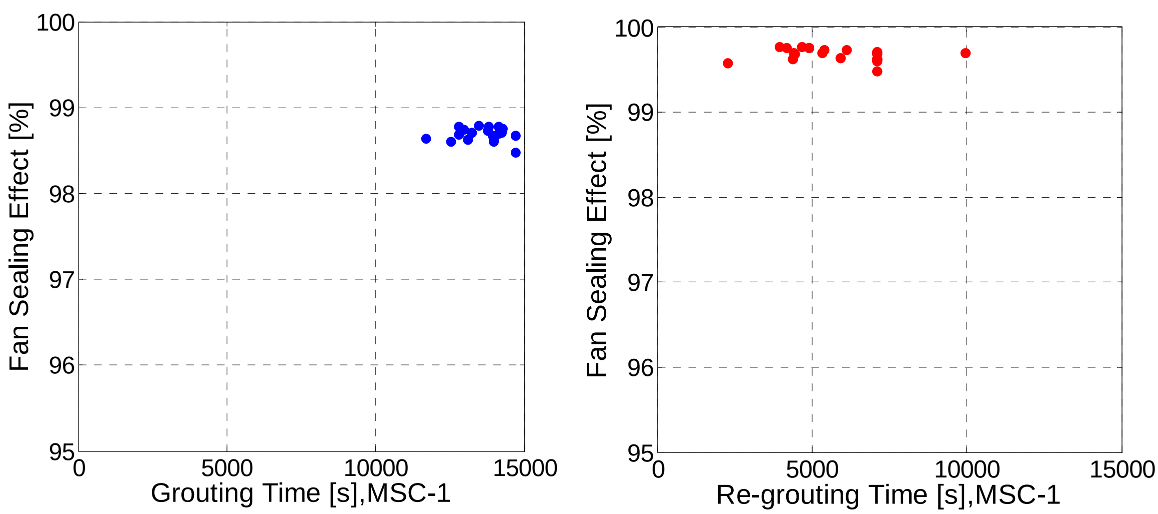


Figure 5-92   Accumulated sealing effect on a fan level after grouting and re-grouting.

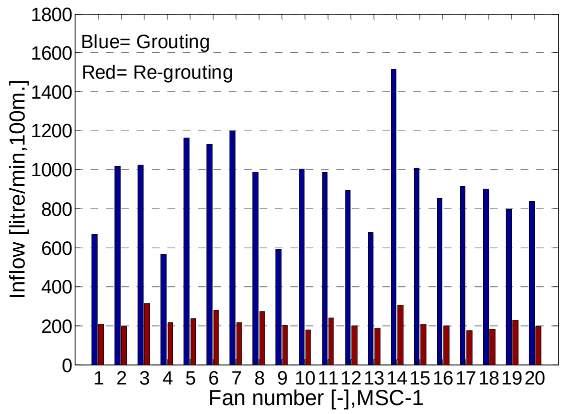


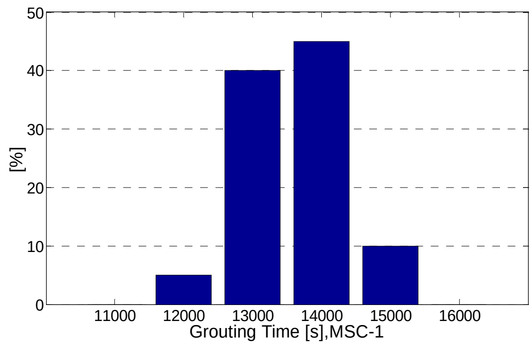
Figure 5-93 Inflow to the tunnel from the individual grout fans.

In Figure 5-93 the inflow from the individual fans after grouting and re-grouting is shown. No outliers are noted. After re-grouting the inflow variability has decreased to an even level in the range of 200 litres/min/100 meters. For this method re-grouting is necessary to reach an even sealing level for all fans. An alternative could also be one grouting round, but with decreased hole spacing.

If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 20 of 20 fans, the inflow after grouting is above that required and in 20 of 20 fans the inflow is even above the requirement after re-grouting. The rock mass C with water pressure equal to 1.5 MPa is far beyond the capacity of method MSC-1.

The calculation of the flow for the different fans gave re-grouting in 20 of 20 fans, which could be compared with the re-grouting criteria for the control holes that also gave 20 re-groutings. To compare them are difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-94 the grouting time distribution for the fans and the calculated inflow after grouting for MSB-1 are shown.



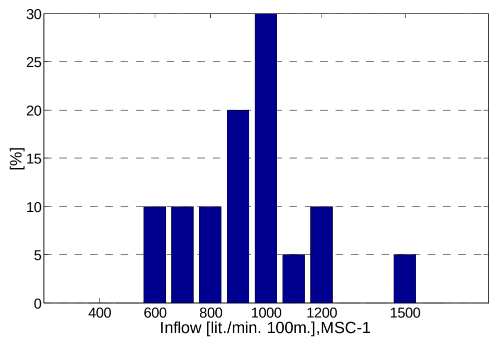


Figure 5-94 Fan grouting time- and fan inflow distribution for method MSC-1.

The average grouting time for a fan was calculated to 13 581 seconds (3 hour and 46 minutes) per fan, with a standard deviation of 774 seconds (13 min.), 85 % of the fans had a grouting time between 13 000 and 14 000 seconds. The average inflow after grouting was calculated to 937 litres/min/100 meters, with a standard deviation of 225 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 1467 m^3 or as an average of 73 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-14. As shown the sealing effect is increased after re-grouting based on the control holes. The sealing effect is not further increased by a full re-grouting. The control holes, can for this situation, be regarded as a good parameter for judging the need for re-grouting.

Table 5-14 Properties on a fan level of MSC-1.

	After grouting	**After grouting + control holes re-grouting**	**After re-grouting**
Sealing Effect for a fan* [%]	μ=98.7 σ=0.08	μ=99.7 σ=0.08	μ=99.7 σ=0.08
Time for a fan [s]	μ=13 581 σ=774	μ=19 252 σ=1989	μ=19 252 σ=1989

*Average sealing effect for 20 fans.

5.7.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grout time to perform the grouting of 20 fans with method MS-1 in rock mass C was calculated to 271 614 seconds. To perform re-grouting for all fans was calculated to 113 428 seconds, which is about 42 % of the time for grouting

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-15.

Table 5-15 Properties on a tunnel level of MSC-1.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After re-grouting**
Inflow to tunnel [l/min, 20 fans]	285 230	3748	885	885
Sealing effect for a tunnel* [%]		98.7	99.7	99.7
Time for a tunnel [s]		271614	385042	385042
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MSB-1 fulfils an inflow requirement of 937 litres/min/100 meters and after full re-grouting an inflow requirement of 221 litres/min/100 meters is achieved.

5.8 Result Method MS, Rock mass C (MSC)

5.8.1 Comparison on joint level

The grouting method, MS, has been applied to rock mass C. The basis for the evaluation is the calculation of each possible joint in the interval 0-1400 µm, which gives 560 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-95. As shown, both the time and the volume are dependent on the joint aperture. For apertures larger than 800 µm, the grouting time occasionally and increasingly shows lower values. The explanation is that for the larger apertures, the Lugeon value before grouting sometimes can be above 1.0 LU, which means that the start grout is changed from W/C 2.0 to a W/C 1.0 grout. The lower W/C ratio gives a higher viscosity and thereby a shorter grouting time. This effect is only rarely visible for rock mass B, due to a lower hydraulic pressure than in rock mass C.

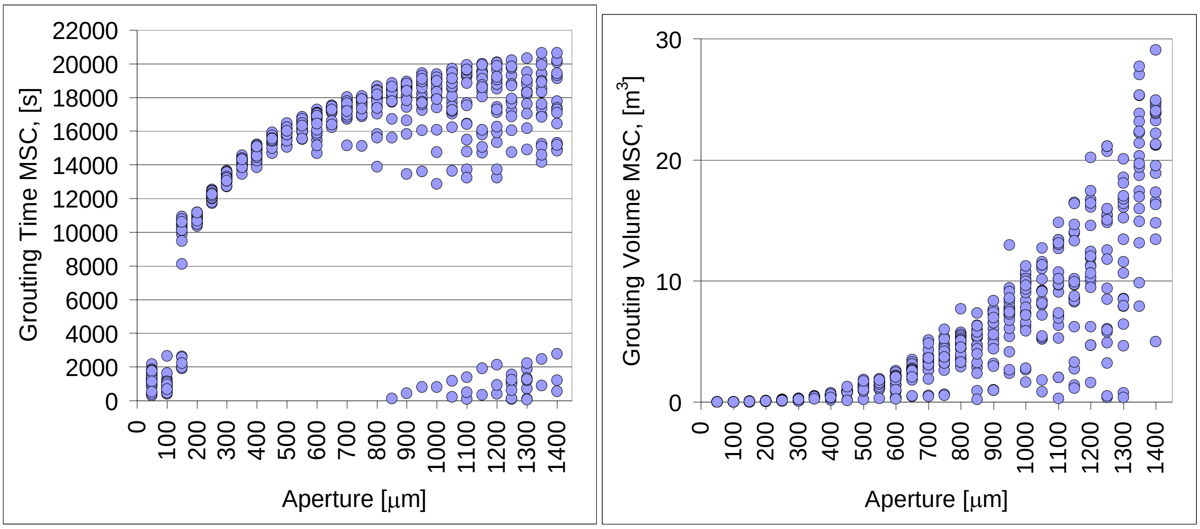


Figure 5-95 Grouting time and grouting volume for different apertures MSC.

All the joints have also been re-grouted, as shown in Figure 5-96. For all joints and apertures the re-grouting time is significantly shorter than the grouting time. The re-grouting is performed with three different concepts, based on the Lugeon value prior to the re-grouting. For Lugeon values below 0.3, re-grouting is started with a micro cement based grout with a W/C ratio of 2.0. For Lugeon values between 0.3 and 1.0, re-grouting is started with a conventional grout cement with a W/C ratio of 2.0. For Lugeon values above 1.0, re-grouting is started with a micro cement based grout with W/C ratio of 2.0. The three starting concepts for re-grouting may be an explanation for the groups of values shown as time /aperture in Figure 5-96.

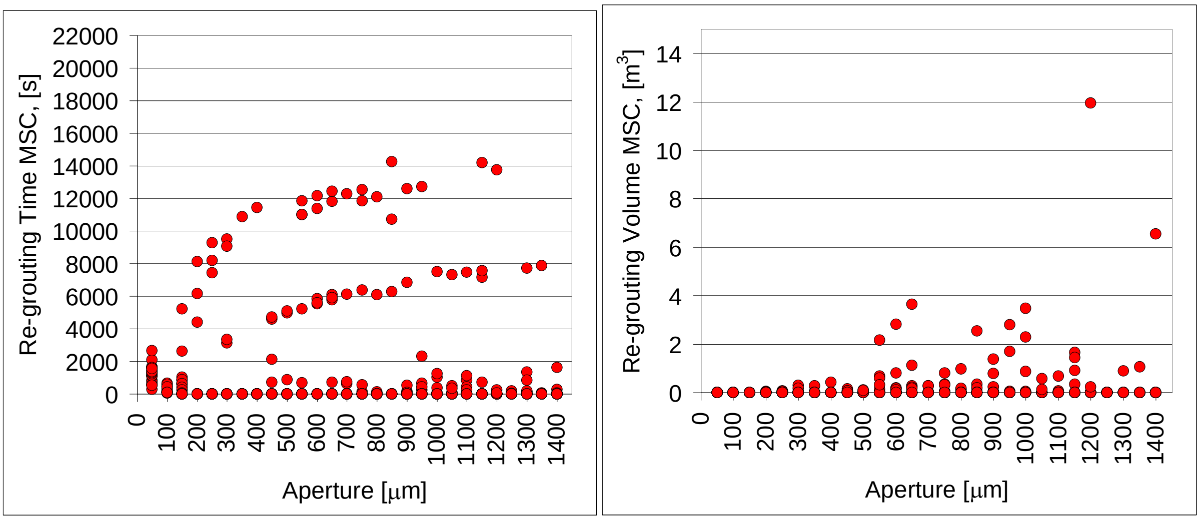


Figure 5-96 Re-grouting time and re-grouting volume for different apertures MSC.

The sealing effect, as shown in Figure 5-97, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect is in general higher when measured as cross joint flow rather than in control holes. The sealing effect after grouting is high for apertures larger than 150 µm, but low for smaller apertures. For joints with an aperture of 50 µm the sealing time as well as the sealing effect were almost zero. A re-grouting improves all apertures, but compared to rock mass A and B, re-sealing is more effective for rock mass C.

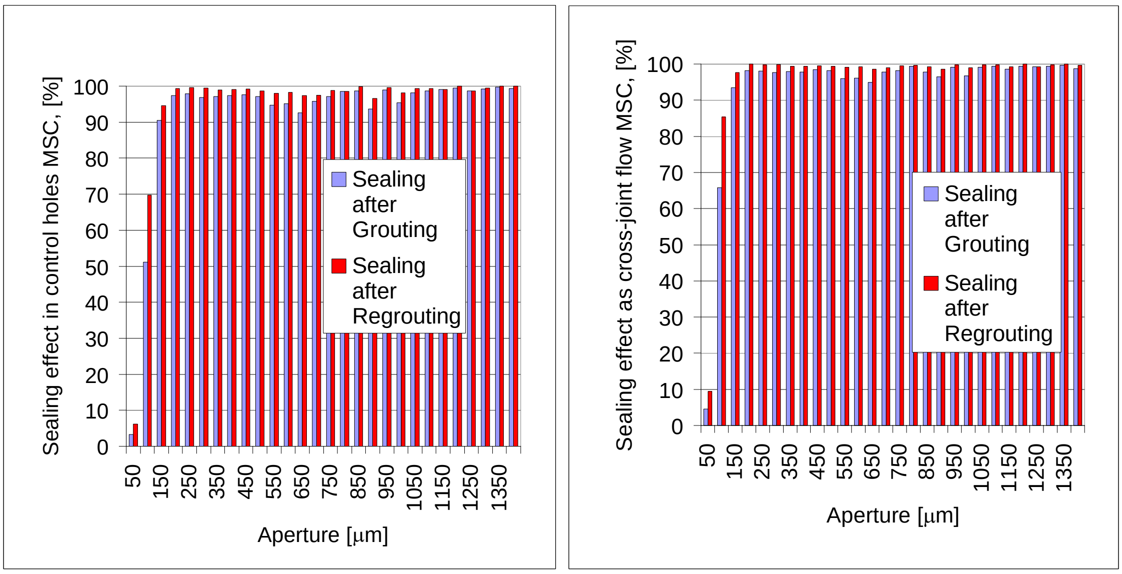


Figure 5-97 Sealing effect for different apertures MSC, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.8.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

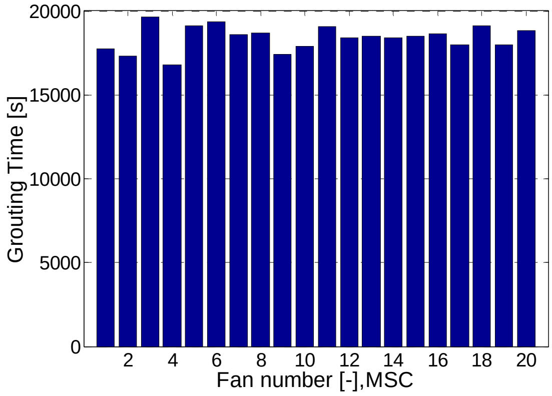
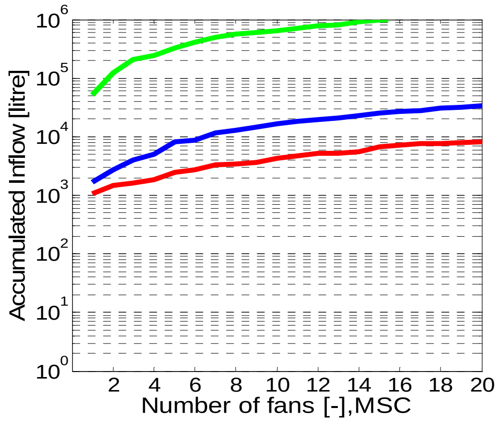


Figure 5-98 Grouting time for the individual grout fans (MSC).

In Figure 5-98 the grouting times for the individual fans are shown. The average grouting time was 18400 seconds, with a standard deviation of 734 seconds. The grouting time is more evenly distributed than for MSA and MSB, which is because of the higher joint intensity in rock mass C. The average time is higher and the standard deviation is lower than in rock mass A and B, which is also because of a higher joint intensity. In rock mass A it was possible to get fans with just a few narrow apertures, which resulted in 25 % of the fans that had a grouting time close to zero. As shown in Figure 5-85 none of the fans have a grouting time close to zero.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-99 the inflow for method MS in rock mass C is shown. From the figure, the large difference before and after grouting could be noticed. Between first and second grouting the flow was further reduced, but by a much lesser magnitude. The average sealing effect was after grouting for the fans calculated to 98.7 % and after re-grouting to 99.7 %.



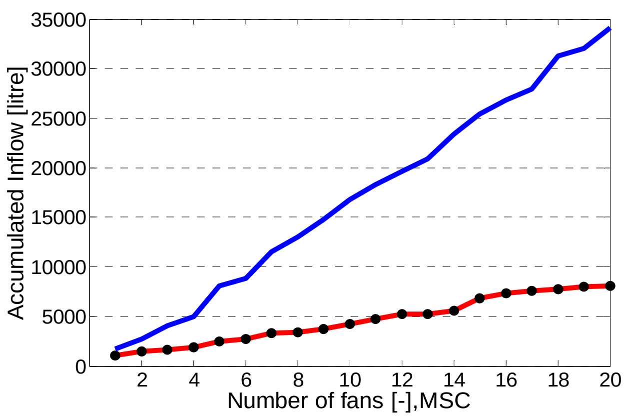


Figure 5-99 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

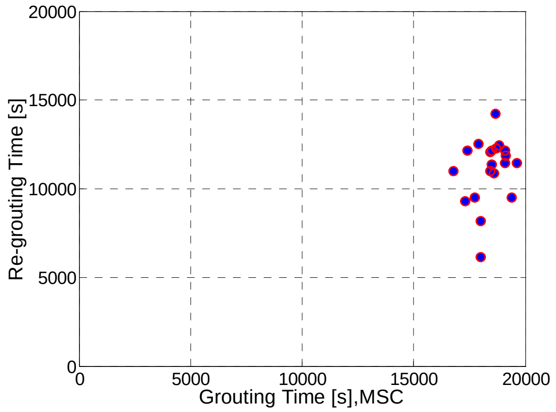


Figure 5-100 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-100 the relationship between the grouting time and the re-grouting time is shown. No relationship could be noticed between the grouting time and the re-grouting time for MSC.

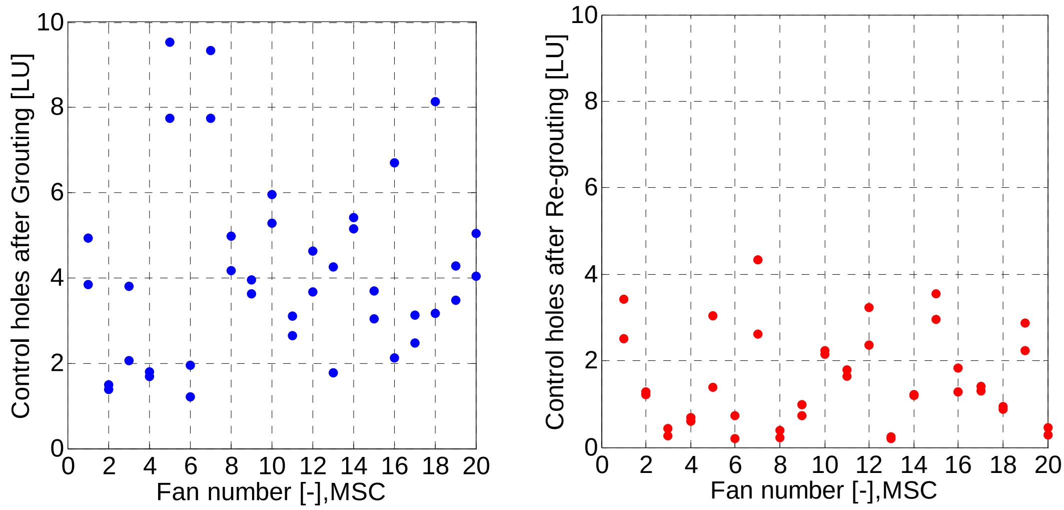


Figure 5-101 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 40 of the 40 control holes the Lugeon value after grouting were above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-101 two control holes for each fan half after grouting and after re-grouting are shown. All of the control holes have a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-101 resulted in 20 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. The total re-grouting time was calculated to 221 666 seconds.

On the other hand, if re-grouting is carried out and tested, 20 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-102 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting, (re-grouting) is below 3606 (338) litres/min/100 meters for method MSC, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 16581 (6015) litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-102 the inflow convergence stabilizes after about 6 fans for the grouting of MSC and after about 6 fans for the re-grouting.

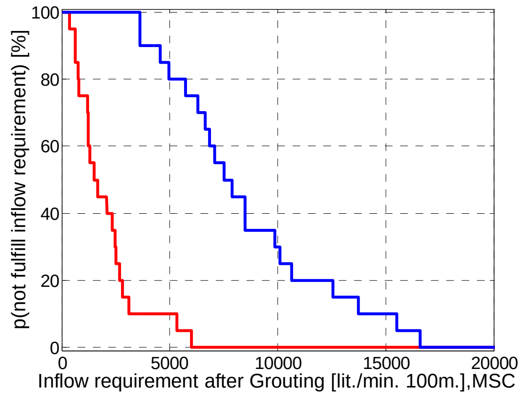


Figure 5-102 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MSC (left). Inflow convergence after n number of fans for MSC (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-103. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On the joint level grouting reaches a high sealing effect and re-grouting a much lower effect. On the fan level the average value of all joints belonging to a fan is shown. The sealing effect will then consequently be higher because narrow joints with low sealing effect give a much smaller contribution to the total inflow than the wider joints. The fan sealing effect after grouting for MSC is equal to the sealing effect for MSB, but is higher than for MSA. The sealing effect is not increased between rock mass B to rock mass C, which may be because the rock mass B is already rather highly conductive and a further increased conductivity does not improve the chance to intersect the conductive part of the joint planes.

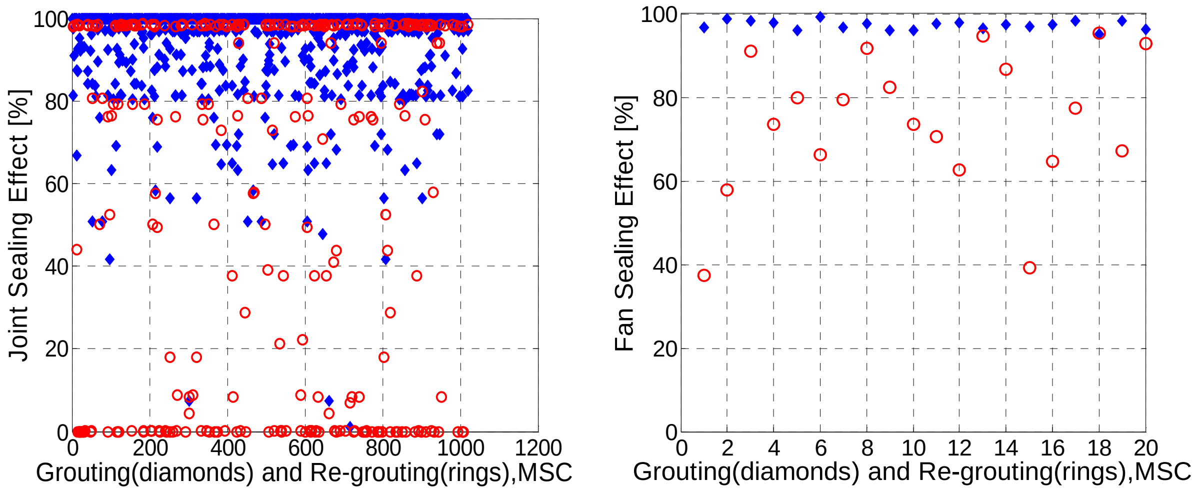


Figure 5-103 Joint and fan sealing effect after grouting and re-grouting for MSC.

In Figure 5-104 the grouting time and the re-grouting time to reach different levels of sealing effect for the randomly selected individual joints on a fan level is shown (sealing effect is calculated before and after grouting, no accumulated values). Grouting performed for more than 9000 seconds imply a sealing effect between 80 % and 100 %, shorter grouting time implies a lower sealing effect. Re-grouting performed for more than 2000 seconds implies a high sealing effect, shorter re-grouting implies a lower sealing effect.

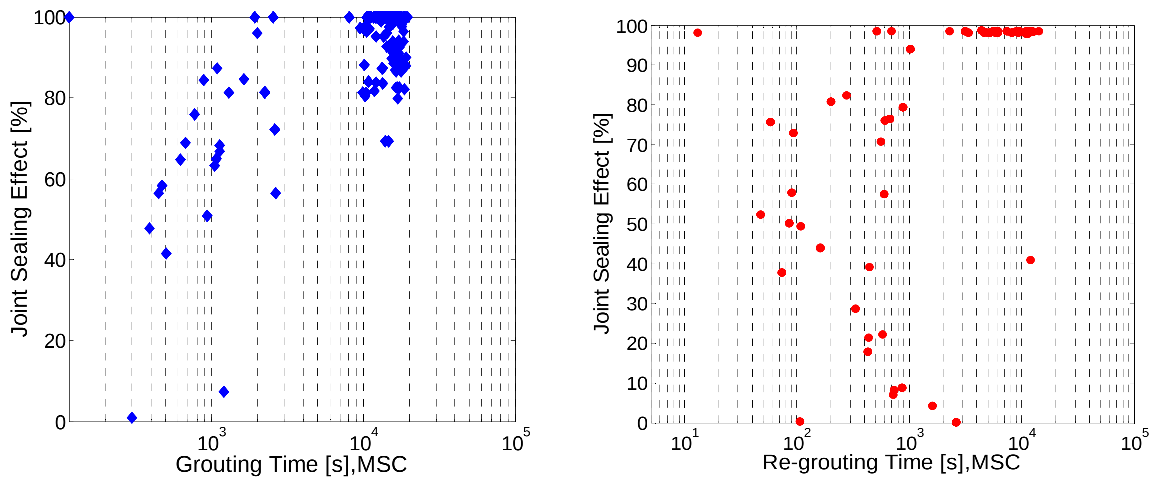


Figure 5-104 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-105 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 12 000 seconds imply a sealing effect of between 95 % and 99 %. Re-grouting for more than 3 000 seconds implies a high sealing effect.

Compared to method MSB-1 the sealing effect for this method is lower and the time is longer, which may be explained by an extended use of grout mixes with a higher viscosity and a decreased minimum stop flow.

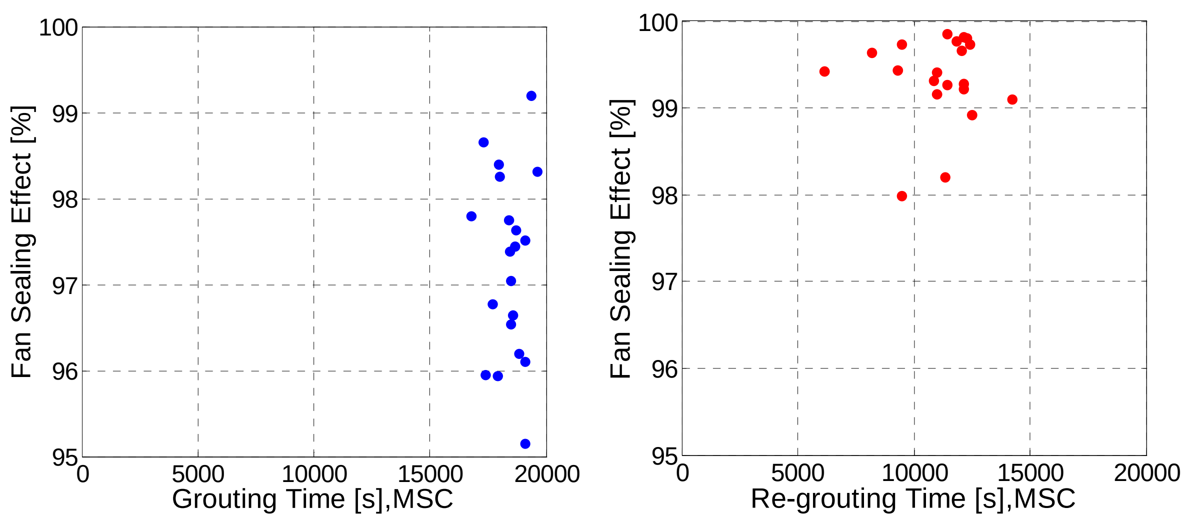


Figure 5-105 Accumulated sealing effect on a fan level after grouting and re-grouting.

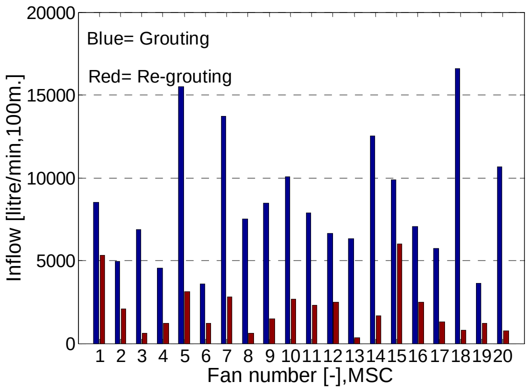


Figure 5-106 Inflow to the tunnel from the individual grout fans.

For rock mass C the inflow is very high, which is explained by a very conductive rock mass. In Figure 5-106 the inflow from the individual fans after grouting and re-grouting is shown. No outliers are noted. After grouting as well as after re-grouting the inflow variability is high, which may suggest that a third grouting round or more is necessary to reach an even inflow distribution. An alternative could also be one grouting round, but with decreased hole spacing.

If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 20 of 20 fans the inflow after grouting is above that required and in 20 of 20 fans the inflow is above the requirement even after re-grouting. The rock mass C with water pressure equal to 1.5 MPa is far beyond the capacity of method MSC.

The calculation of the flow for the different fans gave re-grouting in 20 of 20 fans, which could be compared with the re-grouting criteria for the control holes that also gave 20 re-groutings. To compare them is difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-107 the grouting time distribution for the fans and the calculated inflow after grouting for MSC are shown.

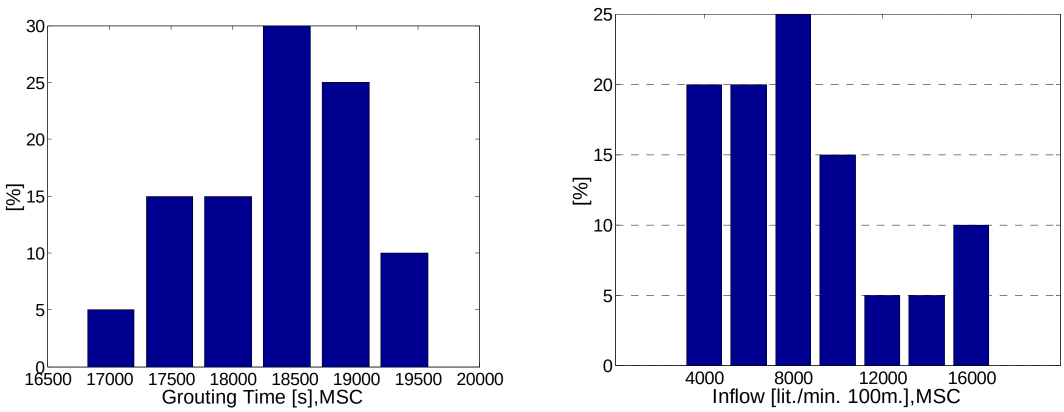


Figure 5-107 Fan grouting time- and fan inflow distribution for method MSC.

The average grouting time for a fan was calculated to 18 400 seconds (5 hour and 6 minutes), with a standard deviation of 734 seconds (12 min.), 30 % of the fans had a grouting time around 18500 seconds. The average inflow after grouting was calculated to 8534 litres/min/100 meters, with a standard deviation of 3738 litres/min/100 meters.

The accumulated grouting volume after 20 fans was calculated to 1416 m^3 or as an average of 71 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-16. As shown, the sealing effect is increased after re-grouting based on the control holes. The sealing effect is not further increased by a full re-grouting. The control holes, can for this situation, be regarded as a good parameter for judging the need for re-grouting.

Table 5-16 Properties on a fan level of MSC.

	After grouting	**After grouting + control holes re-grouting**	**After re-grouting**
Sealing Effect for a fan* [%]	μ=97.2 σ=1.1	μ=99.3 σ=0.5	μ=99.3 σ=0.5
Time for a fan [s]	μ=18 400 σ=734	μ=29 484 σ=2122	μ=29 484 σ=2122

*Average sealing effect for 20 fans.

5.8.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total grout time to perform the grouting of 20 fans with method MS in rock mass C was calculated to 368 008 seconds. To perform re-grouting for all fans was calculated to 221 666 seconds, which is about 60 % of the time for grouting

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters have been calculated and are shown in Table 5-17

Table 5-17 Properties on a tunnel level of MSC.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After re-grouting**
Inflow to tunnel [l/min, 20 fans]	1 267 300	34 137	8108	8108
Sealing effect for a tunnel* [%]		97.3	99.4	99.4
Time for a tunnel [s]		368 008	589 674	589 674
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MSB fulfils an inflow requirement of 8534 litres/min/100 meters and after full re-grouting an inflow requirement of 2027 litres/min/100 meters is achieved.

5.9 Result Method MA, Rock mass C (MAC)

5.9.1 Comparison on joint level

The grouting method, MA, has been applied to rock mass C. The basis for the evaluation is the calculation of each possible joint in the interval 0-1400 μm, which gives 560 individual joint calculations, with randomized properties. The grouting time for different apertures and the corresponding grouting volume are presented in Figure 5-108. As shown, both the time and the volume are dependent on the joint aperture, but the variance is much higher for the larger apertures. This is due to the fact that a joint can be intersected in a more or less conductive part and this affects the larger apertures more than the smaller, due to the "Cubic law" equation.

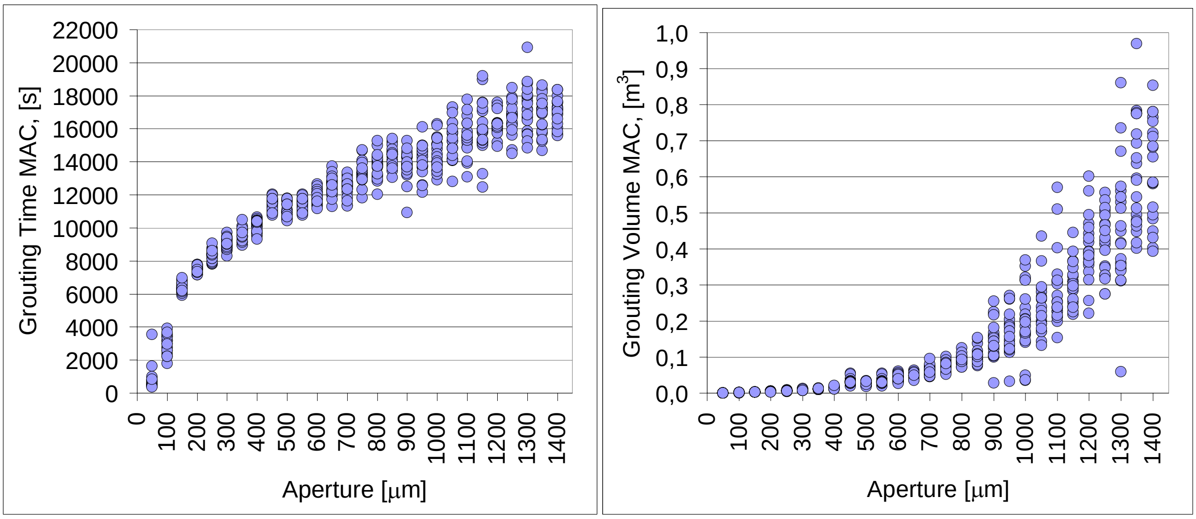


Figure 5-108 Grouting time and grouting volume for different apertures MAC.

All the joints have also been re-grouted, as shown in Figure 5-109. The re-grouting time is almost as long as the grouting time, which indicates that the hole spacing is to wide.

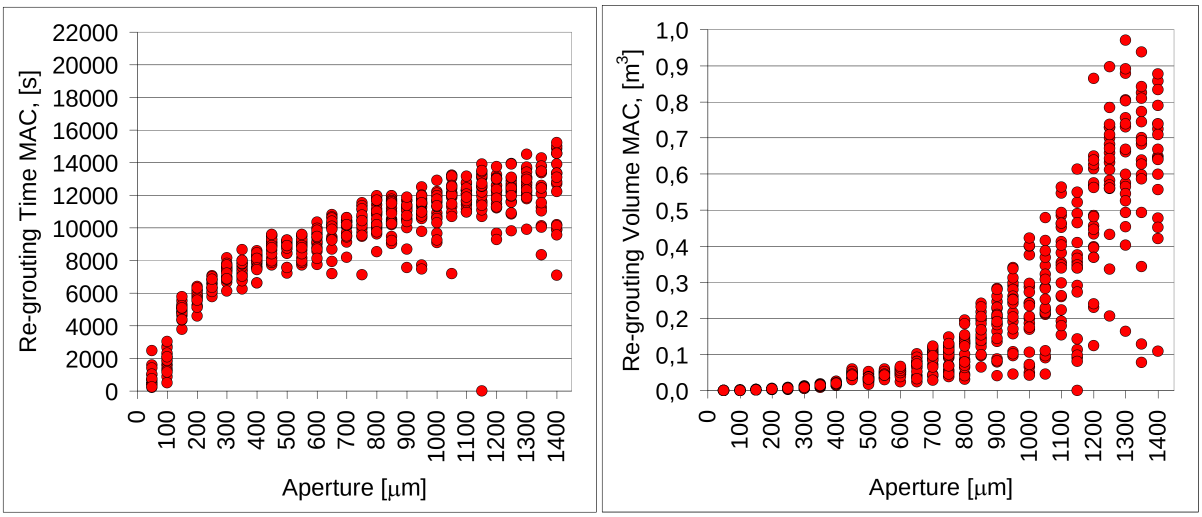


Figure 5-109 Re-grouting time and re-grouting volume for different apertures MAC.

The sealing effect, as shown in Figure 5-110, is calculated from cross joint flow and from Lugeon values measured in control holes. From the figure, it could be noticed that the sealing effect in general is higher when measured as cross joint flow rather than in control holes. The sealing effect after grouting is low. The re-grouting improves the sealing effect from around 30 % to around 90 %.

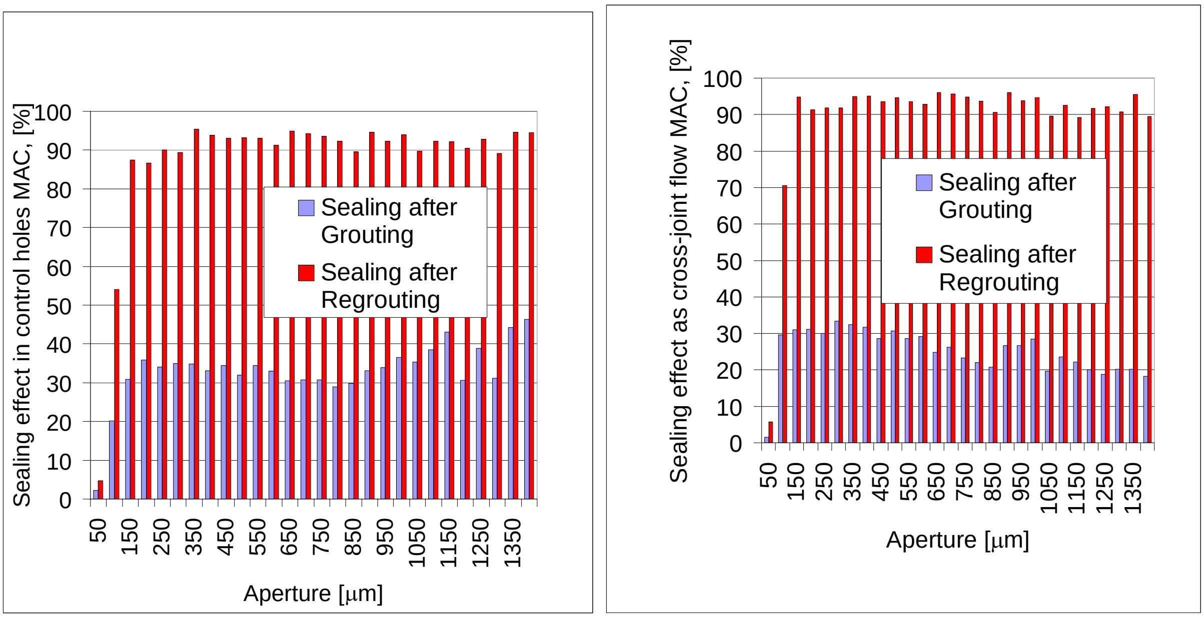


Figure 5-110 Sealing effect for different apertures MAC, shown as Lugeon value from control holes (left) and as inflow to the tunnel (right).

5.9.2 Comparison on fan level

In order to study the expected behaviour of a fan, a set of joints have randomly been added together from a log-normal distribution. The grouting time is the maximum time among the joints of each fan and the grouting volume is the sum of the volume of the joints of each fan. The number of randomly chosen fans used to represent a specified rock mass were calculated to 20 in chapter 4.

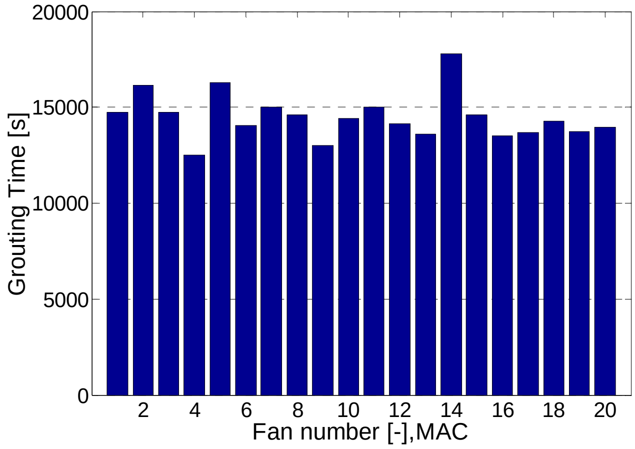
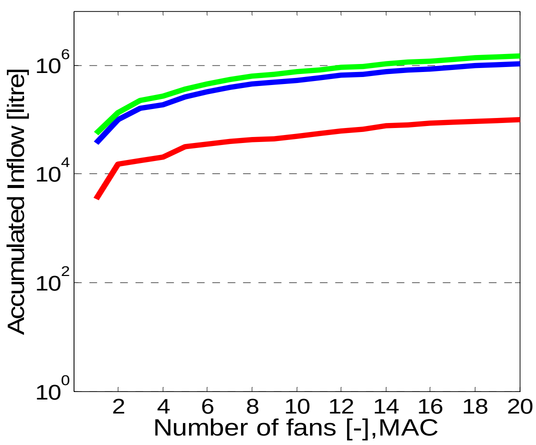


Figure 5-111 Grouting time for the individual grout fans (MAC).

In Figure 5-111 the grouting time for the individual fans are shown. The average grouting time was 14 491 seconds, with a standard deviation of 1201 seconds. The average time is higher and the standard deviation is lower than in rock mass A and B, which is because of a higher joint intensity. For MAA it was possible to have fans with just a few narrow apertures, which resulted in fans that had a grouting time close to zero. As shown in Figure 5-111 none of the fans had a grouting time close to zero.

The accumulated inflow of water for the fans has been calculated before grouting, after grouting and after re-grouting. In Figure 5-112 the inflow for method MA in rock mass C is shown. From the figure the large difference before and after grouting could be noticed. Between grouting and re-grouting the flow was further reduced.



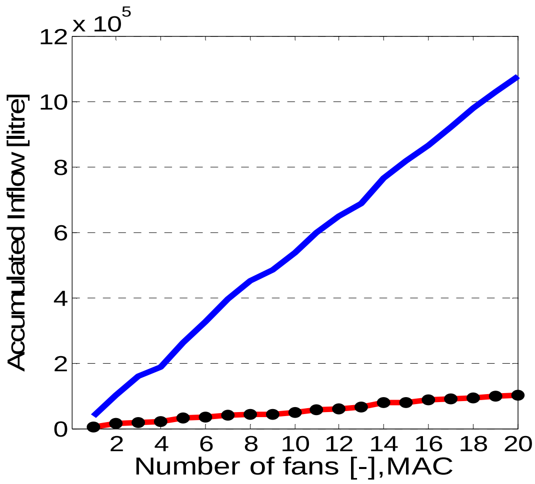


Figure 5-112 Accumulated inflow before, after and after re-grouting.
The dots are after grouting + re-grouting based on control holes.

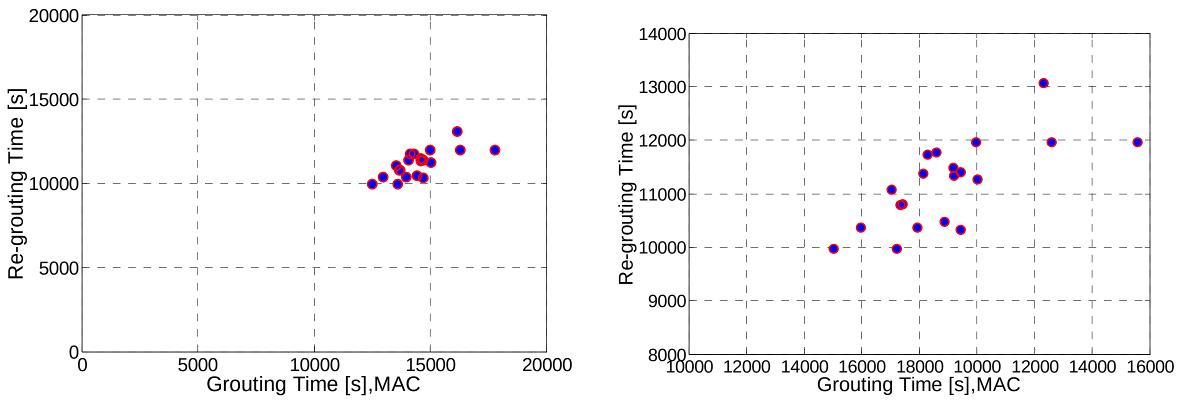


Figure 5-113 Relationship between grouting time and re-grouting time.

The re-grouting is performed in new holes positioned between the original grouting holes. In Figure 5-87 the relationship between the grouting time and the re-grouting time is shown. There is a tendency that a higher grouting time gives a higher re-grouting time for MAC. Among all nine calculated combinations of method and rock mass, relationships between grouting and re-grouting time have only been noted for MAA and MAC.

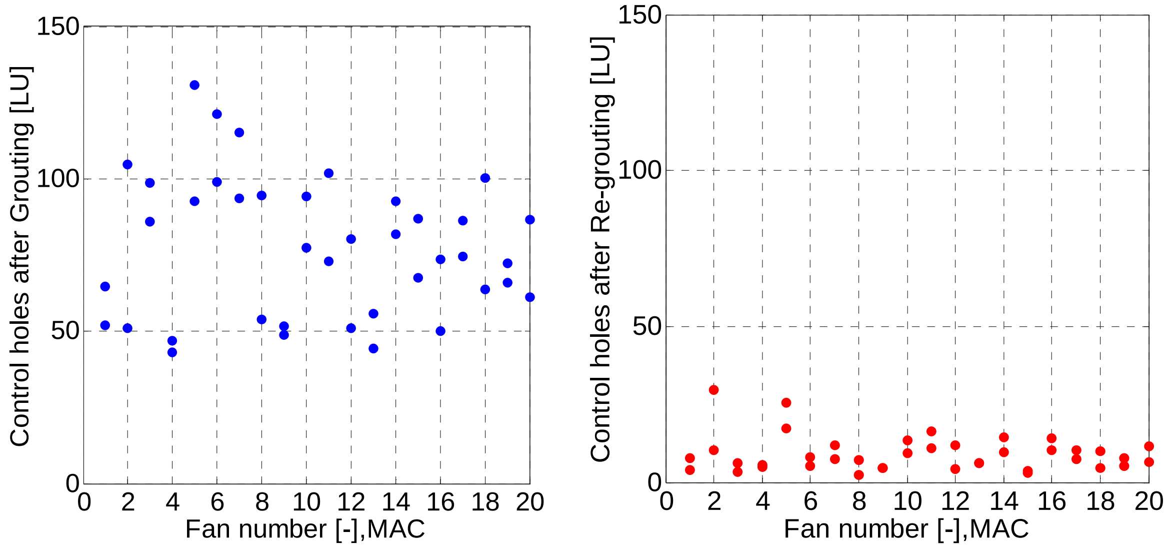


Figure 5-114 Water loss measurements after grouting (left) and after re-grouting (right) in 2 control holes for each fan half. For 40 of the 40 control holes, the Lugeon value after grouting was above 0.15 LU, which was the criterion for re-grouting.

In Figure 5-114 two control holes for each fan half after grouting and after re-grouting are shown. All of the control holes had a Lugeon value equal to or above the set criteria for re-grouting of 0.15 Lugeon. The control holes shown in Figure 5-114 resulted in 20 re-groutings. The time for re-grouting and the re-grouting volume are then added to the total sealing time and volume. The total re-grouting time is 223 450 seconds.

On the other hand, if re-grouting is carried out and tested, 20 of the control holes would have a Lugeon value above 0.15 Lugeon and should, if the method suggested, consequently be re-grouted a second time.

For different inflow criteria the probability to fulfil the requirement will vary. In Figure 5-115 the probability to be below the inflow requirement is shown. If the inflow requirement after grouting is below 150 000 litres/min/100 meters for method MAC, the chance to not fulfil the requirement is 100 %. If on the other hand the inflow criterion is above 390 000 litres/min/100 meters the chance to not fulfil the requirement is 0 %.

The inflow convergence is a prediction of the final inflow after 20 fans. The inflow convergence is calculated as the accumulated average inflow after *n* number of fans. As shown in Figure 5-115 the inflow convergence stabilizes after about 5 fans for the grouting of MAC and after about 4 fans for the re-grouting.

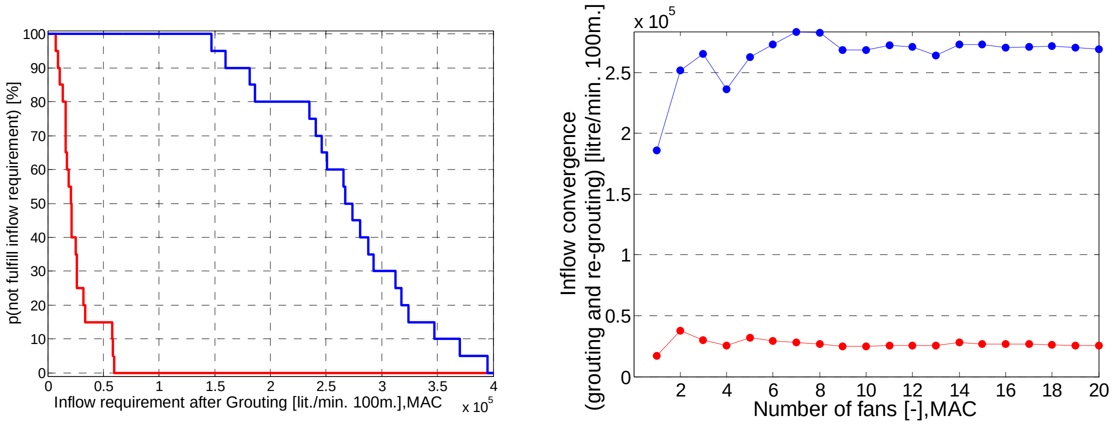


Figure 5-115 Possibility to not fulfil the inflow requirement after grouting and re-grouting for MAC (left). Inflow convergence after n number of fans for MAC (right).

The sealing effect after grouting and re-grouting on a joint level and on a fan level is shown in Figure 5-116. The sealing effect is calculated as the difference before and after grouting and before and after re-grouting (no accumulated sealing effect). On a joint level re-grouting reaches a higher sealing effect than the grouting, which is the opposite to that found for the other methods. The explanation is that for the other methods re-grouting affected joints that after grouting were partly sealed. For Method MAC, re-grouting seals new joints, which are unaffected by the prior grouting. After grouting with MAC the inflow is still very high because a large part of the joints are completely unaffected. The method MA, with 1.8 meters between the grout holes, is therefore considered as unsuitable for rock mass C.

The above explanation is applicable even for the fan sealing effect.

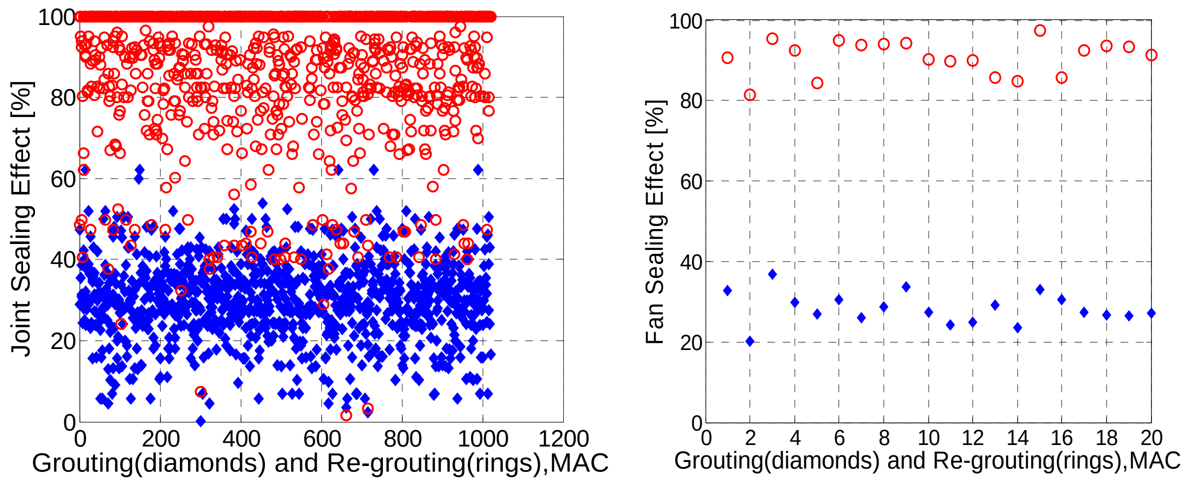


Figure 5-116 Joint and fan sealing effect after grouting and re-grouting for MAC.

In Figure 5-117 the grouting time and the re-grouting time to reach different levels of sealing effect, for the randomly selected individual joints on a fan level, is shown (sealing effect is calculated before and after grouting, no accumulated values).

Grouting performed more for than 6000 seconds imply a sealing effect between 10 % and 50 %, shorter grouting time implies lower sealing effect. Re-grouting performed more than 5000 seconds imply a sealing effect of between 60 % and 100 %, shorter re-grouting time implies a lower sealing effect.

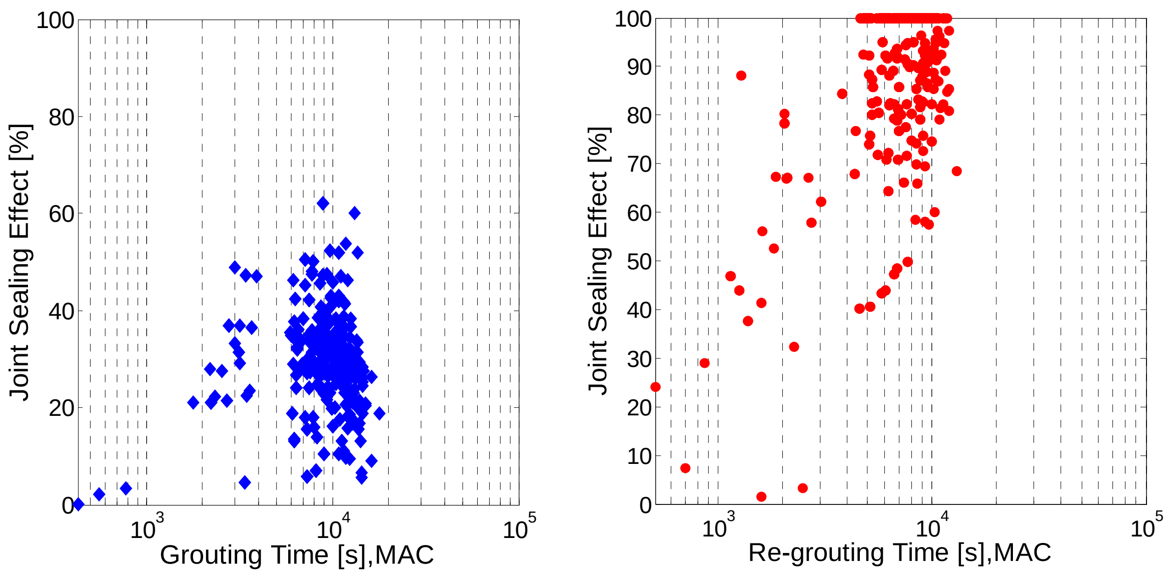


Figure 5-117 Sealing effect of joints on a fan level after grouting and re-grouting.

In Figure 5-118 the grouting time and the re-grouting time to reach different levels of accumulated sealing effect on a fan level is shown. Grouting for more than 13 000 seconds implies a low sealing effect of around 28 %. Re-grouting for more than 10 000 seconds implies an increased sealing effect around 90 %.

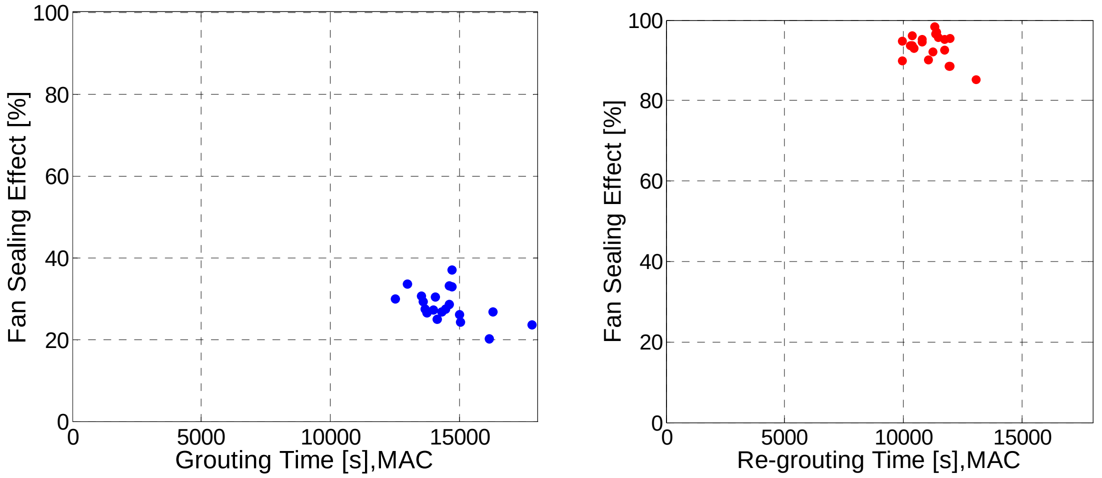


Figure 5-118 Accumulated sealing effect on a fan level after grouting and re-grouting.

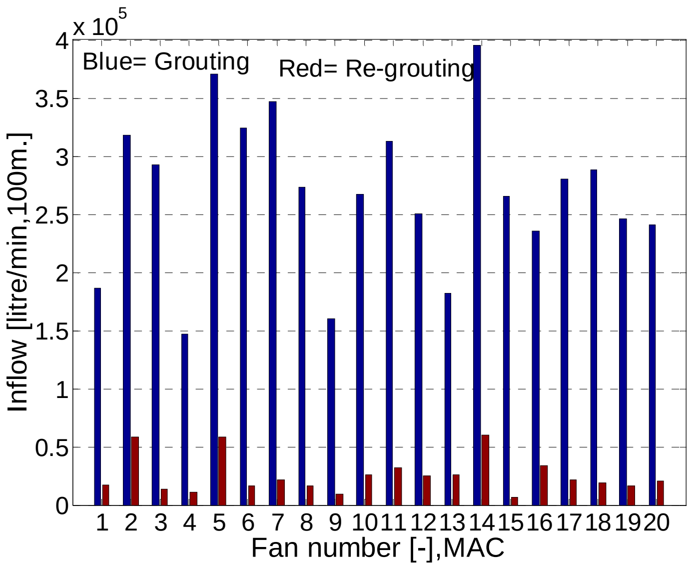


Figure 5-119 Inflow to the tunnel from the individual grout fans.

In Figure 5-119 the inflow from the individual fans after grouting and re-grouting is shown. As noticed the inflow is very high, which is due to an extremely conductive rock mass (hydraulic conductivity of 1.52 $*10^{-4}$). The rock mass may normally only occur for a short section of a tunnel and not as calculated here, for an entire tunnel. Therefore the inflows calculated here are only for comparison. The achieved sealing effect received between grouting and re-grouting is very high and much higher than have been calculated earlier for other methods and rock masses.

If the inflow requirement of 5 litres/min/100 meters, is applied on a fan length of 20 meter and substituted to 1 litre/min/20 meters, then in 20 of 20 fans the inflow after grouting is above that required and in 20 of 20 fans the inflow is even above the after re-grouting. The rock mass C with water pressure equal to 1.5 MPa is far beyond the capacity of method MAC.

The calculation of the flow for the different fans gave re-grouting in 20 of 20 fans, which could be compared with the re-grouting criteria for the control holes that also gave 20 re-groutings. To compare them are difficult, because the relationship between inflows in probe holes and inflow as cross joint flow depends on e.g. different radii of influence.

In Figure 5-120 the calculated grouting time distribution and the inflow after grouting for MAC are shown.

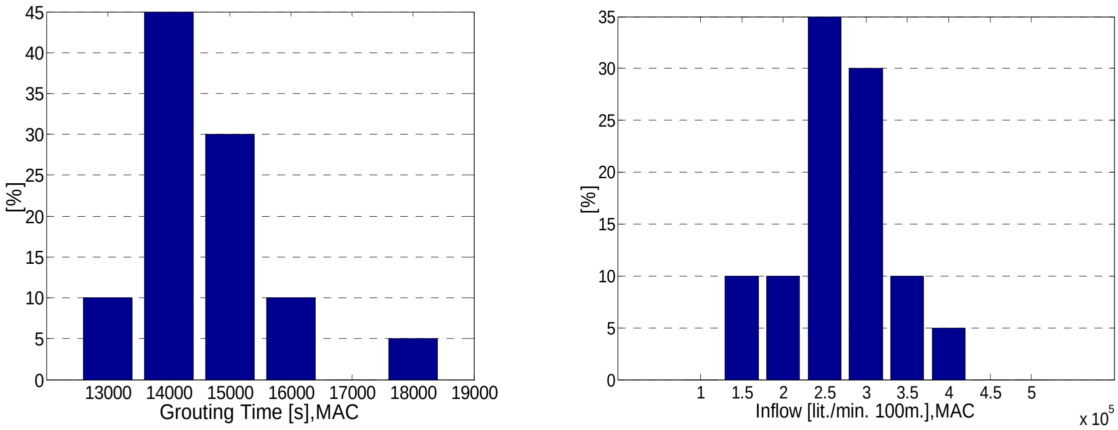


Figure 5-120 Fan grouting time- and fan inflow distribution for method MAC.

The average grouting time for a fan was calculated to 14 491 seconds (4 hour and 1 minute) per fan, with a standard deviation of 1201 seconds (20 min.), 45 % of the fans had a grouting time close to 14000 seconds. The average inflow after grouting was calculated to 269 060 litres/min/100 meters, with a standard deviation of 66 754 litres/min/100 meters. The accumulated grouting volume after 20 fans was calculated to 38 m^3 or as an average of 1.9 m^3 per fan.

The sealing effect and the time, after grouting and after re-grouting are shown in Table 5-18. As shown, the sealing effect is increased after re-grouting based on the control holes. The sealing effect is not further increased by a full re-grouting. The control holes can, for this situation, be regarded as a good parameter for judging the need for re-grouting.

Table 5-18 Properties on a fan level of MAC.

	After grouting	**After grouting + control holes re-grouting**	**After re-grouting**
Sealing Effect for a fan* [%]	μ=28.3 σ=3.9	μ=93.3 σ=3.4	μ=93.3 σ=3.4
Time for a fan [s]	μ=14 491 σ=1201	μ=25 664 σ=1854	μ=25 664 σ=1854

*Average sealing effect for 20 fans.

5.9.3 Comparison on tunnel level

On a tunnel level the randomized fans have been added to represent a tunnel section with rock mass properties corresponding to a characteristic region, as discussed in chapter 4.2.2.

The total time to perform the grouting of 20 fans with method MA in rock mass C was calculated to 289 823 seconds. To perform re-grouting for all fans was calculated to 223 450 seconds, which is about 78 % of the time for grouting.

The inflow and sealing effect; before grouting, after grouting and after re-grouting as well as the total grouting time and the number of fans, which need re-grouting if the inflow criteria is set to 5 litres/min/100 meters, have been calculated and are shown in Table 5-19

Table 5-19 Properties on a tunnel level of MAC.

	Before	**After grouting**	**After grouting + control holes re-grouting**	**After re-grouting**
Inflow to tunnel [l/min, 20 fans]	1 495 700	1 076 300	116 900	116 900
Sealing effect for a tunnel* [%]		28.0	93.2	93.2
Time for a tunnel [s]		289 823	513 273	513 273
Number of fans above 5 litres/min/100 meters	20	20	20	20

*based on total inflow and total reduction for 20 fans.

After grouting the method MAC fulfils an inflow requirement of 269000 litres/min/100 meters and after full re-grouting an inflow requirement of 254000 litres/min/100 meters is achieved.

Chapter Six

6 Analysis and conclusions of grouting method evaluation

The results of numerical grouting with three different grouting methods in three different rock masses have been presented in chapter 5. In this chapter, the results will be analysed and conclusions from numerical grouting in the different rock masses presented. As a grouting method depends on a number of parameters, the conclusions made here are the author's opinion based on the calculations.

This chapter initially presents conclusions which are compared on a joint-, fan- and tunnel level. Thereafter the conclusions from using the different methods in the different rock masses are presented.

For all comparisons the following is to be regarded.

The three different hypothetical rock masses were chosen to get a wide range of the number of joints, the joint aperture and the hydraulic conductivity. The three different grouting methods were chosen based on their ability to provide a wide range of grout mixes, to use different stop criteria, different hole spacing and to use different criteria for grout mix choice. None of the methods were adapted to achieve a better choice in a certain rock mass.

The differences and likeness between method MS-1 and method MS can be exemplified as:

- Method MS-1 stops grouting earlier than MS, due to the minimum stop flow.
- Method MS-1 is more often started with a grout of a lower viscosity than method MS.

The differences and likeness between method MS-1, MS and MA can be exemplified as:

- Method MS-1 and MS use mainly conventional grout cement, while method MA only uses micro cements.
- Method MS-1 and MS have a shorter hole spacing than method MA.
- Method MS-1 and MS stop grouting earlier than MA, due to the minimum stop flow.
- Method MS-1 and MS commonly start grouting with low viscosity grout mix and then successively thickening the grout mix, while method MA starts with a higher viscosity grout and then thicken the grout mix further.

6.1 Conclusions on joint level

The possibility to seal and the time for sealing varies between the different methods and the different rock masses. For some methods, a critical time to seal the individual joint planes has been noted. For other methods it was not possible to notice a critical time. The critical time is the time for grouting and re-grouting implying that a good sealing effect may have been achieved.

In Figure 6-1, the relationship between grouting time and sealing effect on a joint level is shown. In Figure 6-2, the relationship between re-grouting time and sealing effect on a joint level is shown. Both figures are based on figures 5-11, 5-25, 5-38, 5-51, 5-65, 5-78, 5-91, 5-104, 5-117 from chapter 5.

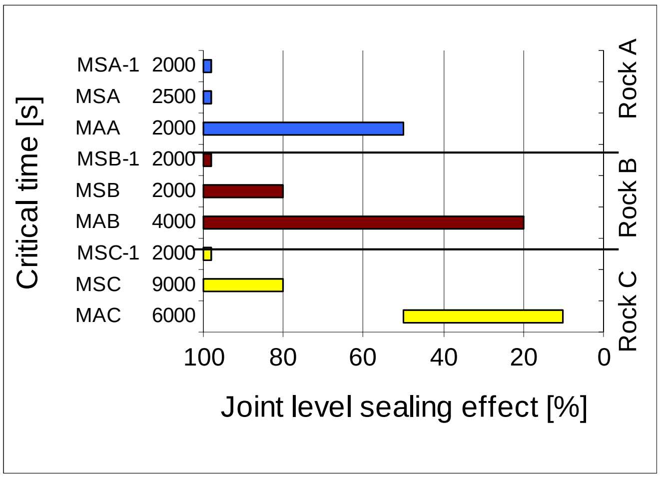


Figure 6-1 Relation between critical grouting time and achieved sealing effect.

It is interesting to notice that for some methods a grouting time or re-grouting time longer than the critical time will always give a good sealing effect. With other words the measurement of the grouting or the re-grouting time can be used for estimation of sealing effect.

For rock mass A shown in Figure 6-1 it could be noticed that, all methods reach the critical time after around 2000 seconds. Method MS-1 and MS imply a high sealing effect and for method MA the sealing effect is between 50 % and 100 %.

For rock mass B, shown in Figure 6-1, it could be noticed that method MS-1 and MS reach the critical time after 2000 seconds. To reach the critical time for method MA takes 4000 seconds. The sealing effect is high for method MS-1, both high and medium for method MS and both high and low for method MA.

For rock mass C shown in Figure 6-1 it could be noticed that method MS-1 gives a high sealing effect after a critical time of 2000 seconds. Method MS can give both high and medium sealing effect, but the critical time is increased, which may be explained by the changed minimum stop flow. For method MA the critical time is long, due to an increased minimum stop flow and the sealing effect is low, due to short penetration lengths and long spacings. Based on Figure 6-1 method MS-1 is to prefer as a grouting method as it achieves the highest sealing effect in shortest critical time.

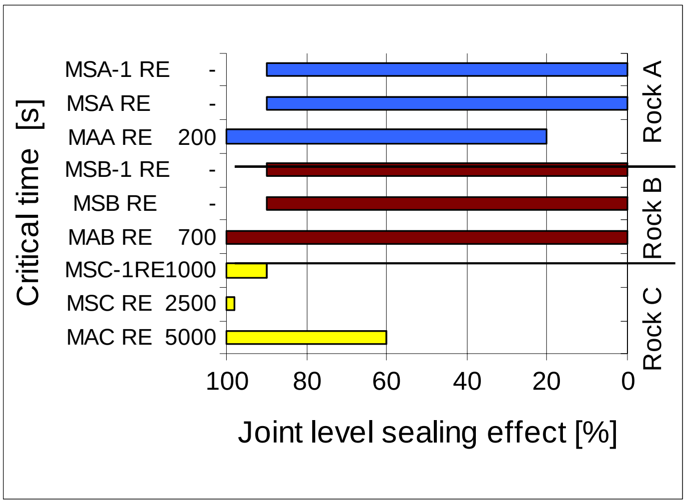


Figure 6-2 Relation between critical re-grouting time and achieved sealing effect.

For both rock mass A and B shown in Figure 6-2, it could be noticed that the sealing effect connected to re-grouting varies largely for all methods and that only method MA shows a critical time for sealing.

For rock mass C shown in Figure 6-2, it could be noticed that all methods show a critical time, and that all methods have the possibility to give a sealing effect close to 100 %. The critical time is shortest for method MS-1 and increases for method MS and MA. The critical time corresponds well to the different stop flow criteria for the different methods.

If a high sealing effect is achieved from grouting, see Figure 6-1, then, no critical time is reached during re-grouting, as shown in Figure 6-2.

Based on Figure 6-2 and Figure 6-1 it may be concluded that a combination of higher minimum flow and a lower viscosity of the grout is to be preferred for grouting.

In a study by Eriksson (2002) the importance of the geometrical factors of the joint and grout mix functionality factors for the sealing effect are summarised, as shown in Figure 6-6.

Table 6-1 Geometrical- and grout mix factors influencing the sealing effect according to Eriksson (2002). ++ = high importance, + = important, - = not important.

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

The geometrical factors and the grout mix functionality factors used within this work are reported in chapter 4 and in the appendix. The rock mass A can, in general, be compared to the column with apertures between 0.1 and 0.2 mm (Table 6-1). The rock mass B can, in general, be compared to the column with apertures above 0.2 mm, as shown in Table 6-1.

The amount of contact area was higher for rock mass B (30 %), than for rock mass A and C (10 %). Based on the calculations of method MS-1:

- The amount of contact does not affect the sealing effect for joints with an aperture of 0.5 mm.
- The amount of contact thus has an influence for joints with an aperture of 0.1 mm.
- The amount of contact influences the sealing effect for the joints with an aperture of 0.2 mm, but this influence is considerably diminished for the joints with an aperture of 0.1 mm.

That the contact area has an influential effect especially for joints with small apertures, as presented by Eriksson (2002), can be verified by the calculations carried out here.

The viscosity of the grout mixes were generally lower for method MS-1 and method MS, than for method MA. None of the grout mixes used were able to affect the sealing effect of the joints with an aperture of 0.5 mm. For method MS-1, which to a higher degree uses grout mixes with a low viscosity, the sealing effect was higher than for the other methods for joints with an aperture of 0.1 mm. For joints with an aperture of 0.2 mm, the difference in viscosity between MS-1 and MS could not be noticed as a difference in sealing effect. For the method MA, which uses a grout mix with a much higher viscosity the sealing effect was decreased. The viscosity dependency on the sealing effect as presented in Table 6-1 can be verified based on these calculations with different grouting methods.

The bleed of the used grout mixes varies between 1 to 56 %. The methods which to a higher degree use grout mixes with high bleed have been favourable regardless of the aperture. The conclusion from Table 6-1, can not be verified from these calculations. However, it should be noticed that the bleed have only been regarded in the joints and that bleed in grout holes, which may influence the sealing effect in a real grouting situation, is left out of this analysis.

The penetrability of the grout mixes varies as different mixes are used within each method. It is therefore not possible to make conclusions about the different values of penetrability of the grouts. The variation of the standard deviation of the aperture and the variation of yield value have not been studied within this work.

6.2 Conclusions on fan level

The different grouting methods, which have been evaluated, have different criteria for re-grouting. In the following, to facilitate the evaluation of different components of the grouting methods, grouting and re-grouting will be compared separately, and not according to the method description.

In Figure 6-3, the average fan grouting time after completion of the grouting is compared for the different methods. From Figure 6-3 it could be noted that the average fan grouting time is increased by increased hydraulic conductivity of the rock mass for all grouting methods. The average fan grouting time is, in general, shortest for method MA and highest for method MS.

The variance of the average fan grouting time was in general decreased by increased hydraulic conductivity of the rock mass for all methods, as shown in Figure 6-3. A higher hydraulic conductivity of the rock mass is, for the rock masses used here, equal to a higher joint intensity. A higher joint intensity increases the possibility to intersect a conductive part of a conductive joint plane, as discussed in chapter 3.

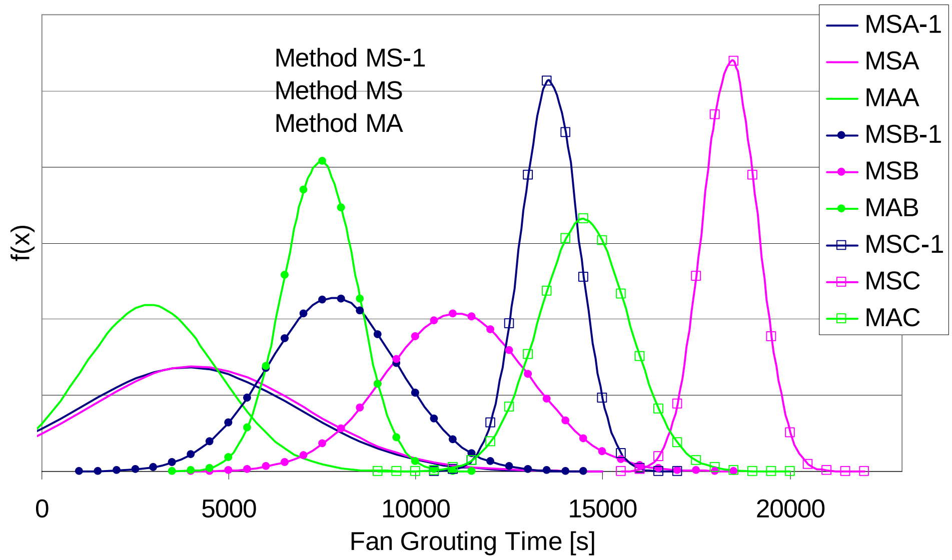


Figure 6-3 Average fan grouting time after first grouting.

The performance of the three methods has in Figure 6-4 been compared as the average fan sealing effect for all rock masses.

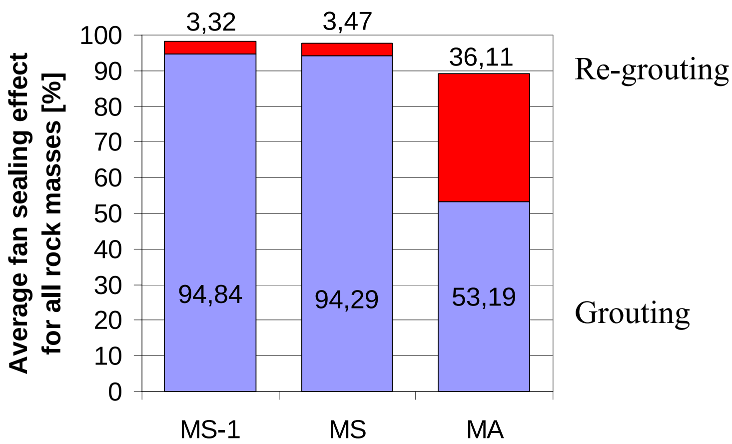


Figure 6-4 Total average fan sealing effect after grouting and re-grouting.

As shown in Figure 6-4 both method MS-1 and method MS yield an average total sealing effect after grouting in the region of 95 %. While method MA yield a much lower sealing effect of 53 %. After re-grouting of all fans, the sealing effect is raised to ~98 % for method MS-1 and MS and to ~90 % for method MA. As an average for all rock masses, method MS-1 and MS are regarded as equally good and method MA is regarded as inferior to the other methods.

As shown in Figure 6-5 the highest sealing effect (99.4 %) was reached after the first grouting for method MS-1 in rock mass B, as shown in Figure 6-5. The corresponding average fan grouting time was 7789 seconds which is similar to the time for method MA with 7439 seconds. The corresponding sealing effect for method MA, was much lower (64.9 %).

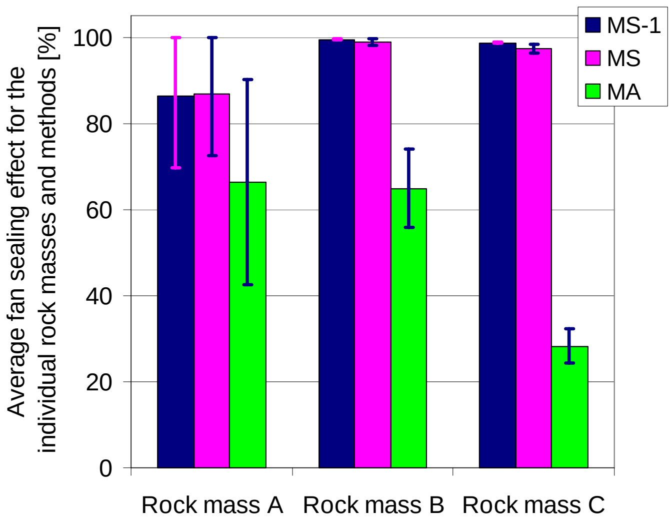


Figure 6-5 Average fan sealing effect with +/- 1 standard deviation after grouting.

In the highly conductive rock mass C, the method MS-1 gives the highest sealing effect in the shortest time, as shown in Figure 6-5. For the same rock mass, method MA gives the lowest sealing effect and consumes more time than method MS-1.

To compare the sealing effect for a method in different rock masses should be done with some care, as the sealing effect needs to be related to the initial rock mass hydraulic conductivity and ground water pressure, which was described in chapter 2.

The sealing effect for method MS-1 and MS are similar and was increased by increased hydraulic conductivity of the rock mass. Although the sealing effect is highest for the medium conductive rock mass B, which may be explained by wide enough joints for penetration, but not too wide as may be the case in rock mass C, where sealing may be more affected by e.g. grout separation. A low viscosity of the grout and an earlier stop of grouting is therefore regarded here as advantageous.

The sealing effect for method MA was decreased by increased hydraulic conductivity of the rock mass, as shown in Figure 6-5. The explanation may be that method MA has a shorter penetration length due to a higher viscosity. In combination with a longer hole spacing compared to the other methods, the result may be an increased number of unsealed joints as the number of joints increases with increased hydraulic conductivity.

The lowering of the minimum stop flow has, within these calculations, not proved to fully compensate for the higher viscosity of the grout mix. Consequently, the grouting time was in general shorter for method MA. The positive effect of a lower stop flow and a decreased separation was not sufficient to raise the sealing effect to the levels achieved by method MS-1 and MS.

In chapter 5, the probability to exceed different inflow requirements for a fan level were presented. For a tunnel, the inflow requirement can be fulfilled by sealing different sections to different degrees. For comparison the inflow requirement for rock mass A is set to 10 litres/min/100 meters, for rock mass B to 20 litres/min/100 meters and for rock mass C to 200 litres/min/100 meters. As shown in Table 6-2, based on full re-grouting of all fans, method MS-1 has a good chance to fulfil the requirements in rock mass A and B and a moderate chance in rock mass C. Method MS has a good chance in rock mass A and B, and method MA has a small chance to fulfil the requirement in any of the rock masses.

Table 6-2 Probability to fulfil inflow requirements based on grouting and full re-grouting.

	Probability to fulfil requirements for a fan [%]		
	MS-1	MS	MA
10 liters/min/100 meters, Rock mass A	84	90	15
20 liters/min/100 meters, Rock mass B	100	90	0
200 liters/min/100 meters, Rock mass C	40	0	0

6.3 Conclusions on tunnel level

The grouting time and the sealing effect based on the nine combinations of method and rock mass for a tunnel consisting of 20 fans is shown in Figure 6-6. Both the grouting time and the sealing effect have, for each of the nine combinations, been calculated in three stages:

1. After grouting
2. After grouting and re-grouting based on control holes
3. After grouting and re-grouting of all holes

For method MS-1 and MS, re-grouting is prescribed based on water loss measurements in control holes. For method MA, no re-grouting is prescribed. Even though, control holes and re-grouting have, in the following analysis, been performed for all methods, to show the effect of control holes and re-grouting.

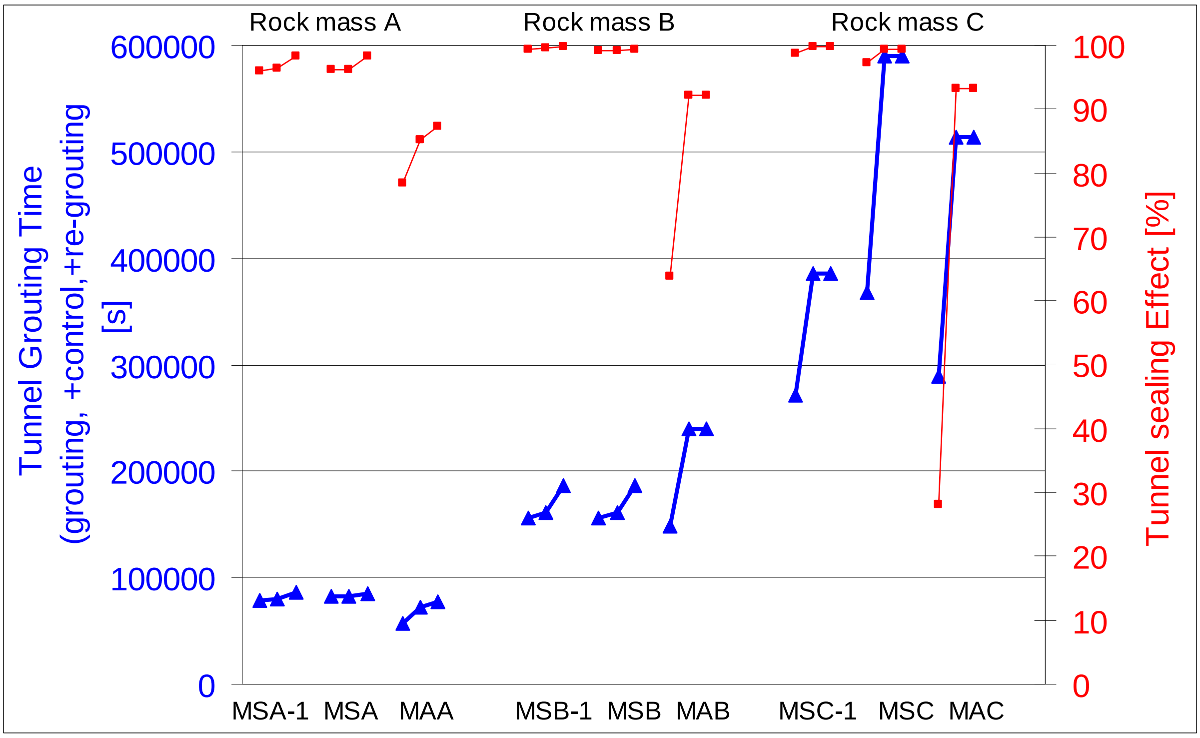


Figure 6-6 Tunnel grouting time and sealing effect for:
1. grouting, 2. grouting + re-grouting based on control holes
3. grouting + re-grouting of all holes.

The grouting time increases with increased hydraulic conductivity (rock mass: A⇒B⇒C) for all grouting methods, as shown in Figure 6-6.

Method MS-1 and MS, has a higher sealing effect then MA and could from this perspective, independent of type of rock mass, be regarded as a better performing method. Method MA, has in general a shorter grouting time than method MS-1 and MS and could from this perspective, be regarded as a better performing method.

The criterion for re-grouting was fixed to 0.15 Lugeon for all rock masses, as measured in control holes. Based on Figure 6-6, the above defined control hole criteria, as a method for deciding about re-grouting, is regarded as less appropriate for "normal" rock mass A and B, ($K=10^{-5} - 10^{-7}$) with sealing effects after grouting above 90 %. If the sealing effect after grouting instead was below 90 % the control hole criteria is more appropriate.

Based on Figure 6-6, the above defined control hole criteria, as a method for deciding about re-grouting, is regarded as appropriate for the highly conductive rock mass C, ($K=10^{-4}$) for all results of sealing effect after grouting.

Based on the above, the control hole criterion as defined here, is able to separate between a very conductive and a less conductive rock mass and to separate between high- and low performing grouting methods, but it has not shown appropriate to suggest re-grouting where re-grouting is most needed.

The increase of the grout volume was studied between the grouting and the re-grouting based on control holes for all methods, as shown in Table 6-3. The volume increase was noted to between 0 and 117 percent for the different methods. The value of 117 % represents re-grouting in 20 out of 20 fans in the very highly conductive rock mass C. For some methods, the re-grouting volume is very low because the more conductive parts of the joint planes have already been sealed and because very few fans have been re-grouted due to the control hole criterion. For other methods the re-grouting is very high because the conductive parts of the joint planes have not been reached earlier and because the control holes suggest re-grouting of all fans.

Table 6-3 Increase of grout volume from grouting to re-grouting on a tunnel level.

[%]	Rock mass A	Rock mass B	Rock mass C
MS-1	0.6	0.2	0.9
MS	0.0	0.2	6.7
MA	12.9	60.1	117.3

In the highly conductive rock mass C, method MS-1 has the shortest grouting time and almost the highest sealing effect (97 %), as shown in Figure 6-6. Low viscosity and close spacing are therefore regarded as advantageous for a highly conductive rock mass.

If the inflow after grouting and after re-grouting reported in chapter 5 are compared to the, by control holes suggested, re-grouting in rock mass A and B. Then the control holes gave a 36 % chance to choose re-grouting to the correct fans (the fans which would gain most on re-grouting). For rock mass C, the control holes showed a 100 % chance to chose re-grouting to the appropriate fans, which is equal to re-grouting in all fans due to the very high inflows of rock mass C.

6.4 Performance in three rock masses

Earlier in this chapter, the performances of methods have been reported after grouting and re-grouting independently. In the following the effect of re-grouting is added to the effect of grouting as described by the grouting method.

The three methods have been calculated for three different rock masses. The following analysis will therefore focus on how the different methods perform based on a specified rock mass.

For all methods the grouting volume was increased with increasing hydraulic conductivity of the rock mass.

6.4.1 Rock mass A

Based on tables 5-2, 5-4 and 5-6, the average fan time is shown in Figure 6-7. The fan times shown are based on grouting times and the method dependent re-grouting times.

Method MS-1 and MS finishes on equal time with almost equal variance. Method MA finishes on a shorter time than MS-1 and MS and with less variance. The explanation may be that method MA uses a grout with a higher viscosity than MS-1 and MS and that the minimum stop flow was lower for MA than for MS-1 and MS. The result may then be a shorter penetration length and a slower penetration speed, which also gives lower spreading complexity (see chapter 3.3.2) with a lower variance for method MA.

The sealing effect for MA is only 66 % which should be compared to the sealing effect for MS-1 of 86 % and for MS of 87 %, as shown in Figure 6-7. If the sealing requirement is low method MA may be used, if a higher sealing effect is required, method MS-1 can be used. The fan time will then increase with ~40 % and the grout consumption with an average of ~13 times compared to method MA.

The average consumption of grout is much lower for MA (30 litres per fan) than for MS-1 (371 litres per fan) and for MS (410 litres per fan). Low grout consumption implies a short fan time and a low sealing effect, as shown in Figure 6-7.

Based on the combination of fan time and sealing effect, the method MS-1 may, depending on the requirements, be the choice for rock mass C.

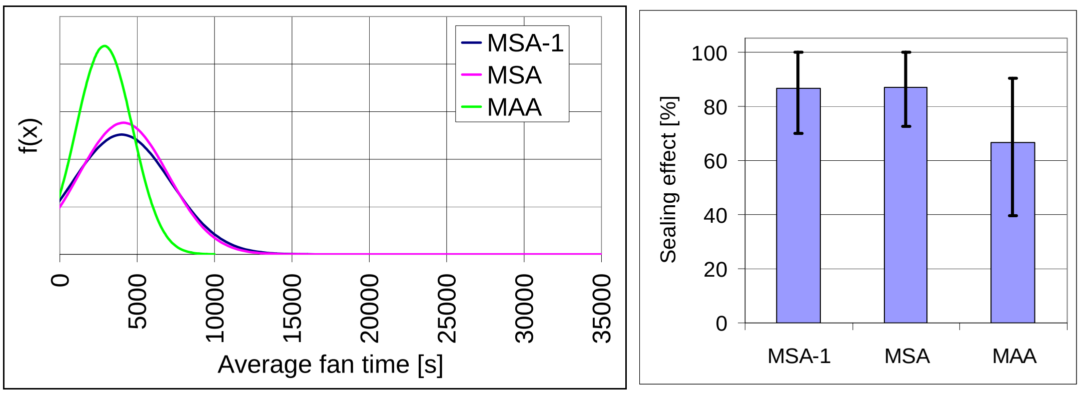


Figure 6-7 Time for grouting and re-grouting with corresponding sealing effect based on the specified method in rock mass A.

Based on different figures presented in chapter 5, the following could be noted.

The re-grouting volume was higher for method MA, than for MS-1 and MS and one explanation is the lower grouting volume. The small apertures sealed by MS-1 and MS have a relatively high re-grouting volume compared to the larger apertures. For method MA, the small apertures were in line with the larger apertures. This may be an effect of a lower yield value for the grout mixes used for method MS-1 and method MS, which allowed a further penetration of the fine joints.

The single wide joint in fan number 20, was well sealed by the grouting or re-grouting with method MS-1 and MS, but not by the grouting or re-grouting with method MA. The methods MS-1 and MS are therefore regarded as a better choice for sealing of wide apertures than method MA in rock mass A.

6.4.2 Rock mass B

Based on tables 5-8, 5-10, 5-12, the average fan time is shown in Figure 6-8. The fan times shown are based on grouting times and the method dependent re-grouting times

Method MA finishes on shortest fan time and with smallest variance. The explanation may be due to a higher viscosity of the grout and a higher yield value of the grout for method MA than for method MS-1 and MS. In addition the number of grout holes was lower for method MA than for method MS-1 and MS. The minimum stop flow was lower for method MA, but this could not compensate for the increased viscosity, the increased yield value and the decreased number of grout holes.

Method MS-1 has, average, a shorter fan time than MS, but they have equal variances. The longer fan time for MS, may be because the minimum stop flow was higher for MS-1 than for MS.

The sealing effect for MA is only 65 % which should be compared to the sealing effect for MS-1 of 99.5 % and for MS of 98.8 %. If the sealing requirement is low method MA may be used, if however a higher sealing effect is required, method MS-1 should be used. The fan time will then increase with ~8 % and the grout consumption with an average of ~16 times compared to method MA.

The sealing effect variance, as illustrated in Figure 6-8, is higher for MA than for MS-1 and MS. The high sealing effect for method MS-1 and MS, may be explained due to the fact that the rock mass is open enough to enable penetration and not to open which may result in separation of the grout. For method MA the low sealing effect may, as in rock mass A, be due to short penetration distances as a result of the higher viscosity, a higher yield value and too large spacing between holes in relation to the penetration length.

The average consumption of grout is much lower for MA (158 litres per fan) than for MS-1 (2559 litres per fan) and for MS (2943 litres per fan). Low grout consumption implies a short fan time and a low sealing effect, as shown in Figure 6-8.

Based on the combination of fan time and sealing effect, the method MS-1 may, depending on the requirements, be the choice for rock mass B.

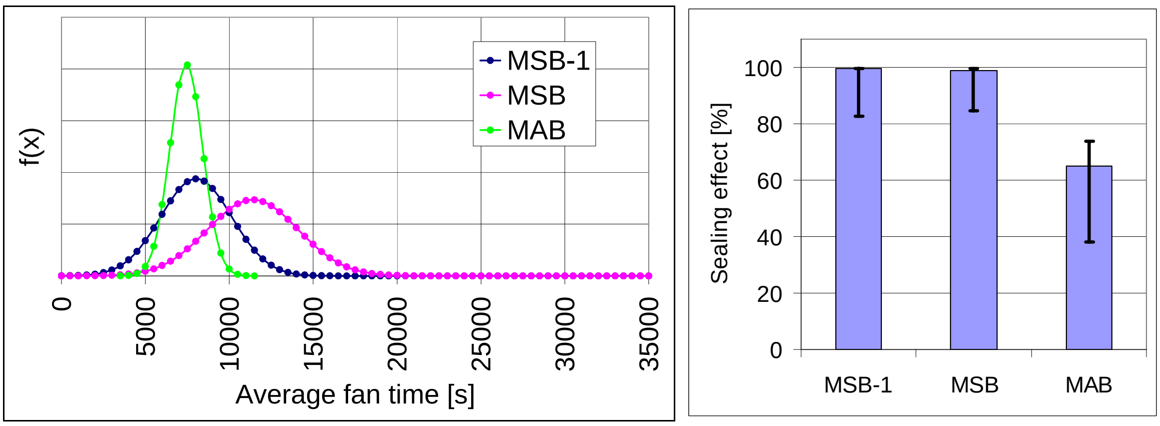


Figure 6-8 Time for grouting and re-grouting with corresponding sealing effect based on the specified method in rock mass B.

6.4.3 Rock mass C

Based on tables 5-14, 5-16, 5-18 the average fan time is shown in Figure 6-9. The fan times shown are based on grouting times and the method dependent re-grouting times.

Method MA displays the shortest average fan time and the lowest variance. Method MS-1 finishes on a longer fan time than MA, but shorter than MS. The explanation for why method MS-1 has shorter fan times, may be related to the higher minimum stop flow criterion. On the other hand side, method MS-1 more often uses a grout with lower viscosity than MS which should increase the time.

The sealing effect for method MA is only 28 % which could be compared to the sealing effect for method MS-1 of 99.7 % and for method MS of 99.3 %. The sealing effect variance, as illustrated in Figure 6-9, is higher for method MA than for method MS-1 and method MS. The high sealing effect for method MS-1 and MS, may as reported for rock mass B, be because the rock mass is open enough to enable penetration and not too open which may result in a lower sealing effect from separation of the grout. For method MA, the low sealing effect may be due to short penetration distances as a result of the higher viscosity, the higher yield value and a too large hole spacing in relation to the penetration length. Method MA may not, depending on the requirement, be regarded as a suitable method for sealing rock mass C.

The average consumption of grout is much lower for method MA (1.9 m^3 per fan) than for MS-1 (74 m^3 per fan) and for MS (75.5 m^3 per fan). Low grout consumption implies a short fan time and a low sealing effect, as shown in Figure 6-9.

Based on the combination of fan time and sealing effect, the method MS-1 may, depending on the requirements, be the choice for rock mass C.

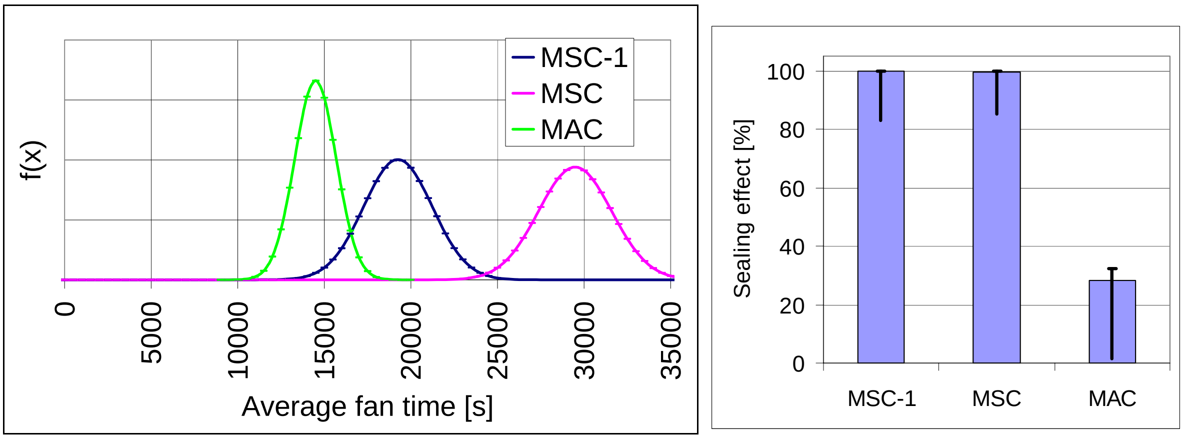


Figure 6-9 Time for grouting and re-grouting with corresponding sealing effect based o the specified method (Rock mass C).

6.5 Miscellaneous conclusions

Based on the properties used for the calculations, it is concluded that a decreased grout viscosity and decreased yield value of the grout can not always be fully compensated by a decreased minimum stop flow (MS-1 to MS).

A relationship is implied between the grouting time and the re-grouting time for method MA, but not for the other methods. The explanation may be, that the shorter penetration length and the large hole spacing of method MA, result in a rock mass, surrounding the re-grouting holes, that could be regarded as untreated by the original grouting.

Based on the calculation of cross joint flow for method MS-1 and MS and for all rock masses, grouting was more effective than re-grouting, which may be because most joints already were sealed or partly sealed by the prior grouting.

Based on the calculation of cross joint flow for all rock masses, method MA gains most on re-grouting, but still the inflow is much higher than for method MS-1 and MS, which may be an effect of a shorter penetration length for the grout mixes used with method MA. Due to the shorter penetration length, method MA may leave joints unaffected after grouting and the high sealing effect after re-grouting may therefore refer to sealing of those unaffected joints.

A high grout separation has not been proven to decrease the sealing effect in the calculated rock masses. Therefore, separation in grout holes has not been considered within the calculations.

In chapter 2 the relationship between the degree of grouting difficulty, inflow requirements and the initial inflow were discussed. The degree of difficulty has been evaluated for a circular tunnel with a diameter of 10 metres. All calculations in this thesis are based on the assumption of such a tunnel.

Based on the Figure 2-1, by Bergman & Nord (1982), the difficulty of grouting for method MS-1 in rock mass A is shown in Figure 6-10. The degree of difficulty as presented by Bergman & Nord (1982), can be verified by the numerical calculations of method MS-1 in rock mass A. The verification assumes that a grouting method, which has the possibility to seal, is chosen.

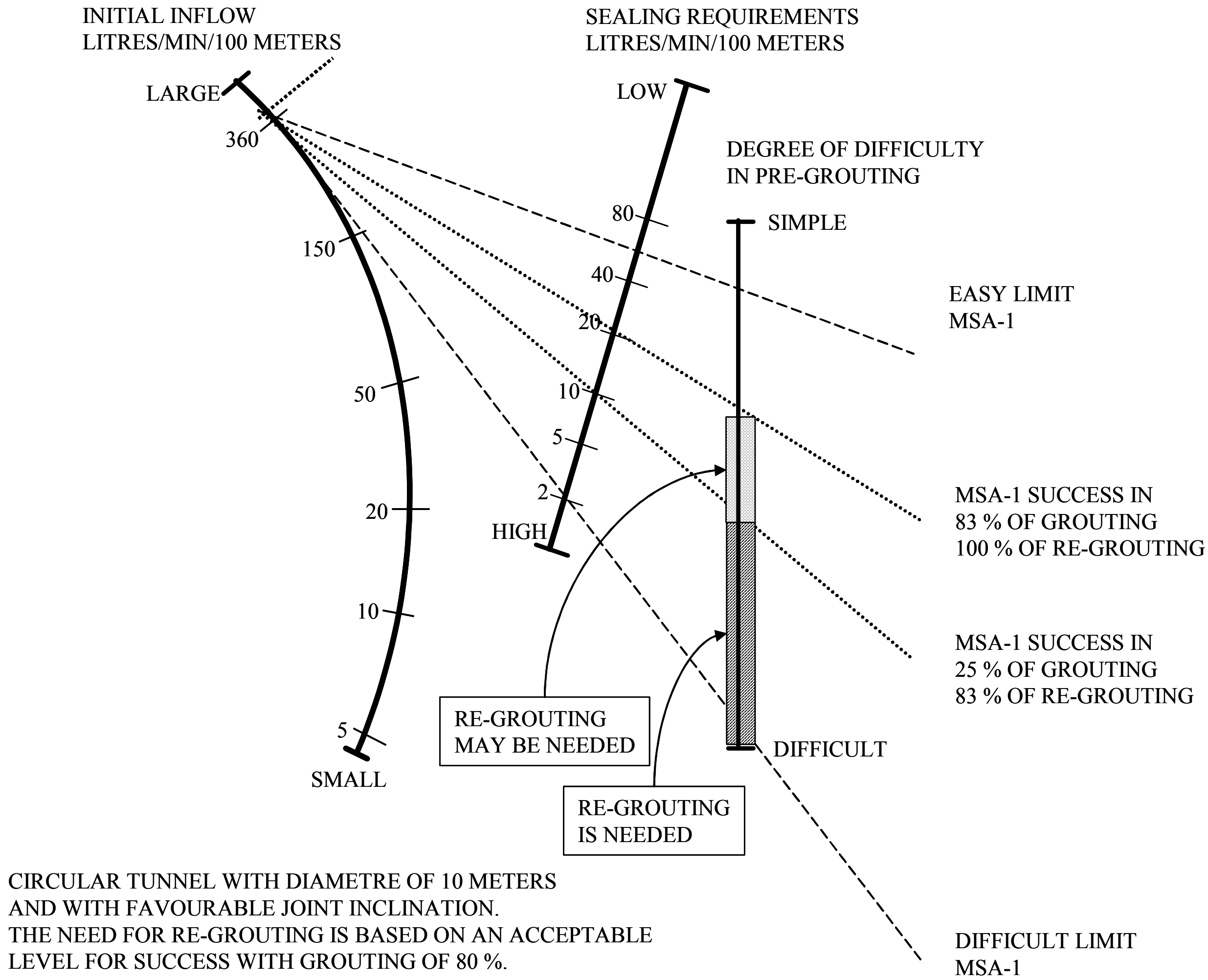


Figure 6-10 Relationship between initial inflow, Sealing requirements and degree of difficulty in pre-grouting (based on Bergman & Nord, 1982).

In chapter 5 the probability of failing with grouting were calculated for grouting and re-grouting. A success criterion is defined here. If at least 80 % of the grouting fans fulfil the inflow requirement without the need of re-grouting, then success is achieved. Based on this success criterion and the results from method, MS-1 and MS in rock mass A and B, the limits between grouting and re-grouting can be estimated as shown in Figure 6-10. It should be noticed that the limits in Figure 6-10 have been evaluated for the better performing grouting methods, for a less effective grouting method the limits for re-grouting would be higher.

Chapter Seven

7 Sealing time calculation and grouting optimization

7.1 Introduction

By evaluating the performance of different grouting methods it may be possible to choose a suitable grouting method for the specific rock mass situation and thereby from this respect optimize the grouting process.

Evaluation of the different grouting methods has resulted in e.g. different grouting times, different sealing effects and different inflows after grouting, as presented in chapter 5.

A model for calculating the sealing time was developed in chapter 4. The model regards the different activities which are carried out in order to seal a rock mass. In this chapter the sealing time model is used to calculate the sealing time and the sealing time variance for the three evaluated grouting methods.

7.2 Sealing time

According to chapter 4, the sealing time is calculated for each grouting method for a tunnel length of 500 meters. The sealing time for a fan is calculated as the sum of the different activities of each grouting method. The sealing time for a tunnel is then calculated as the sum of a number of fans and when calculating the sealing time variance for the tunnel, theories of variance reduction discussed in chapter 4, have been applied.

The time while waiting for the grout to harden was discussed in chapter 4. As shown in Table 7-1 the waiting time is the single most important factor which affects the sealing time. This emphasizes the importance of the discussion from chapter 4, of using an appropriate hardening time for the different grout mixes. In Table 7-1, the waiting time is calculated based on a risk of flushing out the grout and that the shear strength to resist flushing of the grout needs to be at least 12 kPa. If there is no risk of flushing, the waiting time presented in Table 7-1 could be reduced.

After the waiting time, the drilling time and then the grouting time are the most important factors for the total sealing time. The grouting and re-grouting are calculated as single hole grouting. For a real situation the time for grouting and re-grouting could be further shortened by using simultaneously grouting of more than one grout hole.

The time of establishment is based on the number of establishments used within each method. For method MS and MS-1 the number of establishments has been set to 3, which regards one for drilling, one for water loss measurement and grouting and finally one for drilling and measuring the water loss in control holes. If it is not possible to combing them as described here, the time for establishment may need to be increased. For method MA, the number of establishments has been set to 2, one for drilling and one for grouting, which may also need to be increased if the assumed combination not is possible.

The time for post-grouting has been left out of the sealing time calculation carried out here, as it is not known if post-grouting is a critical factor or not.

Based on Equations 4-23 and 4-24 from chapter 4, the total sealing time and the time for the individual activities are shown in the Table 7-1, for a tunnel length of 500 meters which is equivalent to 25 fans.

Table 7-1 The time for the different activities of the sealing time equation, for 9 combinations of grouting method and rock mass.

		t_d Drilling mean 80 m/h std 7 m/h	t_p Waterloss mean 8 min std 2.5 min	t_p Controll, drill mean 80 m/h std 7 m/h	t_p Controll, wlm mean 8 min std 2.5 min	t_e Establismment mean 33 min std 12 min	t_w Waiting mean 10h std 2h	t_{g+rg} Time for pump. Grouting+ Re-grouting	t_s Total time [mean h/fan] [stdev h/fan]
Rock mass A	**MSA-1**								
	Antal	26.0	26.0	4.0	4.0	3.0	21.0	21.0	
	Mean[h]	6.8	3.6	0.8	0.6	1.7	10.5	1.1	25.2
	Standard[h]	0.7	0.6	0.1	0.1	0.6	2.1	0.9	2.5
	MSA								
	Antal	26.0	26.0	4.0	4.0	3.0	20.0	20.0	
	Mean[h]	6.5	3.5	0.8	0.5	1.7	10.0	1.1	24.0
	Standard[h]	0.7	0.6	0.1	0.1	0.6	2.0	0.8	2.4
	MAA								
	Antal	17.0	0.0	0.0	0.0	2.0	20.0	20.0	
	Mean[h]	4.3	0.0	0.0	0.0	1.1	4.5	0.8	10.7
	Standard[h]	0.4	0.0	0.0	0.0	0.4	0.9	0.5	1.2
Rock mass B	**MSB-1**								
	Antal	26.0	26.0	4.0	4.0	3.0	26.0	26.0	
	Mean[h]	8.5	4.5	1.0	0.7	2.1	13.0	2.2	32.0
	Standard[h]	0.8	0.8	0.1	0.1	0.8	2.6	0.6	3.0
	MSB								
	Antal	26.0	26.0	4.0	4.0	3.0	21.0	21.0	
	Mean[h]	6.8	3.6	0.8	0.6	1.7	10.5	3.2	27.2
	Standard[h]	0.7	0.6	0.1	0.1	0.6	2.1	0.8	2.5
	MAB								
	Antal	17.0	0.0	0.0	0.0	2.0	20.0	20.0	
	Mean[h]	4.3	0.0	0.0	0.0	1.1	4.5	2.1	11.9
	Standard[h]	0.4	0.0	0.0	0.0	0.4	0.9	0.3	1.1
Rock mass C	**MSC-1**								
	Antal	26.0	26.0	4.0	4.0	3.0	40.0	40.0	
	Mean[h]	13.0	6.9	1.5	1.1	3.3	20.0	5.3	51.1
	Standard[h]	1.3	1.2	0.2	0.2	1.2	4.0	0.6	4.6
	MSC								
	Antal	26.0	26.0	4.0	4.0	3.0	40.0	40.0	
	Mean[h]	13.0	6.9	1.5	1.1	3.3	20.0	8.2	54.0
	Standard[h]	1.3	1.2	0.2	0.2	1.2	4.0	0.6	4.6
	MAC								
	Antal	17.0	0.0	0.0	0.0	2.0	20.0	20.0	
	Mean[h]	4.3	0.0	0.0	0.0	1.1	4.5	7.1	17.0
	Standard[h]	0.4	0.0	0.0	0.0	0.4	0.9	0.5	1.2

In Table 7-1, the grouting and re-grouting times are based on figure 6-7, 6-8 and 6-9, other values in the table e.g. drilling-, waiting- and establishment time are based on data given in chapter 4. The grouting and re-grouting time have been added to a total grouting time, t_{g+rg}, according to the definition of each grouting method.

The average sealing times and the corresponding variance presented in Table 7-1 are then used to calculate the sealing time distributions. As data in Table 7-1 are influenced by many small and unrelated random effects they are approximately normally distributed. The central limit theorem was briefly explained in chapter 4.

The sealing time can then be calculated using Equation 4-27 and 4-28 in chapter 4. The result is then presented as normal distributions for the characteristic regions with a length of 500 meters, as shown in Figure 7-1 In the figure the corresponding inflow to the tunnel is also shown. The inflows are based on figures 5-13, 5-27, 5-40, 5-53, 5-67, 5-80, 5-93, 5-106, 5-119 in chapter 5. The distribution of the inflow has as well been based on the theory of variance reduction, since the inflow to the tunnel reach is also a sum of independent elements.

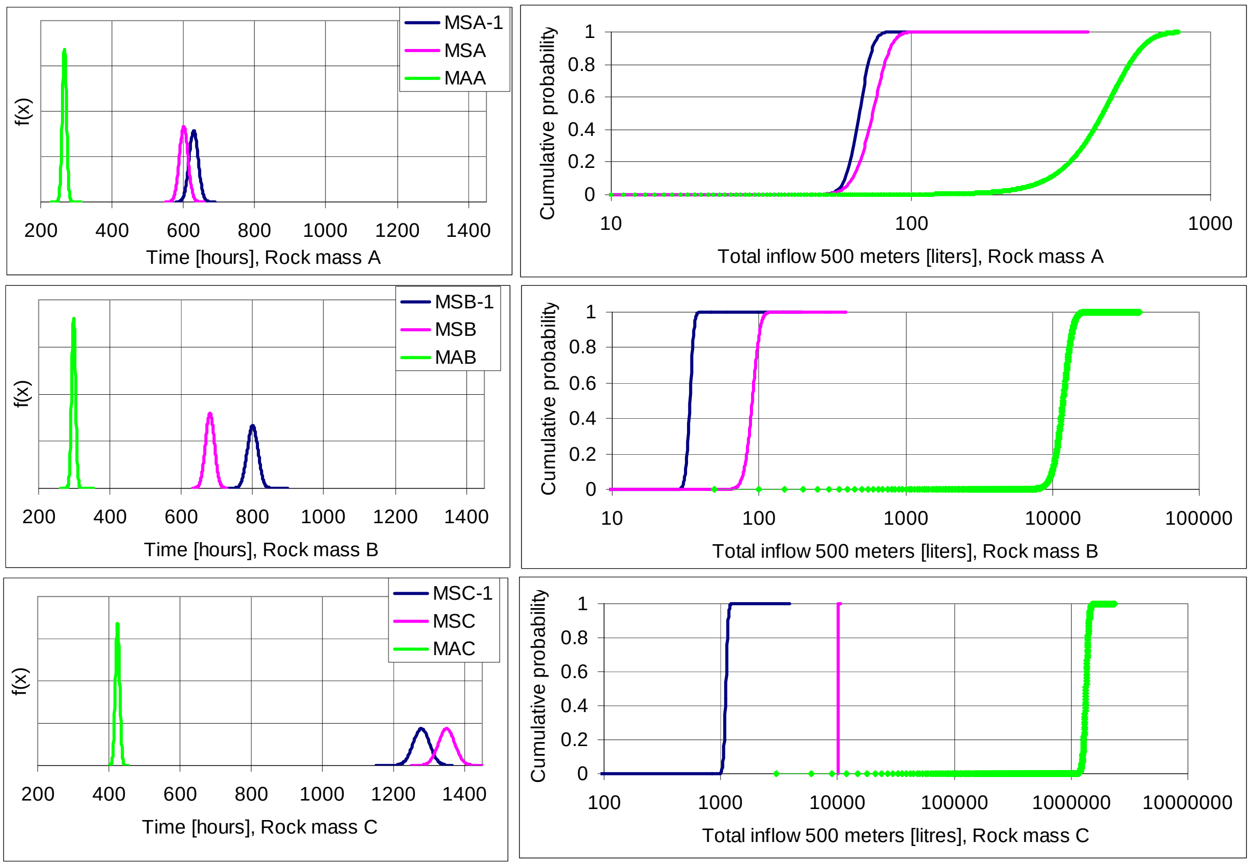


Figure 7-1 Sealing time and corresponding inflow for the three grouting methods in a tunnel with a length of 500 meters.

As shown in Figure 7-1 the sealing time is highest for Method MS-1 and shortest for Method MA. At the same time the inflow is highest for method MA and lowest for method MS-1. The very high inflows noticed in Figure 7-1 depends for example on the fact that:

- The conductivity of the rock masses B and C are very high.
- The grout mix has a poor penetration of joints less than 100 µm.
- The calculated rock masses include finer joints than 100 µm.
- The model is defined by 30 x 30 m^2 joint planes and do not regard "skin" effects.

For all three rock masses the inflow has been calculated for a length of 500 meters. For a real situation this could be an appropriate assumption for rock mass A, and to some extent for rock mass B. For rock mass C, the length 500 meters is not appropriate. The rock mass C has a conductivity which could be compare with a highly crushed zone, which may extend a few meters. This fact should be regarded when comparing the high inflows in Figure 7-1.

Based on the Figure 7-1 it could be noted that method MA is a fast grouting method which may be suitable for situations with a low water head and for situations that allow a high water inflow. Method MA may also be suitable for situations when a fast grouting method is preferable due to e.g. productivity and when sealing can later be complemented with e.g. post-grouting.

Method MS-1 and MS are both more time consuming than method MA, at the same time they reduce the water inflow to much higher extent. Method MS-1 and MS are therefore regarded as a better choice than MA if reducing the inflow is more important than a short sealing time.

As show in Figure 7-1, Method MS-1 gives a higher inflow reduction in all rock masses compared to MS. Method MS-1 consumes more time than MS for all rock masses except rock mass C. The use of a low viscosity grout and an earlier stop is therefore regarded as advantageous in a highly conductive rock mass.

7.2.1 Discussion

Three different methods and three different rock masses have been evaluated. By using the suggested system for evaluation it was possible to separate the performance of the different methods.

Each specific tunnel project has specific requirements. Depending on the requirements, the sealing time may or may not be a critical factor, as earlier discussed in chapter 4.3. For a project which incorporates many tunnels, the number of tunnel faces where pre-grouting can be performed may be higher than for a project with only one tunnel, if the latter is only excavated from one direction. With many tunnels, sealing time may be a less critical factor.

If sealing time is not a critical factor, there is a linear relationship between sealing time and cost. As regards other activities in the project, pre-grouting may not be acceptable to carry out beyond a critical sealing time. The relationship between sealing time and cost may then be no longer linear, as illustrated in Figure 7-2. Pre-grouting is commonly more effective in reducing the water inflow, than post-grouting, which is one reason why costs are increased after the critical time.

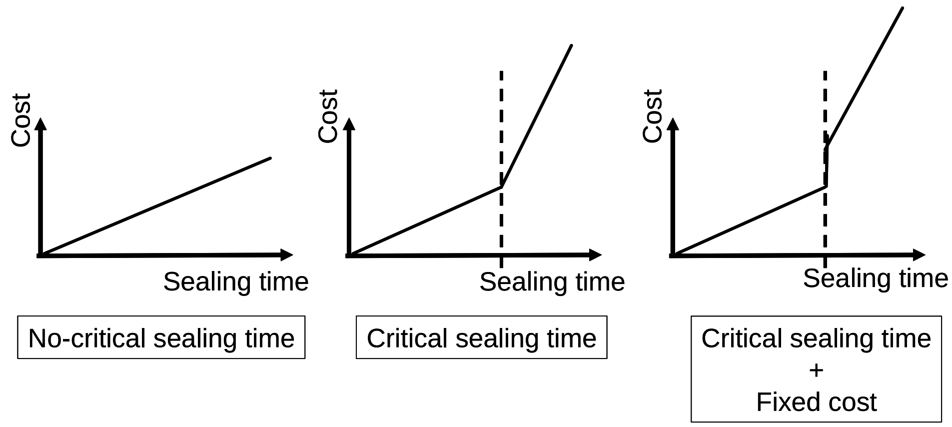


Figure 7-2 Principal relationship between sealing time and cost.

Based on the relationship between sealing time, costs and critical sealing time, the choice of grouting method may be regarded as a decision problem, as illustrated in Figure 7-3 and discussed in chapter 4.

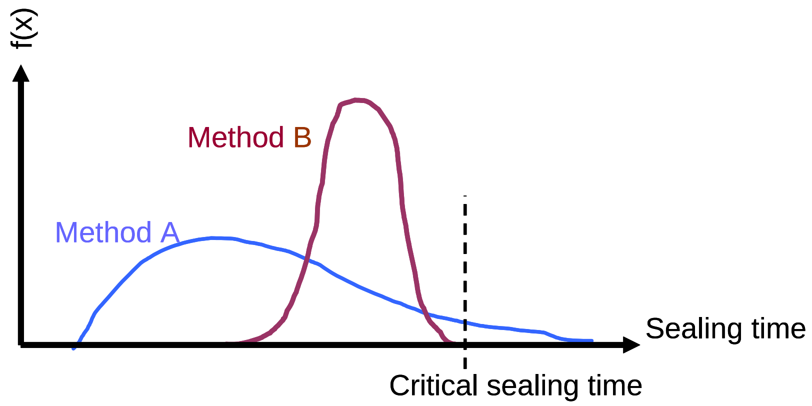


Figure 7-3 Choice of pre-grouting method as a decision problem.

In Figure 7-3, the sealing times for method A and B, which both have the possibility to fulfil the inflow requirement, are shown. Method A has a low average time and a high variance. Method B has a high average time and a low variance. If sealing time is not a critical factor, there is a linear relationship between sealing time and cost and the average times for the methods could be used to compare them. The expected sealing time for a tunnel consisting of a number of fans will then be lower if method A is chosen. If the sealing time is a critical factor, the calculation of the sealing time needs to regard the non-linear relationship as illustrated in Figure 7-2.

By suggesting and evaluating a number of methods for grouting according to the suggested methodology for evaluation, the sealing time and the inflow can be calculated as presented in Figure 7-1. Then with the method evaluation as a base, a grouting method can be chosen taking into consideration the possibilities and restrictions related to the specific project.

7.3 Suggested concept of optimization

The calculation of sealing time, grouting time and inflow presented in chapter 7 raises a number of questions regarding the possible optimization of the evaluated grouting methods.

In this chapter a theory for optimization of the grouting process, developed by Dalmalm & Stille (2003) will be briefly presented.

7.3.1 General

The methods evaluated in chapter 7 have fixed parameters. For some parameters, such as the number of grout holes, the optimum number may actually be higher than this chosen value for method MA and lower for method MS-1.

Method MS-1 and MS are in general very similar, with only some small differences. Those small differences have resulted in large difference for some of the outcomes. As an example, the inflow after grouting and re-grouting in rock mass B for method MS was twice the inflow of method MS-1, as shown in Figure 7-1.

The reason why small changes between MS-1 and MS can make these large differences may be explained as: A more suitable grout mix for method MS-1 and an earlier stop criterion have resulted in decreased inflow and decreased grouting- and re-grouting time for rock mass B. Therefore the grouting and re-grouting times refer to times presented in Table 7-1 and the inflows refer to inflow presented in Figure 7-1.

7.3.2 Concept for optimization of the grouting time

To reach a sealing of a joint plane there are two parameters that are of special interest:

- The penetration length of the grout
- The number of grout holes

For different rock masses and for a different number of grout holes, the penetration length of the grout needs to be different, as illustrated in Figure 7-4.

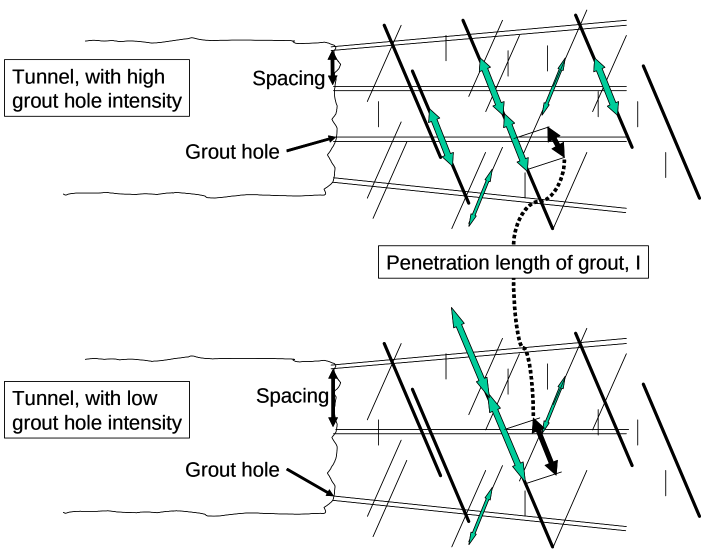


Figure 7-4 The relationship between hole spacing and penetration length of the grout.

The penetration length of the grout depends on the grouting pressure, the aperture and the yield value of the grout, as was shown for a one dimensional case in Equation 3-32. The penetration length is therefore dependent on both the rock mass and the grouting method.

The geometry of a joint is complicated and so is the grout spread in a joint plane. Commonly, the grout spread has, in literature, been described as radial (one dimensional) or canalised (two dimensional). Depending on the geometry, the penetration length will vary. In the following, the grout spread is assumed radial as illustrated in Figure 7-5.

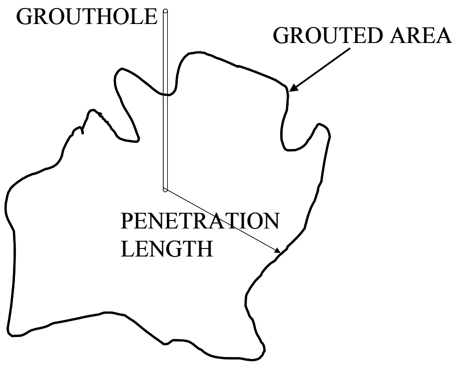


Figure 7-5 Grout spread in a joint plane.

The grout volume V, from spreading of grout in joint planes of the rock mass, could geometrically be calculated according to Figure 7-5 and Equation 7-1. The expression is similar to that presented in Gustafson & Stille (1996).

$$V = I^2 \cdot \pi \cdot n \cdot L \qquad \text{Equation 7-1}$$

Where: I = penetration length [m]
n = porosity [$^o/_{oo}$]
L = effective length of grout hole [m]

The penetration length could then be expressed as:

$$I = \sqrt{\frac{V}{k}} \qquad \text{Equation 7-2}$$

Where k is a constant, $k = \pi \cdot n \cdot L$.

By differentiating, with respect to the time, an expression for the penetration length increase could be noted as:

$$\frac{dI}{dt} = \frac{1}{2} \cdot \frac{1}{\sqrt{k}} V^{-\left(\frac{1}{2}\right)} \cdot \frac{dV}{dt} \qquad \text{Equation 7-3}$$

From practice the grout flow is commonly registered during the grouting process, as illustrated in Figure 7-6.

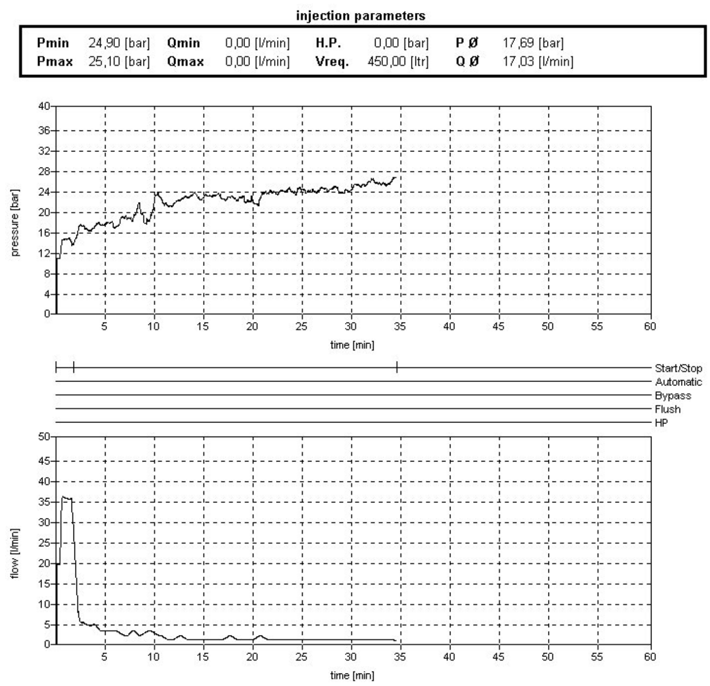


Figure 7-6 Example of pressure and flow records from a grout hole.

If the flow from the grouting records is integrated and the volume from filling of the grout hole is deducted, as shown in Figure 7-7, an expression for the volume *V* as a function of the time *t* could be noted, based on a regression analysis, as:

$$V = 16.566 \cdot 10^{-3} \cdot t^{0.5133} \qquad \text{Equation 7-4}$$

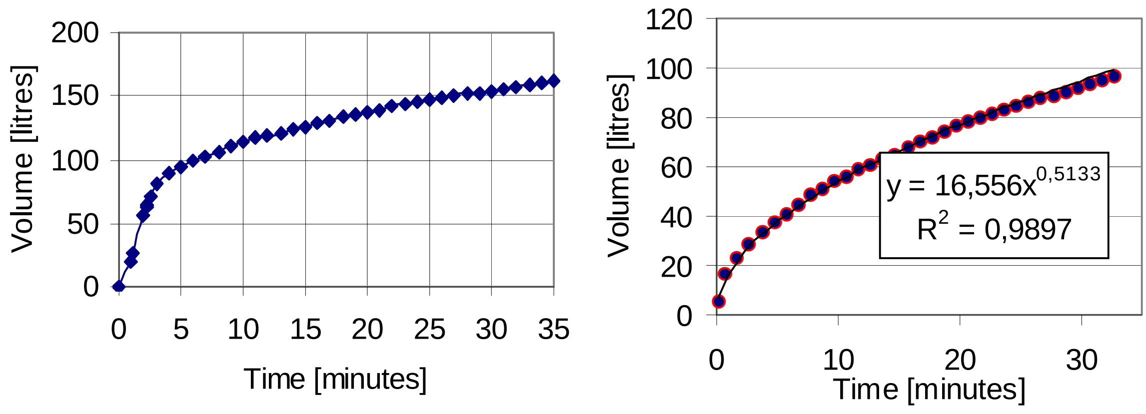


Figure 7-7 (left) Volume increase from the studied grouted hole, (right) Volume increases, with filling of grout holes excluded.

The difference in penetration length could then, based on Equation 7-3 and 7-4, be noted as:

$$\frac{dI}{dt} = \frac{3.30 \cdot 10^{-2}}{\sqrt{k}} \cdot t^{-0.74335} \qquad \text{(Equation 7-5)}$$

If the purpose is to optimise the grouting process, the increase in penetration length could be compared to the initial increase in penetration (after hole filling) as is shown in the semi empirical Equation 7-6,

$$\frac{dI/dt}{dI/dt_0(t=1)} = t^{-0.7434} \qquad \text{(Equation 7-6)}$$

and illustrated in Figure 7-8.

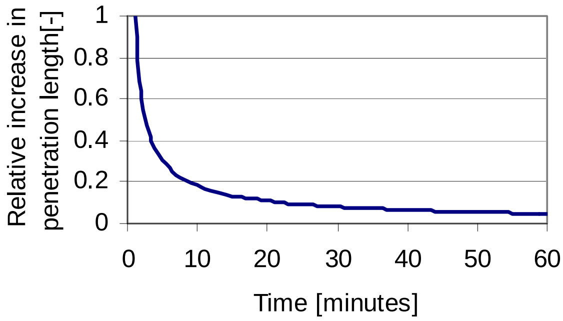


Figure 7-8 Relative increase in penetration length, dI/dt.

From Figure 7-8 it could be noted that the relative penetration speed has been considerably decreased after about 15 minutes of grouting. To calculate the absolute penetration the porosity and the length of the grout hole needs also to be known.

If the porosity of the rock mass is also known, the absolute penetration length increase and the accumulated penetration length could be calculated according to Equation 7-3. In Figure 7-9 the absolute penetration length increase and the accumulated penetration length is shown for a hole length of 20 meters and an assumed porosity of 0.015 $^{0}/_{00}$.

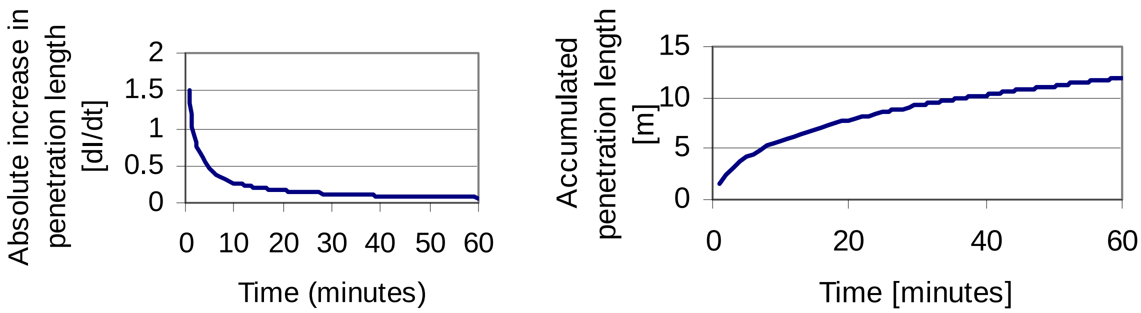


Figure 7-9 The absolute penetration length increase, with 0.015 $^{0}/_{00}$ conductive porosity.

From Figure 7-8 and Figure 7-9 it is noted that both the grouted volume and the grouting time have an influence on the filling of the joint system in a rock mass. As an example, a reduction of the water inflow to a tunnel with approximately 90 % may be carried out in a short time (~15 min.), but to reach the last filling of the joint system usually takes long time.

For an observed situation, the inflow after grouting may be higher than that acceptable from the requirement. The sealing effect reached by grouting needs then to be increased. Two alternatives could be considered:

- To increase the pumping time
- To decrease the hole spacing.

The decision could then be based on the relationship between accumulated penetration length and time as illustrated in Figure 7-10.

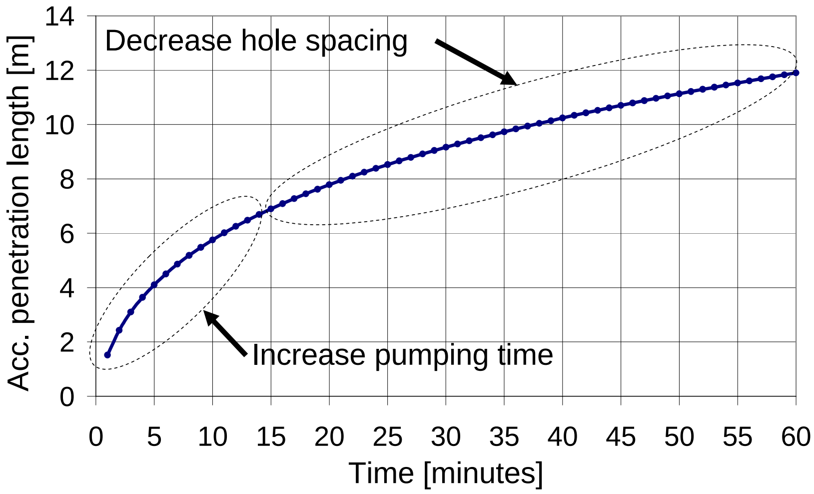


Figure 7-10 The curve of the accumulated absolute increase in penetration length, to be used as a base for decision making.

As shown in Figure 7-10, for a short pumping time, the penetration length is significantly increased by prolonging the pumping time. For a long pumping time, the marginal effect is reduced and it maybe more optimal to reduce the hole spacing and thus the demands on the penetration length.

As an alternative to the semi-empirical relation, Gustafson & Claesson (2004) modelled the radial relative penetration length in a joint plane from a theoretical point of view, as shown in Figure 7-11.

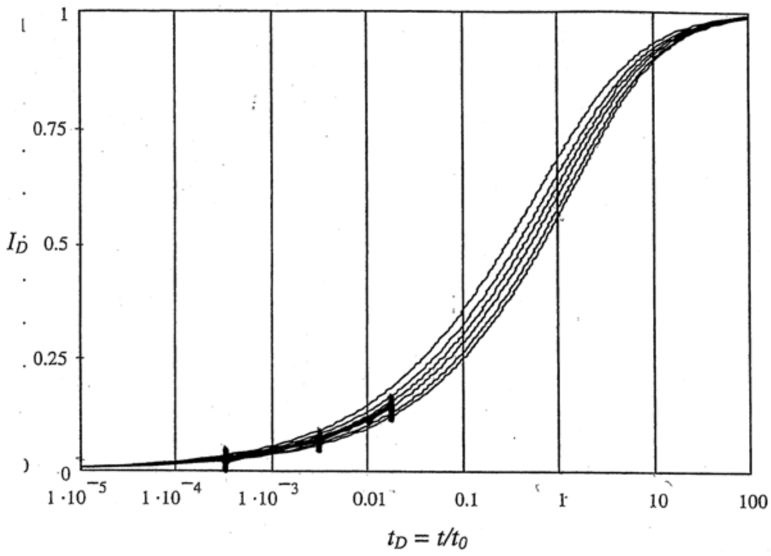


Figure 7-11 Relative grout penetration for radial flow based on Gustafson & Claesson (2004). For different values of $\gamma = I_{max}/r_b$. Where t_d is the dimensionless grouting time, I_D is the relative penetration length, I_{max} is the maximum penetration length and r_b is the radius of the borehole.

Comparison carried out by Gustafson & Claesson (2004) show that the parallel flow is somewhat slower than the radial flow and that it could be regarded as poor economy to continue grouting beyond a certain time.

A preliminary analysis has been carried out and the result given in Figure 7-9 has been recalculated in relative terms according to Gustafson & Claesson (2004). The results have been plotted in Figure 7-11. As noticed the agreement is quite good and that an ordinary grouting operation will never reach full penetration as a result of the stop flow criterion.

The stop flow criterion will give a stop in the grouting operation long before the maximum penetration length has been reached. The theory of Gustafson & Claesson (2004) confirms the semi empirical relation that increasing of the pumping time is only optimal in small relative penetration length.

The semi empirical relation was carried out in rock mass with similar properties as rock mass A. The used grout mix was similar to the grout mixes used for method MS-1 and MS. The corresponding maximum penetration length, $I_{max,}$ was calculated to 90 meters and the corresponding characteristic time, $t_{0,}$ was calculated to 2468 minutes, based on equations given by Gustafson & Claesson (2004).

The grouting methods, earlier described and evaluated had a fixed number of grout holes per fan (spacing) and a fixed minimum stop flow. If the method, as described here, is applied, the number of grout holes and the minimum stop flow could be optimized for the different methods.

Chapter Eight

8 Concluding remarks and suggestions for further research

8.1 Concluding remarks

Within this thesis a methodology for evaluation of different grouting methods has been presented. The methodology focuses on predicting the sealing time needed to reach a required water inflow level for a jointed hard rock mass and further to use the predicted sealing time in order to facilitate choice of grouting method. The sealing time depends on activities such as: drilling, grouting, waiting, probing, re-grouting and post-grouting.

An increased requirement of a low water inflow into the tunnel may increase the sealing time and thereby the cost for sealing. By optimizing the grouting methods, the cost for grouting could be decreased. By predicting the sealing time for different grouting methods an appropriate method could be chosen and thereby also a cost effective sealing of the tunnel.

Based on calculations, it was shown that a fast grouting method may not always be the most cost effective method and that a cost effective method should be chosen based on both the sealing time and the obtained flow reduction.

Based on the calculation of sealing times, it was concluded that the sealing time for a medium conductive rock mass is more dependent on the number of establishments, than on the grouting time. For the more conductive rock masses, the sealing time was instead more dependent on the grouting time than on the establishment time. The sealing time was further shown to be highly dependent on the time for hardening of the grout, which accentuates the importance of using an appropriate hardening (waiting) time.

From the calculations, the importance of choosing the right grouting method, if the required sealing level should be possible to reach, has been demonstrated. A grouting method with a high sealing effect can fulfil the requirement and result in no post-grouting, while a grouting method with low sealing effect can result in extensive post-grouting.

To be able to choose a method, it is necessary to know if pre-grouting and post-grouting are critical factors for the specific project. If they are critical, the time is added to the total sealing time, if not, the time is not added.

If pre-grouting is a critical factor, it may be advantageous to choose a faster pre-grouting method with a lower sealing effect and later finish sealing with post-grouting. In general, post-grouting is not a critical factor during an underground project, but if the post grouting turns out to be extensive; it could become a critical factor.

For some situations with high demands on the sealing effect, it may not be possible to achieve the requested sealing effect with post-grouting. For these situations, grouting and re-grouting may have to continue until a certain sealing level has been reached, even if pre-grouting is a critical factor.

In general the variability between different fans within the same rock mass has shown to be large. For a less conductive, less jointed rock mass the variability was higher than for a more conductive, more jointed rock mass. The variability depends on the probability to intersect the conductive structures of the rock mass, which is dependent on the chosen spacing of the grout holes. One single unsealed joint with an aperture of 100 μm in a grouting fan with a length of 20 meters was shown to destroy the overall sealing effect of the entire fan.

The number of grout holes which is required to fulfil the sealing requirements depends on, for example, the grout mix, the grouting pressure, the connectivity between joints and the joint aperture. Based on experience, it is complicated to decide optimal grout hole spacing, but by using the methodology presented here, it is possible to optimize the grout hole spacing for a specific rock mass situation. Another way to optimize the grout hole spacing has also been conceptually suggested. The concept regards the relative penetration length of the grout and can be used to optimize the relationship between grout penetration length, hole spacing and pumping time.

A grouting method with too few grouting holes can, if the spacing and the grout mix is appropriate, be corrected by a re-grouting with complementing holes. But if the grout mix is not appropriate a too short penetration length can not easily be compensated by further re-grouting in complementing holes. A correlation between the grouting time and the re-grouting was an indication of a less effective grouting method.

Separation of the grout in a joint plane has not shown to negatively affect the sealing result. Instead, an extended initial use of a low viscosity grout mix with a high separation gave an improved sealing effect, compared to the initial use of a higher viscosity grout mix with a lower separation.

Control holes are often used to decide the extent of the re-grouting. Depending on the criterion, control holes may or may not assist in making an appropriate re-grouting decision. The control hole criterion used within these calculations was shown to be accurate if the grouting method performs badly or if the rock mass was highly conductive. For other situations, a decision based on the result of the control hole criterion, as defined here, gave more or less a randomized outcome.

8.2 Suggestions for further research

The work presented in this thesis suggests several topics for future research. Such work may cover topics relating to grouting optimization and practical tools or those relating to more fundamental understanding of parts of the grouting process.

Grouting optimization and practical tools

- Develop into a practical application to be used on site the theory of using the relative penetration length as a tool for optimization.
- By practical experiments and/or by calculation demonstrate the degree of flushing a grout mix may be exposed to during drilling.
- Study of cost and times from case histories, as a base for validation, of the sealing time model.

Fundamental understanding

- Decide, the level of accuracy for using different control hole criteria in different rock mass situations.
- Decide, the geological correlation distance in a rock mass, for which the same grouting performance would be an appropriate decision.

Chapter Nine

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Appendix

Grout data used for grouting method evaluation.

Grout	Super plasticizers	b min	B critical	Setting Time[h]	Viscosity [Pas]	Yield value [Pa]	Final separation %
Method MS-1 and MS							
Uf 12 vct 2	1.4% HPM	46	104		0.0053	0.13	23.5
Uf 12 vct 1	1.4% HPM	55	120	~7	0.0188	1.38	1.5
Uf 12 vct 0.8	1.4% HPM	65	140		0.1239	2.62	0
Injektering 30 vct 2	0.65%HPM	58	92		0.0043	0.05	56
Injektering 30 vct 1	0.65%HPM	72	101	~10	0.0085	0.45	10
Injektering 30 vct 0.8	0.65%HPM	74	104		0.0117	0.71	6
Injektering 30 vct 0.6	0.65%HPM	80	120		0.035	4.1	3
Injektering 30 vct 0.5	0.65%HPM	82	125		0.0574	6.13	1
Method MA							
Uf 16 vct 0.8	0.8 % HPM	44	100		0.093	10.3	1
Uf 16 vct 0.6	0.8%HPM	56	125	~4.5	0.99	16.7	0
Method ML							
UF 12 W/C 2. 26.3% GA	2.5% SP40	39	96		0.0171	1.53	8
UF 12 W/C 1.3. 17.5% GA	2.5% SP40	35	76		0.0252	4.27	2
UF 12 W/C 1.1. 17.5% GA	2.5% SP40	32	88		0.0284	5.86	1
UF 12 W/C 0.9. 13.1% GA	2.5% SP40	54	85		0.0872	9.48	0
UF 12 W/C 0.7. 13.1% GA	2.5% SP40	63	86		0.3056	21.42	0
UF 12 W/C 0.55. 13.1% GA	2.5% SP40	69	87		1.178	33.81	0
CONV. 125 W/C2. 26.3%GA	2.5% SP40	121	260		0.0142	0.25	11
CONV. 125 W/C1.3. 17.5%GA	2.5% SP40	92	144		0.0144	1.36	8
CONV. 125 W/C1.1. 17.5%GA	2.5% SP40	98	159		0.0158	5.51	6
CONV. 125 W/C0.9. 13.1%GA	2.5% SP40	108	170		0.0287	1.13	2
CONV. 125 W/C0.7. 13.1%GA	2.5% SP40	108	221		0.0397	1.23	1
CONV. 125 W/C0.55. 13.1%GA	2.5% SP40	113	1069		0.0456	2.1	0

The author's relief to have reached this point is only exceeded by that guy checking the English.