STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

JACKING OF ROCK FRACTURES DURING PRE-GROUTING IN SCANDINAVIAN TUNNELING PROJECTS

– a study of the effects from chosen grouting pressure

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JACKING OF ROCK FRACTURES DURING PRE-GROUTING IN SCANDINAVIAN TUNNELING PROJECTS – A STUDY OF THE EFFECTS FROM CHOSEN GROUTING PRESSURE

Deformation av bergsprickor vid förinjektering i skandinaviska tunnelprojekt – en studie av effekter från valt injekteringstryck

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PREFACE

In the Nordic countries, different praxis concerning applied grout pressure has developed. In Norway, very high pressures are used while considerably lower is used in Sweden. Any research that shows which pressure that is suitable to use, and the consequences if an incorrect pressure is used, has not been performed in Norway or Sweden. However, knowing which grout pressure that is optimal to obtain an acceptable sealing of the rock mass is essential for cost efficient grouting.

In this work, two different grouting projects have been analysed, a Norwegian where a high grout pressure was used and a Swedish where lower pressure was used. Both projects have been analysed with the theory for the Real Time Grouting Control Method together with the new theories that has been developed at KTH for jacking of rock fractures due to the grouting pressure. Based on the results from these analyses, advantages and disadvantages with high and low pressures respectively are discussed and conclusions presented. The idea is that this work could be used as a basis for future research concerning the work on suitable grout pressure.

The work has been performed by Jalaleddin Rafi together with Håkan Stille and Fredrik Johansson at the Division of Soil and Rock Mechanics, KTH. The analyses of the Holm-Nykirke project has partly been performed by Simon Nikolaev as a part of his master thesis project at KTH.

A reference group has followed the work and contributed with valuable contributions and review comments. The group consisted of Thomas Dalmalm (Swedish Transport Administration), Björn Stille (Sweco), Mats Holmberg (Tunnel Engineering), Johan Funehag (Chalmers), Lars Hässler (Golder), Eivind Gröv (Sintef &NTNU) and Per Tengborg (BeFo).

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Per Tengborg

FÖRORD

I de nordiska länderna har olika praxis vuxit fram vad gäller vilket injekteringstryck som används. I Norge används mycket höga tryck medan det i Sverige används betydligt lägre värden. Någon forskning som direkt belyser vilket tryck som är lämpligt att använda sig av, samt vilka konsekvenser som kan erhållas om fel tryck används, har inte utförts vare sig i Norge eller i Sverige. Att veta vilket injekteringstryck som är optimalt för att uppnå acceptabel täthet är emellertid väsentligt för en kostnadseffektiv injektering.

I följande arbete har två olika injekteringsprojekt analyserats, ett norskt där ett högt injekteringstryck användes och ett svenskt där ett lägre tryck användes. Projekten har analyserats med teorin för Real Time Grouting Control Method och de teorier som utarbetats vid KTH för vidgning av bergsprickor till följd av det använda injekteringstrycket. Utifrån resultaten från dessa analyser diskuteras för- och nackdelar med höga respektive låga tryck och slutsatser presenteras. Förhoppningen är att detta arbete kan ligga till grund för det fortsatta arbetet med val av lämpliga injekteringstryck.

Arbetet har utförts av Jalaleddin Rafi tillsammans med Håkan Stille och Fredrik Johansson vid Avdelningen för Jord och Bergmekanik, KTH. Analyserna av Holm-Nykirke projektet har delvis utförts av Simon Nikolaev inom ramen för hans examensarbete på KTH.

En referensgrupp har följt arbetet med projektet och bistått författarna med värdefulla synpunkter och granskningskommentarer. Gruppen bestod av Thomas Dalmalm (Trafikverket), Björn Stille (Sweco), Mats Holmberg (Tunnel Engineering), Johan Funehag (Chalmers), Lars Hässler (Golder), Eivind Gröv (Sintef &NTU) och Per Tengborg (BeFo).

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Stockholm i maj 2017

Per Tengborg

SUMMARY

Determination of grouting design parameters such as grout mix properties, stop criteria and pumping pressure is challenging. By using grout viscosity and yield stress, the amount of applied pressure and the duration of injection, the grout spread can be estimated. Thus, different suggestions for the variation of these parameters (based on the geology and function of the project) has been discussed and implemented in previous grouting projects. From this point of view, it is interesting that in Scandinavian projects, different values are considered for these parameters despite the similarity in geology, which mostly consists of hard crystalline rock.

Much higher grouting pressure is used in Norwegian projects compared with grouting projects in Sweden. A relatively thinner grout is also used in the Norwegian practice. Thus, an overspread of grout with the Norwegian method could be expected. Furthermore, the high applied pressure may cause deformation in fractures, which itself can affect the sealing efficiency of the grouting program. In this study, the results from grouting in a railway tunnel in Norway (Holm-Nykirke) and a metro tunnel in Sweden (Stockholm City Link) are compared with respect to these questions. For this purpose, the Real Time Grouting Control method, which enables an estimation of the grout spread theoretically, has been used.

The results from this comparison confirm the overspread of grout in the Norwegian tunnel, mainly due to the relatively long duration of the grouting. The results also showed that clear signs of fracture jacking could be observed in both projects. However, in the Norwegian project, jacking was observed more frequently than in the Swedish project due to the higher applied pressure. It was also observed that the frequency of jacking in the Norwegian tunnel was strongly linked to type of geology, with more jacking in the sedimentary rock, where long persistent joints also were mapped, compared to the hard crystalline rock, with shorter fractures observed in the tunnel mapping. The results also showed that even if a high pressure is used, it may not speed up the grouting; the main reason being that the grout penetration may be reduced significantly if the fracture is jacked. Considering the risk of opening the previously grouted fractures (which may remain unsealed) and the risk of reducing the grout spread, application of high pressure is not an optimum solution, especially in rock masses with persistent fractures.

Based on this work, it can be concluded that the theoretical approach can provide essential information by quantifying consequences of elastic jacking that can be used for an optimizing of the injection pressure and stop criteria in the design phase.

SAMMANFATTNING

Bestämning av parametrar för design av injektering såsom bruksegenskaper, stoppkriterier och injekteringstryck är utmanande. Genom att använda värden på dessa egenskaper är det möjligt att uppskatta brukets spridning i sprickorna. Till följd av detta har olika kombinationer av värden på dessa parametrar föreslagits och använts i olika injekteringsprojekt (baserat på geologi och projektets funktion). Utifrån denna aspekt är det intressant att väsentligt olika kombinationer av injekteringstryck och bruksegenskaper används i Skandinaviska injekteringsprojekt, trots en likartad geologi som i huvudsak består av hårt kristallint berg.

I Norge används normalt högre injekteringstryck jämfört med svenska injekteringsprojekt. Det är också vanligt att tunnare bruk med lägre viskositet används i Norge. Det är möjligt att detta kan leda till bruksspridning som är längre än vad som krävs. Vidare kan det höga injekteringstrycket leda till deformationer i sprickorna, vilket kan påverka den tätande förmågan och brukets inträgning. I denna studie har resultatet från två olika injekteringsprojekt jämförts med avseende på dessa faktorer; ett tunnelprojekt i Norge (Holm-Nykirke) och ett i Sverige (Citybanan i Stockholm). För att kunna genomföra jämförelsen har det teoretiska metodiken ”Real Time Grouting Control Method” tillämpats, vilket möjliggör en beräkning av bruksspridningen.

Resultaten från denna jämförelse bekräftar att en längre bruksspridning än vad som är nödvändigt för att uppnå täthetskraven uppnåddes i det norska projektet, i huvudsak på grund av den långa injekteringstiden. Resultaten visade också att tydliga tecken på jacking av sprickorna kunde observeras i båda projekten. Den observerade frekvensen av jacking var emellertid väsentligt högre i det norska projektet jämfört med det svenska till följd av det högre injekteringstrycket. Det kunde också observeras att den observerade frekvensen av jacking var starkt kopplad till geologi, med en högre frekvens i sandstenen jämfört med det hårda kristallina berget, sannolikt till följd av längre spricklängder i sanstenen samt en mer ogynnsam orientering. Resultaten visade också att även om ett högt injekteringstryck används behöver det inte alltid förkorta injekteringstiden, då det högre trycket kan leda till deformationer i sprickorna vilket därmed begränsar brukets inträngning. Med hänsyn till risken för att öppna upp tidigare injekterade sprickor samt begränsa brukets inträgning är ett för högt injekteringstryck inte en optimal injekteringsfilosofi, speciellt om geologin är ogynnsam med långa horisontella sprickor.

Baserat på detta arbete är slutsatsen att det teoretiska ramverket som ”Real Time Grouting Control Method” utgör kan användas för att analysera om jacking förekommer samt kvantifiera dess konsekvenser. Teorin kan därmed användas för att optimera injekteringstryck och stoppkriterium vid design av injekteringens utförande.

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1. INTRODUCTION

Decades before, grouting was not engineered and there were no specific design for different ground conditions. As mentioned by Wilson and Dreese (2003), prior to the 1980's, there were no clear goal when the grouting work was performed (Wilson and Dreese, 2003), and the focus was only on the economy since it was assumed that the quality of the grouting work did not affect the performance of the constructed grout curtain.

In tunneling projects, pre-grouting is performed with the purpose of establishing an impervious zone around the tunnel periphery by reducing the penetrability of the most conductive fissures in the rock mass (exp. Grov, 2001). He emphasized that pre-grouting can be used to control ground water in order to prevent leakage of stored products (e.g. oil and gas). Bruce (2007) emphasizes the importance of the grouting process and the success of the pre-grouting in order to meet the goals of the project. The failure of the pre-grouting may significantly affect the project in terms of safety, time, performance and costs.

The challenge in grouting projects apart from executive issues is the determination of design elements such as pumping pressure, material properties and stop criteria. With respect to this, different methodologies have been proposed where most of them are based on empirical approaches. However, relying only on previous experiences and so called rules of thumbs have been questioned by many, e.g. by Weaver (1991). He recommended that design parameters, especially grout-injection pressures, should be based on site-specific factors and that the conditions at each specific grout hole should be considered separately (based on grout hole logging and water pressure test data), rather than mindlessly following either empirical "rule".

In the pre-grouting of tunnels, the injection pressure should initially be determined with the aim of achieving a desired spread of the grout around the excavation in the shortest time and with the least damage. As an empirical suggestion, Houlsby (1990) correlated the allowable grouting pressure to the quality of the rock mass and the rock cover above the grouted fracture. He suggested continuing the grouting process until a certain maximum injection pressure or certain maximum injected volume has been attained. In order to control fracture deformation, Lombardi and Deere (1993) suggested to consider a combination of injection pressure and injected grout volume (The Grout Intensity Number or GIN-value) in addition to the previously set stop criteria by Houlsby (1990). This means that with respect to GIN number, the maximum applicable pressure should be decided by considering the amount of injected volume. Although this approach is a step forward in the understanding of the mechanism that causes fracture deformation, there are ambiguities connected to this methodology and the determination of the intensity number is challenging. Different studies have been performed to clarify the function of the GIN method, e.g. Rafi and Stille (2015)[2] examined these ambiguities by using a theoretical approach (see chapter 3).

Grouting practices vary depending on the geological conditions and the requirements of the grouting. For instance, the U.S practice in determination of the initial injection pressure, where modern on-line monitoring tools are utilized, is required to be conservative, while in Norwegian projects it is suggested to practice a very high pressure. With the current grouting applications, the outcomes of the projects are not satisfactory in some cases. That no theoretical solution is practiced in the design and in the evaluation of the grouting process might be one reason for this. Due to this, possible phenomenon correlated to injection pressure and stop criteria are remained hidden in the grouting process. Today, with the advancement in technology and the increase of knowledge through close studies of different aspects in the grouting process, it is possible to analyze the effects of the initially determined design parameters during the grouting process and optimize them in the next design step. Having such methodologies are essential in difficult grouting cases.

1.1 Scope and objective

The objective of this study is to investigate the effect of the choice of injection pressure in the pre-grouting of rock tunnels. Furthermore, the objective is to give an insight to the practicing engineers about the benefits and disadvantages of using low or high pressures. To obtain these objectives, possible requirements and criteria for successful grouting are first discussed. The report is continued in chapter 2 by a state of the art report on current practices concerning the choice of grouting pressure. Different parameters, which may be affected by a change in the pressure, are examined closely in chapter 3. These parameters are mainly fracture aperture, stiffness of the rock mass, penetrability of the grout, depth of grout spread and transmissivity of the fractures after grouting. Following that, the methodology used to compare two Scandinavian tunneling projects is established in chapter 4. In chapter 5 and 6 respectively, grouting in a tunnel project in Sweden is first analyzed followed by a similar analysis of a Norwegian project, where the latter project used considerably higher grouting pressure than the former one. The analyses are performed statistically and the applicability of the theoretical approach is examined. After that, positive and negative aspects of the used pumping pressure at these two projects are discussed quantitatively in chapter 7 by applying “Real Time Grouting Control Method” (introduced in chapter 3). In chapter 8, recommendations about the determination of pumping pressure that can be used in projects with similar geology are given and areas for future research are discussed.

1.2 Requirements on grouting in tunneling projects

According to Dalmalm (2004), sealing requirements in tunneling projects has been better handled when grouting has become an integrated part of the tunnel excavation cycle. Due to the usually good quality of the rock mass in the Nordic countries, which eliminates the need for a concrete lining, it is used as a permanent sealing solution. In order to be successful, this kind of grouting needs to fulfill a list of requirements.

The grouting process should confirm that the inflow to the tunnel is lower than the requirement limit. For this purpose, the functional requirements of the tunnel set the criterion for the acceptable amount of water inflow. To examine if these requirements are fulfilled, measurements of water inflow into the excavation are performed, water loss measurement before and after grouting are conducted, pore pressure and ground water level measurement are executed and settlements in the area surrounding the tunnel area are measured.

Going back to the main goal of grouting, which is achieving a certain sealing efficiency in the shortest time, attention should be given to the execution of the grouting as well as economical and safety considerations. With respect to this, criteria are set for the level of tightness, i.e. required sealing efficiency, the need for re-grouting, grouting time, environmental considerations and safety of the execution. The grouting method and the design parameters are determined based on these criteria. Furthermore, they can be used in defining certain limits in order to evaluate the performed grouting work. Below, a brief description of these requirements is given.

Level of tightness (sealing efficiency): Required sealing efficiency is defined based on the existing inflow of water and the allowable one. According to Dalmalm (2004), it can be estimated by considering the inflow to a tunnel before and after grouting (Eq.1.1).

$$Sealing\ effect\ (\%) = 100 \cdot \frac{Inflow\ without\ grouting - Inflow\ with\ grouting}{Inflow\ without\ grouting} \tag{1.1}$$

It means that the required sealing efficiency is directly correlated to the amount of inflow into the tunnel without grouting, which itself depends on the water pressure and the rock mass conductivity. Palmström and Stille (2010) classified the inflow of water into the underground excavation according to Table 1.1. According to them, a large ingress of water into a tunnel will reduce the excavation capacity as well as causing problems related to the operation of the tunnel. However, based on data in Dalmalm (2004), it is easier to increase a sealing efficiency which is not initially high compared with the effort needed to obtain such an increase in a relatively tighter zone (Figure 1.1). As showed in this Figure, which shows the sealing efficiency of the grouting of the fans at the Hallandsås Rail Link, the highest sealing efficiency (90%) was achieved in the first round of the grouting. The following grouting had less influence on the total sealing efficiency (total efficiency of 94 and 97 percent after second and third rounds respectively, i.e. only 4% and 7% of increase in sealing efficiency).

Table 1.1 Classification of inflow of water (after Palmström and Stille, 2010)

Condition	Inflow Volume
Dry	<0.1 L/min
Seepage	0.1-1 L/min
Dripping	1-10 L/min
Flowing	10-500 L/min
Heavily flowing	30-300 m^3/h
Water in-burst	>300 m^3/h

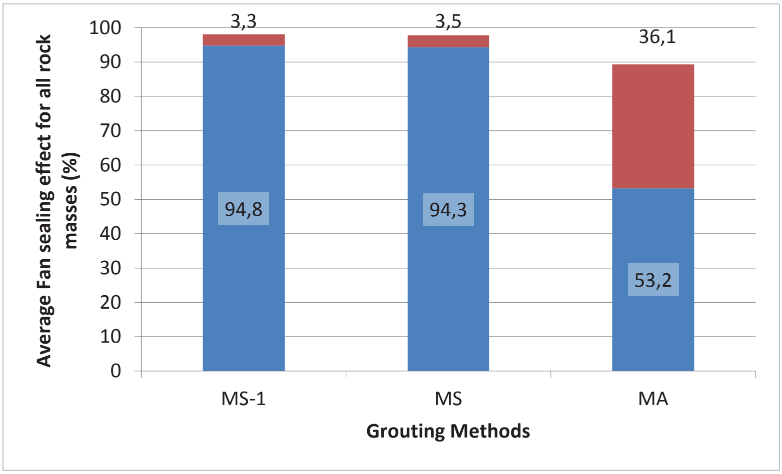

Figure 1.1 Total average fan sealing effect after grouting and re-grouting (after Dalmalm 2004)

The thickness of the grouted zone is an influencing factor on the sealing efficiency. Dalmalm (2004) showed that increasing the thickness of the grouted zone beyond a certain limit is of less benefit. This model of homogeneous grouting thickness around the tunnel should be questioned, especially in grouting of good rock (Stille, 2016). Functionality of the grout mix (high yield value, low viscosity, high penetrability and low bleed) is also an affecting factor (Eriksson, 2002).

The allowable inflow depends on the purpose of the underground structure, location and size of it, the rock cover, the consequences of leakage, safety issues, temporary and functional requirements as well as economical aspects (Hognestad, et al., 2011). Grov (2001) discusses that this allowable inflow is governed by practical limitations related to the excavation process and the injection pressure. Dalmalm (2004) suggests a reduction of the conductivity with a factor of ten to fulfill the sealing requirements. The minimum possible conductivity with cement grouting, according to the Swedish road administration, is 0.5-1.5×10^{-7} m/s. However, a decrease of conductivity from 10^{-7} m/s to 10^{-10} m/s has been reported by Emmelin, et al. (2004) during a test in Äspö Hard Rock Laboratory. An example of another successful project is the Stockholm City line project, where the aim was to limit the ingress of water after grouting to less than 7 liter/min/100m in one of access tunnels (The requirement of the whole project was 4-5 liter/min/100m). However, Tsuji, et al. (2012) reported a decrease of water inflow to 1.9-3.4 Liter/min/100m, i.e. a conductivity which decreased to 3×10^{-9} m/s. In tunneling projects in Norway, the maximum allowable water inflow in urban areas is in range of 2-4 liters/min/100 meter tunnel (Hognestad, et al., 2011). They showed that by simulating hydrological conditions for a specific tunnel, the leakage of water to that tunnel had to be limited to 1 - 3 liters /minute/100 meters in order to avoid the ground water table to decrease more than a few meters. This limit can be increased up to 30 liters/min/100 meter at subsea tunnels (Blindheim and Øvstedal, 2001 and Grov, 2001).

Need for re-grouting: It is important to complete grouting in one round both from a technical and an economical point of view, especially in good rock grouting (Roald, et al., 2001). Re-grouting may be needed in case of poor performance in the first round of grouting. Dalmalm (2004) showed that the probability of not fulfilling the sealing requirement with pre-grouting is low if the grouting effort is extensive in relation to the requirement. In contrast, in case of moderate effort, there is a higher probability of failure and need for re-grouting.

Choosing the right level of effort is important since it influences the cost of the grouting, the waiting time and the possibility of failure in pre-grouting. This is important to consider since, as mentioned by Hognestad, et al. (2011), the costs of pre-grouting will be higher if more than one round of grouting must be carried out before the tunnel excavation can continue.

From the experience of difficult grouting conditions at the Romeriksporten tunnel, Kveldsvik, et al. (2001) suggested that insufficient pre-grouting, i.e. thin grout cover around the tunnel, leads to excessive leakage from the floor of the tunnel. Furthermore, the high ground water pressure makes post-grouting much more challenging compared to the pre-grouting process. They concluded that in serious tunneling projects, postponing the effort of grouting until later should not be an option. As discussed by Roald, et al. (2001) the marginal increase of cost in one round will save the cost of a second round.

Grouting time: In an effort to correlate the sealing efficiency with the grouting time, Dalmalm (2004) found that for some methods a grouting time or re-grouting time longer than a critical time (the time implying that a good sealing effect may have been achieved) will always give a good sealing effect. Although a grouting procedure to achieve a high sealing efficiency in the pre-grouting may takes longer time, it might be more efficient in the end since the execution of post-grouting might become very time consuming and costly.

According to Dalmalm (2004), the average grouting time for a fan increases with the increase of the hydraulic conductivity of rock mass. However, the grouting time in different fans is quite close. As mentioned by him, post-grouting can also affect the grouting time significantly if it turns out to be extensive. This will occur in cases of low sealing efficiency of the chosen pre-grouting method. Thus, he emphasizes that "the requirements for post-grouting" is an important criterion in the choice of the pre-grouting method.

Environmental considerations: There are adverse environmental aspects connected to the excavation of a tunnel. Tunneling projects may lead to a lowering of the ground water table, which result in settlement of buildings and surface structures in urban areas (Hognestad, et al., 2011). In this case, the leakage into a rock tunnel may lead to a settlement of the surface due to the reduction of the pore pressure. In order to avoid that, they propose that the acceptable limit of the pore pressure reduction is of the magnitude 1-3 meters. However, these values are site specific and can to vary from case to case depending on the hydro-geological conditions.

In addition, in natural lakes and ponds this construction work may disturb existing biotypes. This disturbance is strongly correlated to the vulnerability of the nature to the amount of water. Kveldsvik, et al., (2001) found that a leakage less than 10% of the run-off in the catchment area has no or small effect on the nature, while this effect is medium when the leakage increases to 10-20 %. The natural hazard is significant in larger amount of leakages. Other environmental hazards according to Hognestad, et al. (2011) are reduction of the ground water reservoir (wells) and the possibility of pollution from leakage of gasses and liquids. They suggest evaluating damage connected to leakage of ground water based on practical use of the water source, the biodiversity and the presence of water-dependent vegetation. Besides ground water leakage, there are environmental hazards related to leakage of grout to the surface which can lead to contamination of soil and water.

Safety of the work: One of the major safety considerations is related to a possible ejection of the packer at high pressure. Hognestad, et al. (2011) describes the major causes for this ejection as poor fixing of the packers, inadequate friction between packers and borehole (the cement in the hole acts as a lubricant), densely fissured rock that the packer has been placed in, wrong type of used packer and failure in the packer locking system. To avoid these hazards they suggest using packers adaptable to the pressure as well as chaining it to the working face properly. Furthermore, flushing out the cement in the hole before installing the packer or allowing the water to first pass through the packer can avoid the sliding problem. Splatter of grout fluid during grouting and buckling of the face or the ceiling due to the high applied pressure, especially in tight boreholes, are examples on other hazards.

Summary: The highest possible pressure to shorten grouting time as well as stop criteria that confirm fulfilment of sealing requirement can be the optimum grouting design. Additional grouting time might be helpful to achieve the required thickness in one round of grouting. Considerations for leakage of grout to the surface and the safety of the work are also essential parameters to be considered when the grouting pressure is decided.

2 CHOICE OF PUMPING PRESSURE

Grouting is an effective long-term solution if we use best practice to intersect and fill those fractures. (Heinz, 2012) proposed a method in which grouting pressures should be as high as possible to achieve reasonable penetration in the shortest time. According to them, hydro-fracturing should be avoided, as it is difficult to grout all voids that are induced by this phenomenon. For this purpose, considering the overburden as the only existing stresses, the choice of the grout pressure has been related to the quality of the rock mass and the vertical depth of the fracture from the ground surface. Many have depicted this approach, e.g. Houlsby (1990) established a graph that shows the maximum applicable pressure at different depths sorted by the rock mass quality. According to this graph, high pressure is applicable in projects situated in hard crystalline rock masses such as those frequently occurring in Sweden. Under these conditions, application of a pressure even higher than the overburden was acceptable considering the vertical orientation of the fractures. This high pressure in fractures with orientation close to horizontal may lead to elastic deformations of the fractures. However, it is not only the weight of the overburden and the rock mass quality that influences the chosen grouting pressure, other parameters might also influence it. In this chapter, the practice in the choice of grouting pressure in different countries and methods are discussed.

2.1 Norwegian practice

In Norwegian projects, where drained sprayed concrete linings without waterproofing measures are utilized, Grov and Woldmo (2012) emphasis that pre-grouting is the only mean for control of the water flow. According to them, this technique is important especially in sub-sea tunnel projects with an infinite source of water above the tunnel that requires strict limitations on the water inflow. This concept has been developed during construction of high-pressure water tunnels for hydro-power projects, oil and gas storage and sub-sea rock tunnels to the current generation of urban tunneling projects (Broch, 2001). The geography of Norway and the spread of its population over the country imply a challenge for the development of the infrastructure, especially in tunneling projects. In these projects, the hydrogeological situation is dominated by a high groundwater level and usually also a good quality of the rock mass.

In Norwegian practice, the permissible grouting pressure will vary from a few bars up to 100 bars and it depends on the in-situ stresses and the position of the grouting hole (Table 2.1).

Table 2.1 Variation of grout pressure in Norwegian urban tunnels based on location of holes (After Grov, et al., 2014)

Rock cover	Max grout pressure in holes in roof & walls	Max grout pressure in invert holes
0 – 5m	20bar	30bar
5 – 15m	40bar	60bar
>15m	100bar	100bar

Grov and Woldmo (2012) found the use of a high applied pressure effective, even though this high-applied pressure might exceed the in-situ stresses in the rock mass and in that case imply that elastic jacking can occur. The resulted deformation has been expected to be favorable since the grouting time may decrease and the penetrability of small fractures may increase. Furthermore, it is desired to open up the fractures if there is a lot of clay in the fissures and/or if there are other problems to penetrate the water-bearing channels with grout (Hognestad, et al., 2011). Roald et al. (2001) emphasize on using high pressure under good rock mass conditions, where the probable small deformations from the grouting help to consolidate the rock mass. Furthermore, according to Roald et al. (2001), a high-applied pressure ensures that a required depth of the grout spread is achieved in the finest joints. By comparing different projects in Norway, Davik and Andersson (2001) concluded that using a pressure as high as 90 bar result in a better penetrability, but this is not proved to be applicable in general cases.

Despite these advantages, there are difficulties with applying a high pressure. One of the negative consequences of a high-applied pressure can be an over spread of the grout and leakage of the grout to the surface in case of a small rock cover. Furthermore, in poor rock mass conditions, Grov and Woldmo (2012) point out that the grout consumption increases and can impose harm to the tunnel surroundings. In order to avoid that, Hognestad, et al. (2011) suggest establishing an extra “cut-off cover”. In this method, initially the grout is injected with a lower pressure and fills the wide cracks in a zone outside the required grout thickness zone. This will make it possible to use a higher pressure in the second step to seal the rock mass sufficiently while avoiding leakage of grout to the surface or disappearing outside the intended zone.

The other negative aspect of a high applied pressure can be related to the induced deformation of the fracture, i.e. jacking of the rock fractures. These deformations are initially elastic and recover if the grout pressure is released. However, permanent rock movement can occur in case of large induced forces from high pressures. The significance of elastic jacking in tunneling projects depends on the order which the boreholes are injected, since the new void due to deformations can be filled during grouting of holes in the vicinity. However, usage of higher pressure in the final grouting holes may open up the previously grouted fractures, i.e. un-grouted void remains in the final product. In Norwegian projects, it is common practice to start by grouting the floor holes and then continue progressively upward towards the ceiling (Hognestad, et al., 2011). The reason is the difficulty in accessing the ceiling holes. Furthermore, the cement mix has higher density than the water and by starting grouting the floor holes, water is “squeezed” forward and upward. Furthermore, by considering the rock cover, smaller pressure is usually applied in the ceiling holes (e.g. see table 2.1) and thus it is favorable to perform grouting in ceiling holes after the other holes in the floor and the walls. Table 2.2 compares some of the Norwegian projects with regard to the sealing requirement and the applied pressure. In all cases, grouting was successful and the sealing requirements were fulfilled.

Table 2.2 Sealing requirements and applied pressure in some Norwegian grouting projects (Some data were not available).

Project	Allowable inflow	Final flow	Rock cover	Pressure	Grout hole length	Material	Remarks
Baerum Tunnel	4 l/min/100m	2 l/min/100m	23.5 m	30-80 bar in face, 20-80 bar in ceiling		W/C 0.5-0.8 industrial cement	-
T connection	5 l/min in fan	-	139 m sub sea	60 bar	28 m	Industrial cement W/C 0.8-0.6	Grouting as deem necessary
Gevingåsen Tunnel	15 l/minute or 5 l /minute in exposed area	-	-	70 bar (stop pressure)	18m	W/C 0.8-1 Micro cement	Reduction of pressure based on observation
Mongstad		2 l/min/100m	24m	80 bar stop pressure			
Loren	-	-	-	80 bar floor 60 bar ceiling	15-19m	W/C 0.5-1 Micro cement	V<100 liter P increase to 80 bar and maintain 5 min V>1500 liter P decrease to 30 bar and stop
Tåsen tunnel	10 l/min /100	1.3 l/min /100	-	Initial 20-25 bar Final 35-45 bar		Micro cement	-
Svartdal Tunnel	5 l/min/100	4,3 l/min/100m	2.5 m		21 m		Poor rock
Storhaug tunnel	3-10 l/min/100m	1.6 l/min/100m	4-6 m	30-50 bar, max 70 bar	14 m	Micro cement W/C 1.1-0.4 usually 0.9-0.7	-
Bargernes tunnel	10-30 l/min /100	8-25 l/min/100m	10-150m average 100m	80-90 bar	22 m	W/C 0.5	Active design High pressure resulted to time efficiency
Baneheia tunnel	2 l/min /100 under the lake	1.7 l/min /100	10-40m	50-80 bar lower under the lake	21-24m	W/C 0.9-0.7 micro cement U12	Some parts under the lake

2.2 U.S. Practice

In projects in the United States, exploration and assessment of the situation, responsive execution and verification, and monitoring of the grouting performance are main steps needed to confirm success in remedial grouting (Bruce, 2007). The later requirement, the real time monitoring, results in confidence in the quality of the product (Dreese, et al., 2003). Monitoring of the grouting process, as mentioned by Barison, et al. (2012), can provide an optimum control of the grouting operations as well as a chance of better decision making. Wilson (2012) listed the advantages of monitoring the grouting process. According to him it has the ability to detect hydro-jacking and hydro-fracturing, could lead to a substantial reduction in the number of staff (in QC and QA departments), results in less effort for preparing daily reports, shortening grouting and water testing time, facilitating evaluation of data, providing immediate understanding of the subsurface and grouting result (by converting the drilling and grouting results into understandable visual imagery), and eliminating ”mysteries” associated with grouting. Besides all of the factors above, Dreese, et al. (2003) claim that this methodology lead to a faster grouting at lower cost. Wilson and Dreese (2003) reported a 15% reduction in the total grouting cost in the construction of the grout curtain for dams using this monitoring of the grouting process.

The first computerized software package for monitoring of the grouting process was developed by Bachy in 1986 (Barison, et al., 2012). The first requirement for use of computer monitoring by the U.S. army Corps of Engineers (USACE) was in remedial grouting at Patoka Lake Seepage, which set the standards for contracting methods and field applications (Dreese, et al., 2003). The technology was developed further in Hunting Run Dam and the largest computer monitoring for a cement grouting project was reported by Barison, et al. (2012) at McCook reservoir at 2006.

Dreese, et al. (2003) described that methods with limited calculations cannot survive since the obtained information is not enough for decision making. Even in case of having huge recorded data, a systematic analyses method is needed. In order to overcome these obstacles, they recommended the usage of the recent developed monitoring tools, which contain real time onsite and offsite analysis. It can display geological features, water test data and hole geometry graphically. By using this system, the total volume of the injected grout, the effective injection pressure and the flow rate are recorded and displayed graphically. According to Dreese, et al. (2003) this monitoring system can automate operations and allows real time onsite display, which provides the chance of onsite or remote analysis of the grouting results.

In grouting projects in the U.S., the stop criteria generally bring each grouting stage to absolute refusal by progressively varying the rheology of the grout (changing the mix from “thin” to “thick”, i.e. decreasing the water cement ratio). The goal of this progressive refusal, according to Bruce (2007), is to gradually reduce the Apparent Lugeon value to zero at target refusal pressure. Therefore, the main recorded and monitored parameter is the Apparent Lugeon, which expresses the relative permeability change of the formation during the application of the grout by relating the injection pressure to the grout take. The Corps of Engineers (2006) and (2008) define it as the grouting permeability of the formation, using a grout that behaves as a Bingham fluid with a known apparent viscosity as follow:

$$Apparent\ Lugeon = \frac{Flow\ Rate}{\frac{Effective\ Grout\ Pressure}{10\ Bar} \times Stage\ Length} \times \frac{Marsh\ Grout}{Marsh\ Water} \qquad (2.1)$$

Regarding the choice of pressure, Bruce (2007) suggests limitations on pressure to prevent "blow back" to the face of the tunnel (where there is only atmospheric pressure) as well as hydro fracturing and jacking of the rock. In this practice, the effective grouting pressure has been set empirically to a maximum pressure of 0.0055 MPa/m of overburden and 0.011 MPa/m of rock material, which as mentioned by Barison, et al. (2012) is conservative and lower than overseas. However, monitoring grouting in real time allows the usage of higher pressure, even exceeding the rule of thumb for U.S. practice, since damages due to the high applied pressure can easily be detected (Dreese, et al., 2003). When using a monitoring system, experienced personnel can reduce the injection pressure or stop the injection process in this case, i.e. the continuous monitoring confirms the usage of a safe pressure.

Although a modern monitoring system reduces onsite inspection staff time and increase quality of recorded data, this system is appropriate to be used in large projects or where there is a need for avoiding consequences of poor sealing, i.e. environmental contaminant projects (Dreese, et al., 2003). One of the main objections to use this system is the lack of theoretically analyzed data in order to examine if the criteria for successful grouting (obtaining required depth of grout spread in shortest time while avoiding undesired jacking) has been achieved. Thus, due to the lack of enough information, it is difficult to update the design parameters only based on continuous monitoring (Grouting pressure, mixture properties and stop criteria).

2.3 Swedish Practice

In Swedish grouting projects, there is awareness about applicable pressures and environmental considerations, including occupational health, safety and contractual requirements. Level of tightness in these projects depends, among other things, on the function of the project. For instance, in the design of the final repository for nuclear waste, Emmelin, et al. (2007) suggests flow requirement according to the values presented in Table 2.3. However, in the Stockholm City line project, which is an urban commuter train tunnel, the inflow requirement of the whole project is 4-5 liter/min per 100 m. The access tunnel was allowed a higher ingress to shorten the construction period in order to allow a faster access to the main tunnels (7 liter/min per 100m).

Table 2.3 Inflow requirements in repository (after Emmelin et al., 2008)

Deposition hole Spot	0.1 l/min
Deposition tunnel Spot	10 l/min, 300 m of tunnel
Shafts and access ramp	10 l/min, 100 m of tunnel
Other underground facility parts	10 l/min, 100 m of tunnel

One approach in the grouting design for projects with very high sealing requirement is to analyze the investigations stepwise. This includes the investigation of an initially drilled core followed by probe and grouting boreholes with pressure-build-up tests, and with measurements of inflow during the drilling. The aim of these investigations is mainly identifying singular fractures needed to be sealed. For that, numerical calculations that consider the distribution of the fractures are used. Afterwards, the sealing efficiency is estimated by measuring the flow before and after grouting and based on that, the distribution of the fractures is justified to be compatible with the estimated sealing efficiency. This methodology has been suggested for grouting in nuclear repository and has been named the characterization method (Emmelin, et al., 2008). Combining the results with inflow requirements, the critical allowed transmissivity is derived, which is the base for the design of the grouting. Emmelin, et al.(2004) suggest to successively update the rock description (from the characterization by the numerical model), as the information increase. Through this model, the grouting design parameters are tested and iteratively determined, i.e. the results are continuously used to confirm or make an updated grouting design. In these calculations the geometry of the fractures, the grouting technique and the properties of the grout are included (Emmelin, et al., 2004).

In case of tolerance for higher inflow, stop criteria and the pressure can be determined based on the geological properties and the flow of grout. According to Eriksson (2002), the stop criteria that have significant effect on the sealing result are suggested to be determined based on the size of the apertures that needs to be grouted to fulfil inflow requirements. Thus, as stop criteria, in fractures with small apertures the minimum inflow criterion is applicable while in larger fractures, a maximum injected volume should be set. Grouting pressure is also determined based on the size of the fracture aperture (Table 2.4).

Table 2.4 Correlation of grouting pressure and fracture aperture (After 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	++	+	-

With recent theoretical developments, which enable an estimation of the distance of the grout spread in real time, a certain grouting time is possible to be suggested as stop criterion for any setting of grouting pressure and material properties. This approach has been applied for analyzing the performed grouting work in a trial section of an access tunnel related to the Stockholm City line project. The results showed that in 90 percent of the grout holes, adequate depth of grout spread was achieved in less than 20 minutes. Based on that, the design was updated and this time limit was introduced as one of the stop criterion (see Tsuji, et al., 2012). Furthermore, the size of the overburden was considered in the determination of the maximum applicable pressure, even though in some cases a larger pressure was applied (Tsuji, et al., 2012).

2.4 GIN Method

Besides a certain maximum limit for pressure and grout volume, the application of this concept proposes to limit the combination of these two parameters, which according to Lombardi (2003) leads to a limit against the dangerous zone for hydro jacking or hydro fracturing. This boundary condition, which appears as a hyperbola trimming the rectangle of maximum pressure and maximum volume, has been named the Grout Intensity number (GIN). By that, grouting is performed until this hyperbola (of the product between pressure and volume) is reached. In case more grout is injected, i.e. increase of the volume of the grout, the pressure should be decreased along the hyperbola to avoid hydro jacking.

This method have been found robust in many projects but have also, however, been reported inefficient in some projects (see e.g. Bonin, et al., 2012 and Steyn and Mouton 2012). The main difficulty with this method is the ambiguity in the nature of the GIN. It is difficult to define the GIN as well as the stop point on the hyperbola, since neither the state of the fracture nor the depth of the grout spread is known at the intersection of the GIN hyperbola. There have been studies in order to clarify these ambiguities by describing the GIN in connection with the theoretical approach (Rafi and Stille, 2015)[2].

2.5 Other Methods

One design method proposed by Willson (2012) is the Equivalent Porous Media (EPM) modeling. In this method, the parameters of the EPM model are obtained from geologic exploration holes and water pressure test. He claims that by employing this method, in which the fractured rock is considered as a porous media, a reasonably defined zone of verifiable residual permeability is constructed. Thus, this method allows a quantitative analysis rather than a qualitative one. However, according to Long, et al. (1982) no equivalent homogeneous permeability will exist for a fractured rock mass without considering a large representative elementary volume.

In contrast, Shuttle and Glynn (2003) suggested a discrete fracture network (DFN) approach where grout injection boreholes and the intersected fractures are modeled explicitly. They claim that this approach is more realistic since it considers the flow pathways through the rock mass better compared with conventional grout analysis. The conventional grout analysis assumes a desired reduction in flow, based on a limited number of borehole hydraulic test, without simulation of the flow of water and grout through the fracture network. This can be used in the pre-grouting stage to optimize the configuration of the grout holes. The outcome is the percentage of tight grout holes and the ones that take large amount of grout as well as the mean and standard deviation of grout take per hole (Shuttle and Glynn 2003). Elmo, et al. (2014) found the DFN approach to be an ideal numerical tool that enables synthesizing realistic fracture network models from digitally and conventionally mapped data. They confirmed this conclusion by establishing a full-scale DFN model, where realistic geometric models of the fracture networks could be provided by defining the equivalent rock mass parameters (fracture orientation, length and intensity).

The realistic 3D model that is developed through DFN modelling has the ability to provide a clue about the optimum grouting by matching the volume of grout injected to the "void filling" aperture of the fractures encountered in each grouted 'stage' of the hole (Bonin, et al., 2012). In this method, which has been named "Aperture Controlled Grouting" by Carter, et al. (2012), achieving the required volume that is necessary to be injected for a certain stage is the point of interest, and grout rheology is then adjusted to match fracture characteristics in order to achieve pressure build for refusal at the required take. The method, according to Carter, et al. (2012), is an extension of the GIN method, however, the main differences are that grout volume is limited based on fracture aperture encountered in each stage and is not an outcome of the grout rheology and injection pressure. Furthermore, the properties of the grout mix are varied to obtain the pressure at refusal.

For grouting of dam curtains, Wilson and Dreese (1998) introduced the term Quantitatively Engineered Grout Curtain (QEGC) design approach. The requirements of this design are dam safety implications, impacts and value of the control of the grouting pressure and value of lost water, which is obtained by cost-benefit analysis and political/public acceptability. In this method, by assigning quantitative parameters to geologic units, requirements, and characteristics of the grout curtain, it is possible to perform a preliminary design for determining the need for grouting, required depth of the grouting and relative benefits of performing the work at higher or lower level of quality by considering the cost impact (Wilson and Dreese, 2003). The finite element method is used for the final design, where complexities with geology and topography are modeled.

2.6 New developments

With the aim of examining the mechanism of fracture deformation, the theoretical approach, which previously was introduced by Gustafson and Stille (2005), was developed further (see Brantberger et al 2000, Stille, et al., 2012 and Rafi and Stille, 2015[1]). In this process, the distance of the grout spread is interchanged with the injected volume in the GIN method and the resultant force due to a combination of grout spread and injection pressure is estimated and compared against the in-situ stresses. Thus, the state of the fracture in real time could be estimated. Furthermore, the amount of deformations along the fracture in real time is estimated, i.e. the profile of the fracture is visualized. This information is significantly helpful in determination of the pumping pressure. In the following chapters, the focus is on justifying this process by applying it on cases from Scandinavian projects and discussing the optimum applicable pressure in these projects.

3 PUMPING PRESSURES AND IN-SITU STRESSES IN THE ROCK MASS

As discussed before, there are different opinions concerning the choice of pressure. Applying a low pressure confirms the safety of the process, i.e. no hazard to over-ground structures. However, it may take a long time to complete the process. On the other hand, a higher pressure may cause some deformations that can increase the penetrability, especially in smaller fractures but this may negatively affect the sealing of the fracture system. Rafi and Stille (2014) discussed the stress- deformation behavior of the fracture, obtained from fractured rock mass loading and unloading experiments (e.g. Bandis et al., 1983). Based on that, they examined the fracture dilation in two phases; the first phase where when the grout pressure is less than the stresses in the rock mass denoted the critical pressure and the second phase where when the grout pressure exceeds the critical pressure. In this chapter, the grout spread, the penetrability and the sealing of the fractures at each of these phases are discussed.

3.1 Applying a pressure less than the in-situ stresses

In this case, the grouting pressure does not exceed the stresses in the rock mass and does not enable to open the fractures, i.e. the fracture apertures are constant. Hässler (1991) discussed the stiff plug of the grout when he examined the spread of a Bingham material between constant boundaries. According to him, in a Bingham material flow between constant boundaries can only take place in a part of the fluid, which means that a stiff plug is formed in the center of the flow channel surrounded by a plastic flow zone. This plug grows to the size of the aperture at refusal (Figure.3.1). This model predicts the velocity of the grout in the fracture based on the size of the plug. The solution for the grout spread is successively solved for a number of time steps or front positions during the grouting procedure. Thus, by considering suitable time steps, the velocity at each section and the corresponding position of the fronts are estimated.

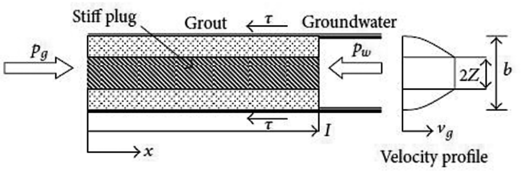

Figure 3.1 Grout penetrating a fracture (After Gustafson, et al., 2013)

To consider the spread of grout in joint sets, a numerical model was developed (Hässler, et al., 1991 and Håkansson, et al., 1992), where equations of the grout velocity at the intersection of the fractures were solved together in form of a matrix. The model considered perpendicular joint sets with constant spacing (Figure.3.2). Later Saedi, et al.

(2013) discussed the effect of the different parameters such as orientation and persistence of the fractures on the grout propagation by a similar numerical model (e.g. Figure. 3.3).

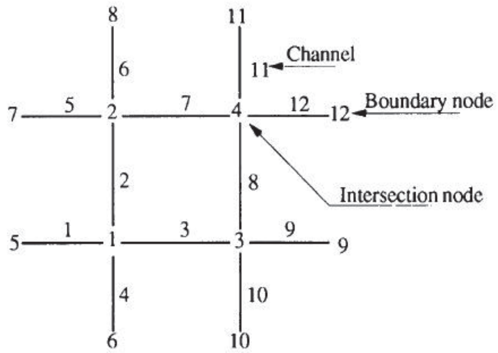

Figure 3.2 Defining a network of fractures as boundary node, intersection node and channel for establishing the matrix (after Hässler, 1991)

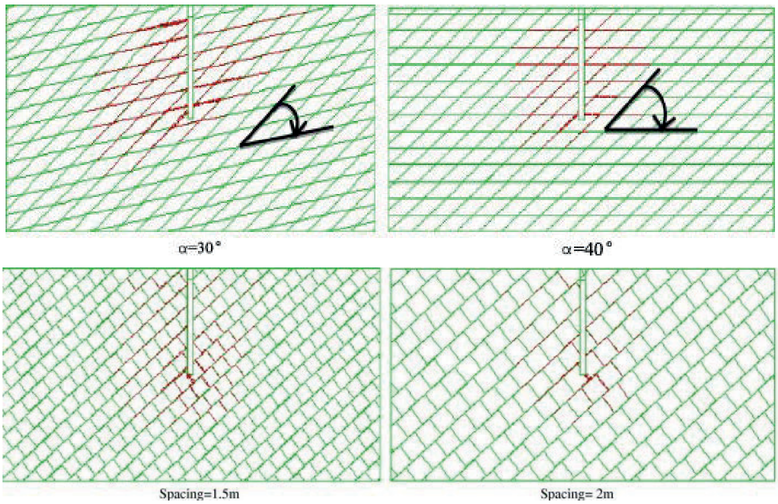

Figure 3.3 The effect of joint dipping on grout flow and grout penetration (red lines) around a borehole with different dipping (after Saedi, et al., 2013).

In order to correlate the variation of plug size to grout spread, Gustafson and Stille (2005) defined the relative penetration (I_D), based on the size of the plug. This was based on a theoretical work that was published later (Gustafson, et al., 2013). They correlated I_D, which is a unit less parameter which indicates the advancement of a grout mix with certain properties and is independent of fracture aperture, to the grouting time, *t*, with t_D (relative time). t_D is also a unit less parameter and is the ratio of the grouting time to the characteristic time (t_D=t/t_0). The characteristic time is defined based on the grouting pressure and the properties of the grout, as defined in Eq. 3.1.

$$t_0 = \frac{6\mu_g \Delta P}{\tau_0^2} \tag{3.1}$$

In Figure 3.4, this correlation has been depicted for simplified flow systems: one dimensional flow (1D) that stands for unidirectional flow in channels, and two dimensional (2D) which is the radial flow around the borehole.

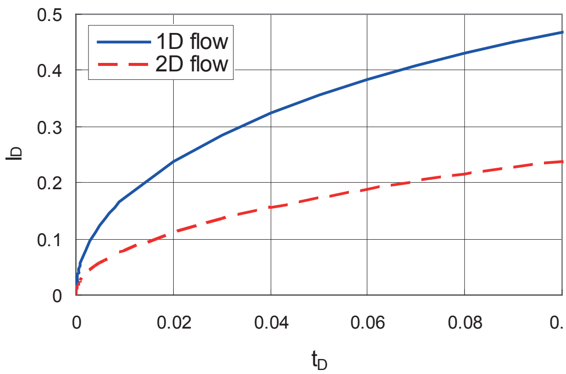

Figure 3.4 Relative penetration as a function of relative grouting time for different dimensionalities (Gustafson and Stille, 2005)

I_D is the advancement of the grout independent of the fracture size and shows how much of its maximum penetration (I_{max}) a certain grout mix under a certain pressure can proceed. (I_{max}) is the maximum possible grout spread in a fracture with aperture b and results from a force balance between effective grout pressure (difference between grouting and resisting water pressure ($\Delta P = P_g - P_w$) and the shear force developed against the walls of the fracture as defined below:

$$I_{max} = \frac{\Delta P \cdot b}{2\tau_0} \tag{3.2}$$

Where b is the size of fracture aperture and τ_0 is the yield value of the grout mix. Thus, if the aperture is known, the distance of the grout spread can be estimated in real time (I= $I_D \cdot I_{max}$). However, measuring the fracture aperture is challenging. Several efforts have been made regarding the estimation of the fracture aperture. One of the common methods is water pressure test. The estimated aperture with this approach is called the hydraulic aperture. The Lugeon value, which is defined as water loss in liter per meter per minute for the applied pressure of one MPa, is obtained through field tests. This value is interchangeable to transmissivity by the cubic law defined as:

$$T = \frac{\rho g}{12\mu}\sum b^3 \tag{3.3}$$

Where b is the size of the fracture aperture, ρ is the density of the rock mass and μ is the viscosity of the mix. Barton, et al. (1985), suggested an empirical equation for estimating the initial mechanical aperture for joints based on rock strength and joint roughness. They also proposed a correlation for an equivalent smooth wall aperture and conductivity as an indirect approach for estimating the aperture size. Hakami and Larsson (1996) measured the aperture size by statistical analysis of microscopic images from exposed fracture profiles. The results indicated that the mean aperture of the studied fracture is 1.4 times larger than the hydraulic aperture. Dershowitz, et al. (2003), correlated the void filling aperture to transmissivity with empirical coefficients. They introduced coefficients to correlate the hydraulic aperture which corresponds to purely parallel and smooth plates. According to the field results, they found these coefficients larger for relatively rougher fractures, and thus concluded that the void filling aperture is larger than the hydraulic aperture. Carter, et al. (2012), illustrated that for fractures with larger transmissivity, the void filling aperture is significantly larger than the hydraulic aperture, i.e. the cubic law (Eq.3.3) significantly underestimates the aperture size. Through a case study in the Stockholm City Line project, Tsuji, et al. (2012) confirmed that the estimated apertures based on flow of a Bingham fluid in the fractures are larger than the obtained ones from water pressure test. The reason is the lower velocity of the grout flow, which let the voids between the contact points of the fracture to be filled. From the scattered data in Figure.3.5, it can be seen that a multiplier of 1.2 to 2.5 is required to change the hydraulic aperture to the void filling aperture.

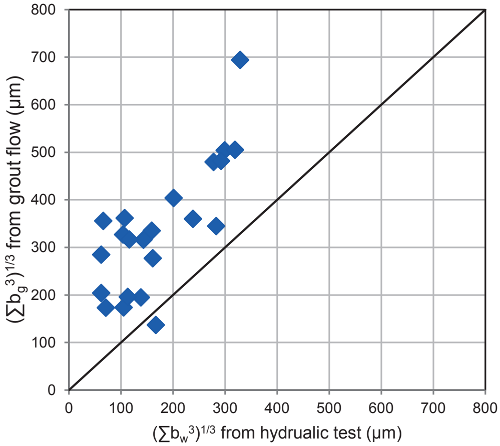

Figure 3.5 Comparison of fracture aperture size obtained from the Lugeon test and the Real Time Grouting Control method, which consider filling the voids with Bingham material using data from the Stockholm City Line project. It is illustrated that the latter measurement method estimates a larger fracture aperture size (After Tsuji, et al., 2012).

In the present study, the size of the fracture aperture is estimated based on the flow of a Bingham fluid (see Gustafson and Stille, 2005). This is possible by expressing the injected volume of the grout based on the fracture aperture size (Eq. 3.4). The estimated aperture size is the average of the void spaces and is larger than the estimated aperture size estimated by water pressure test. Note that this is based on the assumptions that e.g. the yield stress and viscosity are constant. However, these parameters may change with time which could introduce an additional uncertainty into this estimation. It should also be noted that several fractures may intersect the grouting borehole and are grouted simultaneously. The aperture sizes of these fractures are not the same in most of the cases but the largest fracture is dominant for most of the grout flow, since the flow of the grout increases with b^3. Hernqvist (2014) has confirmed this by showing that most of the transmissivity is due to the largest un-grouted fracture. Therefore, in the studied cases in this report, the assumption is that 80 percent of the flow passes through the largest fracture aperture and by that, the fracture aperture size could be estimated by Eq.3.4.

$$V_{tot}(2D) = I_D^2(2D) \cdot \pi \cdot I_{max}^2 \sum b^3 = I_D^2(2D) \cdot \pi \cdot (\frac{\Delta P}{2\tau_0})^2 \sum b^3 \tag{3.4}$$

The flow of the grout in the fracture is possible to be estimated by using the estimated aperture (Eq. 3.5). Comparing this estimated flow with the recorded values during grouting indicates how well the simplified geological model, which consists of one horizontal fracture, can model the existing joint sets.

$$Q\ (2D) = 2I_D \cdot \frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \pi \cdot (\frac{\Delta P}{2\tau_0})^2 . \sum b^3 \tag{3.5}$$

The penetrability of the material into this estimated fracture is another issue that should be taken into consideration. The penetrability meter test is standard equipment developed by Eriksson and Stille (2003) for measuring the aperture limits. The penetrability meter instrument consists of a container and an attached pipe which is equipped with a valve and a cap holder. Different mesh filters can be installed in the cap. A pump is used to apply the pressure on the grout. To perform the test, filters with defined mesh size are being installed in the cap and when a pressure of one bar is reached the valve is opened. The grout which is passed through each filter is collected. The biggest aperture with no pass of grout is called the minimum aperture (b_{min}).

Using different filters, the test continue and the volume of grout passing through each filter is registered. The specific filter through which a certain maximum volume pass is considered as the critical aperture (b_{crit}). The volume of 1000 ml refers to the volume of grout that is necessary to be measured to ensure that no filter cake forms, i.e. that an infinite volume theoretically can pass the filter (Eriksson, et al., 2004).
The penetrability of the material can be examined by comparing the b_{min} and b_{crit} with the largest aperture estimated with Eq.3.4 (b_{max}). A b_{max} close to b_{min} means there are many fractures with aperture smaller than the b_{min} which will remain un-grouted. On the other hand, a b_{max} larger that b_{crit} means that no filtration will occur. With this knowledge, it is possible to determine the penetrability properties of the grout material. This is necessary to be able to fill most of the existing fractures.

Around the tunnel, as mentioned by Hernqvist, et al. (2012), only sections with fractures that are needed to be sealed should be grouted. Sealing the conductive fractures should result in an inflow reduction in line with target values. No grouting is needed in sufficiently tight sections based on hydraulic tests. The size of the smallest fractures required to be sealed depends on the allowable inflow after grouting. This inflow can be examined by measuring the transmissivity before and after grouting from the equation developed by (Bergman and Nord, 1982) as below:

$$Q = \frac{2\pi \cdot T_0 \cdot H/L}{\ln\left(\frac{2H}{r_t}\right) + \left(\frac{T_0}{T_{gr}} - 1\right) \cdot \ln\left(1 + \frac{t}{r_t}\right) + \xi} \tag{3.6}$$

where Q is the calculated inflow of water per unit length of the tunnel (m^3/s per m), T_0 is the transmissivity of the un-grouted rock mass section (m^2/s), T_{gr} is the transmissivity of the grouted zone surrounding the tunnel section (m^2/s), H is the groundwater head (m), L is length of the tunnel section (m), r_t is tunnel radius (m), t is the thickness of the grouted zone surrounding the tunnel (m), and ξ is the skin factor. The transmissivity is estimated through Eq.3.3.

In order for a grouting to be successful, it is not only to fulfill the required sealing efficiency, but also to avoid overspread of grout, which negatively might affect the economy of the project as well as the surrounding environment. But, as depicted by (Funehag, 2007), the grout should spread a minimum distance in small and large fractures (Figure 3.6). It should be noted that in this case the intersection of fractures has not been considered. Defining stop criteria based on a minimum and a maximum spread of grout would be a solution for these obstacles.

A theoretical approach makes it possible to estimate the distance that grout spread in the fracture in real time. Therefore, a certain minimum distance in the smallest grout-able fractures and a certain maximum distance in the largest existing fractures can be the optimum stop criteria. This is reasonable since the fracture aperture and its trace are directly correlated. As a consequence, the grout should travel a shorter distance in finer fractures. However, a longer time may be required to achieve even this short distance compared with the required time for the grout to penetrate the maximum longer distance in large fractures. Therefore, the stop criteria can be refined based on the result of trial tests and imposing limitations for the grouting time and the pumping pressure to achieve a minimum distance of penetration in the fractures, and also to avoid overspread in larger fractures. Re-grouting may be required with the purpose of filling up finer fractures to a certain minimum distance (I_{bseal}) if the limiting minimum distance of penetration in large fractures (I_{bmax}) arrives first (Figure.3.7). It means that to avoid overspread of grout, grouting of relatively finer fractures may need to be performed as re-grouting (This can be the second round of grouting at the same time and even before any test, since not filling of small fractures are confirmed).

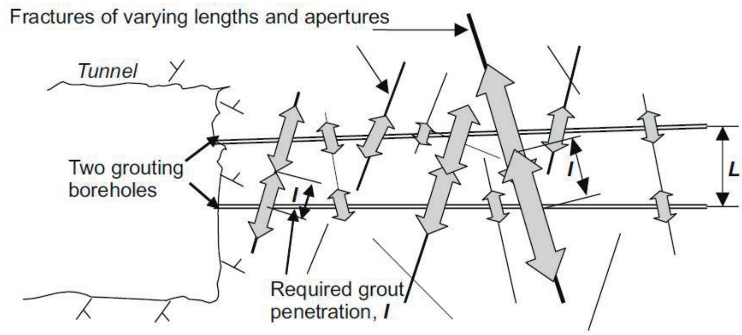

Figure 3.6 Illustration of the required minimum grout penetration. L is the distance between the grouting boreholes (after Funehag 2007).

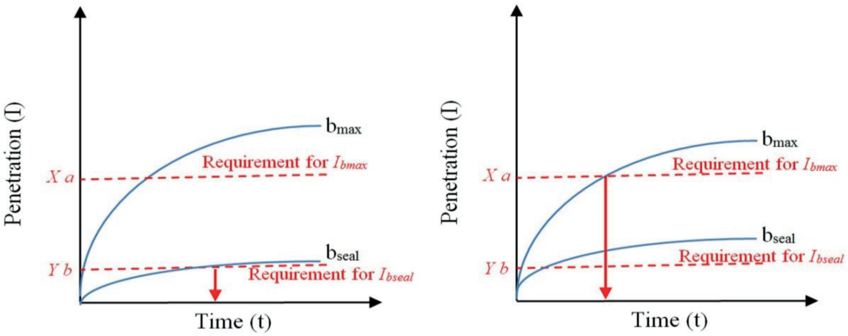

Figure 3.7 Illustration of the basic stop criteria. To the left, grouting is stopped when the distance of grout spread in the finest fracture to be sealed (I_{bseal}) exceeds the design requirements. To the right, grouting is stopped when the grout spread is equal to the maximum allowed (Tsuji, et al., 2012).

The Real Time Grouting Control Method is a suitable tool for preliminary estimation of design parameters (pumping pressure, completion time of grouting and material properties) and examination of the performed grouting program. In this process, pumping pressure and grout flow are recorded in time steps with the Logac machine in the field. The rheological properties of the grout mixture, the yield value (τ_0) and the viscosity (μ), are measured in laboratory and are assumed constant during the grouting period. By that, the relative penetration of the grout at each time step is estimated (Figure. 3.4).

Except for the preliminary design of grouting parameters, the Real Time Grouting Control method is a valuable tool in applying the observational method (Stille, 2012), where the performance of the grouting work is evaluated with back analysis. In this approach, based on recorded pressure and flow data as well as injected grout volume, the fracture aperture size and the dimensionality of the grout at different grouted holes are approximated. Finally, by estimation of the corresponding time to obtain the required distance of penetration, a stop criterion based on the grouting time for the next section of grouting in similar geology is defined. The stop criterion is revised during the execution of the grouting project, i.e. the required grouting time is adjusted as more grout sections are examined.

The above described process can be performed on-line which is a robust approach compared with back analysis. In this purpose, the distance of the grout penetration is estimated in real time during the grouting procedure. However, as mentioned by Holmberg, et al. (2013), this approach requires that on-line analysis can be used as an integral part of the grouting operation, which to date is not a market standard. Nevertheless, the possible procedure for this approach has been depicted in Figure. 3.8. In this procedure, the recorded applied pressure and flow of grout at the first time step, as well as the rheological properties of the grout material, which are obtained from laboratory tests, are the initial input data. By that, the fracture aperture size and the spread of the grout are estimated at certain time steps. Comparing the recorded and the estimated flow provides the chance of adjusting the assumption about dimensionality, and distinguishing probable fracture deformations. The same procedure is repeated in next time steps and the estimated size of the fracture aperture is adjusted at each round. This means that in the online procedure, the flow of the grout at each time step provides information about the porosity of the rock mass, which is helpful to more accurately estimate the dominant fracture aperture. The grouting is stopped as soon as the requirement of the grout penetration is achieved.

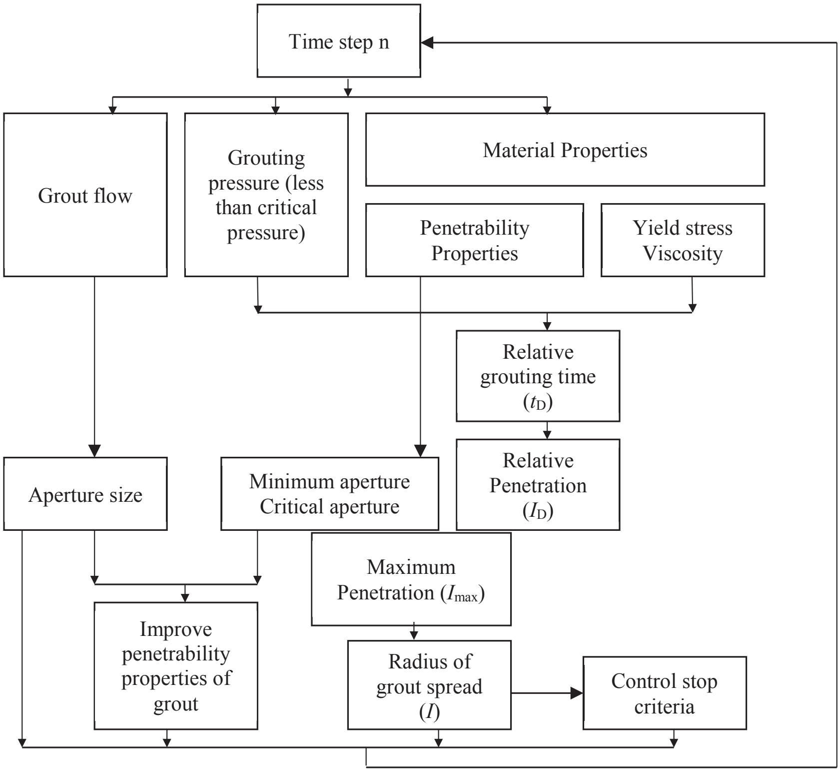

Figure 3.8 Online application of the Real Time Grouting Control Method

3.2 Applying a pressure larger than the in-situ stresses

The grouting pressure induces stresses in the rock mass, and in case of exceeding the in-situ stresses, it leads to fracture deformation. Gothäll and Stille (2009) described this process, from the onset of injecting grout in the fracture to dilation of the fracture, in three phases. Considering the rough surface of the fracture, at the first stage, the major part of the load from the in-situ stresses is carried by the asperities at contact. When grouting continues, the pressure of the grout reaches the in-situ stresses, which is named the critical point. If the grouting pressure exceeds the in-situ stresses (the so called critical pressure), the fractures start to open up. If the grouting continues, this deformation becomes larger and may lead to an uplift of the rock mass. If this process involves unrecoverable deformation of the fracture, then there may be an increase in the rock mass permeability as a result of the grouting, rather than a decrease (Emmelin, et al., 2008).

Lombardi and Deere (1993) showed that it is not only the grouting pressure that deforms the fractures of the rock mass, but also a combination of the pressure and the volume of the injected grout that generates the energy. The reason to use the injected grout volume in the GIN method instead of the theoretically correct distance of penetration was practical. In an attempt to develop the GIN method, Brantberger, et al. (2001) defined the GIN value based on the distance that the grout penetrates into the fracture.

To lift the rock mass, the grout pressure needs to become three times the overburden, since it acts in a cone shape on the fracture's wall. Thus, the ratio of grouting pressure to three times the overburden ($P_g/3\rho gh$), which is called the normalized pressure (P_n), is equal to 1 or larger when the rock mass is moved. The ultimate jacking limit when the rock mass is lifted has been devised by Brantberger, et al. (2001) as shown in Eq.3.7. In the equation, the normalized penetration (I_n) is the ratio of the distance of grout penetration to the vertical distance from the surface to the intersection of the borehole and the fracture ($I_n=I/h$). P_w is the water pressure in the rock mass. From this equation, it can be seen that a relatively larger pressure is applicable at relatively smaller distances of grout penetration, and as grout penetrates further, a smaller pressure is allowed. If the grouting procedure continues beyond the ultimate state and beyond that ($P_n>1$), deformations are permanent.

$$P_n + \frac{P_w}{\rho gh} \leq 1 + \frac{1}{I_n} + \frac{1}{3I_n} \tag{3.7}$$

With the development of the Real Time Grouting Control method, the estimated distance of penetration in real time is inserted into Eq. 3.7, and by that the grouting pressure to reach the desired distance of penetration in shortest time while avoiding uplift of the rock mass is obtained. However, according to Gothäll and Stille (2009), the fracture will be dilated at much smaller pressure, theoretically when the grouting pressure exceeds the critical pressure ($P_n>1/3$). Similar to the ultimate limit state, the acceptable limit state is defined by Stille, et al. (2012) in order to limit the fracture deformation to a certain pre-defined value (Eq.3.9).

$$P_n + \frac{P_w}{3\rho gh} \leq \frac{k}{3I_n} + \frac{1}{3} \tag{3.8}$$

Where

$$k = \frac{3}{4} \cdot \frac{E}{(1-\vartheta^2)} \cdot \frac{\delta_{accept}}{\rho gh^2} \cdot \frac{\Delta P_g}{P_e} \tag{3.9}$$

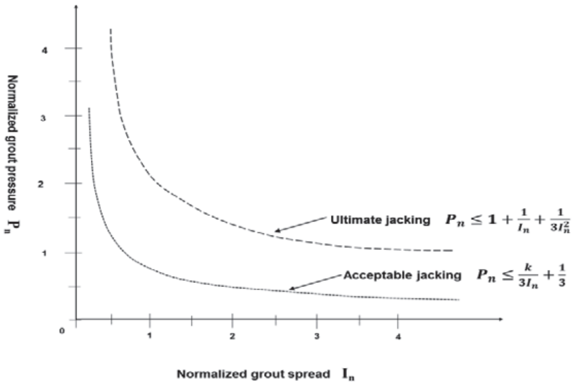

Figure 3.9 Maximum normalized pressure as a function of normalized grout spread for both the ultimate limit state (Eq.3.7) and the acceptable limit state (Eq.3.8). The curves are calculated for $P_w = 0$ (after Stille, et al., 2012).

By assuming the distance of the horizontal fracture to the surface (h) in addition to the geological properties (density of the rock mass, modulus of elasticity and Poisson's ratio), as well as the permitted deformation, the ultimate and elastic jacking limits are established (Eq.3.7 and Eq.3.8). The coordinates of the normalized pressure-normalized penetration specifies the state of the fracture. The scattered data of the estimated normalized distance of the grout spread (I_n) at different elapsed times is plotted versus the normalized pressure (P_n) and examined against the theoretical jacking curves. Therefore, the onset of jacking can be distinguished.

Shaping the profile of the fracture after deformations have occurred is helpful in examining the significance of jacking. By considering the deformations of the rock mass as the deformation of a half-infinite space, Gothäll and Stille (2009) developed an equation, which describes the deformation at a sufficiently large distance from the borehole as:

$$\Delta a(r) = \frac{4}{3} \cdot \frac{P_e}{E} \cdot \frac{r_c^2(1-\vartheta^2)}{r} \tag{3.10}$$

Where

$$r_c = \frac{P_e}{P_g} \cdot I \tag{3.11}$$

r_c is the distance where the excess pressure (P_e) acts, and dissipates beyond that (Figure. 3.10)

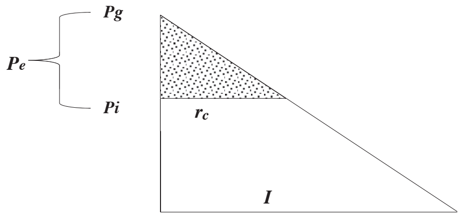

Figure 3.10 Excess pressure (P_e) is due to the difference between grouting pressure and critical pressure (dotted zone) (after Rafi and Stille, 2014).

Considering an infinite solitary fracture inside an infinite homogenous rock mass, the deformation along this distance would be constant (Eq.3.12) and is estimated by inserting r_c in Eq.3.10.

$$\Delta a_j = \frac{4}{3} \cdot \frac{P_e r_c}{E} (1 - \vartheta^2) \tag{3.12}$$

The profile of the fracture has been depicted by (Gothäll and Stille, 2009) as shown in Figure 3.11.

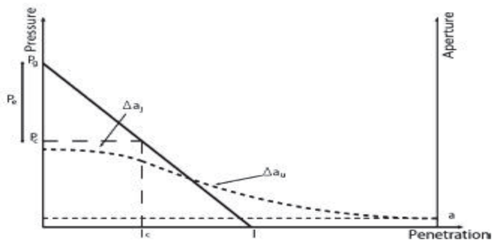

Figure 3.11 The principles of pressure and deformation. In this model the X-axis represents the radial distance from the bore hole. The pressure drops approximately linearly with distance and becomes zero at the grout front. The deformation is approximated by Δaj in distance that pressure acts and beyond that is varied (After Gothäll and Stille 2009).

This deformation might be favorable in opening the fractures up to a critical aperture that facilitate the penetration of the grout, i.e. an increase in penetrability (which means a certain mix can penetrate into the smaller fractures, since these fractures are enlarged due to jacking). However, a deformation over the critical aperture will not improve penetrability anymore (Figure. 3.12).

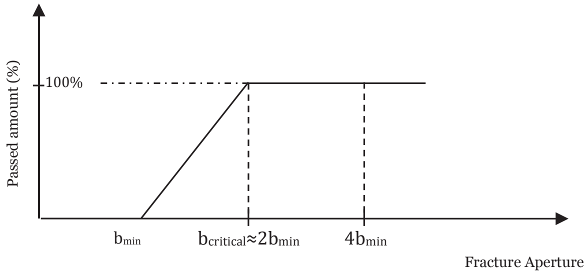

Figure 3.12 Increasing the aperture size beyond the size of the critical aperture will not improve penetrability (after Rafi and Stille[1], 2015).

As discussed before, the Real Time Grouting Control method provides the opportunity of estimating the initial fracture aperture size based on the flow of grout and consequently calculating the corresponding grouting time to the required radius of grout spread in this fracture. However, since the applicable pressure has been approximated to be larger than the critical pressure in order to give the permitted fracture deformation when injecting grout at a constant flow rate, the grouting time will be prolonged. (Rafi and Stille[1] (2015) discussed that this prolongation in grouting time is due to the decrease in the distance of grout spread since an extra volume is developed due to the fracture deformation. Thus, a part of the grout is consumed to fill up that volume. As a consequence, only a part of the grout travels forward (Figure 3.13). Rafi and Stille[1] (2015) proposed to estimate the decrease in depth of grout spread by establishing equilibrium between the injected grout volume (V_{inj}) and the volume of the void needed to be filled, which consists of the volume of the initial aperture (V_{b}) and the discussed extra volume (ΔV).

$$V_{inj} = \Delta V + V_b \tag{3.13}$$

It means that ΔV_{inj}, the injected volume in deformed fracture, is the volume of constantly flowed gout.

$$V_{inj} = Q.\Delta t \tag{3.14}$$

V_{b} is the volume of the grout that initially flows in the fracture with aperture size of b i.e. the volume of grout that moves forward, which in the case of radial flow is the volume of the disk of grout spread around the borehole with a radius of I (Eq.3.15).

$$V_b = \pi b I^2 \tag{3.15}$$

The volume of the space caused by fracture deformation is written based on the depth of the grout spread according to Eq.3.16 (Rafi and Stille[1], 2015)

$$\Delta V = \pi \left(\frac{4}{3}\right) P_e r_c^2 \left(\frac{1-\vartheta^2}{E}\right)(2I - r_c) \tag{3.16}$$

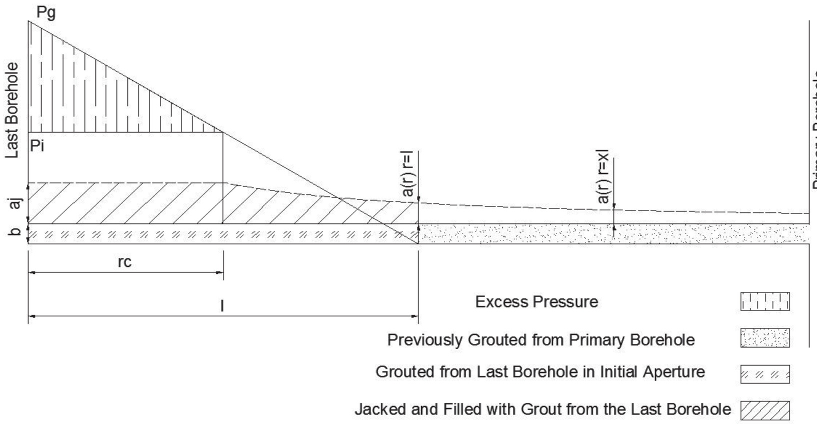

Figure 3.13 Mechanism of elastic jacking. Fracture deformation is extended outside the grouted zone, which may open the previously grouted section from primary borehole (after Rafi and Stille[1], 2015).

Thus, by establishing the equilibrium of Eq. 3.13, the only unknown is the penetration length (I) after fracture deformation. Figure 3.13 shows that the extent of deformation outside the grouted zone due to the load redistribution will open up previously grouted fractures that may lead to a remaining un-grouted area, if no grouting from adjacent boreholes is performed afterwards. This will lead to an increase of transmissivity and consequently an increase of water inflow into the tunnel.

3.3 Applicability of using the GIN method

In addition to the discussed theoretical approach in this chapter, there are empirical methods that are common in today's practice. Among them, the GIN method is the popular method (mostly in Europe) and has been tried in many projects. However, there are limitations connected to this methodology. With the purpose of getting a deeper understanding of these limitations, Rafi and Stille[2] (2015) examined the GIN hyperbola versus the theoretical jacking curves. The main difference between these two is that the GIN hyperbola is dependent on the aperture size. It means that when fractures with different aperture sizes are grouted, the pressure-volume recorded data intersect the hyperbola differently in different sections along the hyperbola. This can imply that by moving along the hyperbola (decreasing the pressure as injecting grout), different sizes of apertures are filled. It was shown by Rafi and Stille[2] (2015) that by stopping grouting at a certain point, the maximum penetration length is attained in fractures smaller than a certain size. Furthermore, the grout penetrates in all of the penetrable fractures during grouting but at different distances which depends on the size of fracture aperture, but the percentage of the maximum penetration length (I_D) is the same (Figure 3.14).

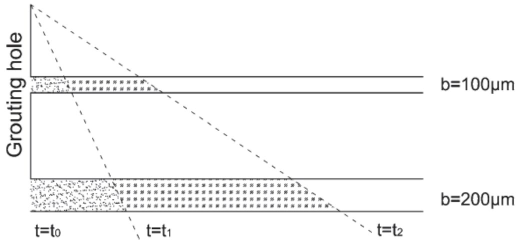

Figure 3.14 When several fractures are grouted, the same percentage of maximum penetration length is traversed at each time interval for all the fractures, regardless of the aperture size (after Rafi and Stille[2], 2015)

In contrast, the theoretical jacking limits are independent of the aperture size. In other word, a certain curve is defined for a certain aperture size, since the jacking limit has been defined based on the distance of the grout spread. Therefore, there is one point that corresponds to the required spread for a certain fracture aperture size with a certain deformation, and this point is the intersection of the GIN hyperbola with an elastic jacking curve that limits the fracture deformation to a certain value (Figure 3.15).

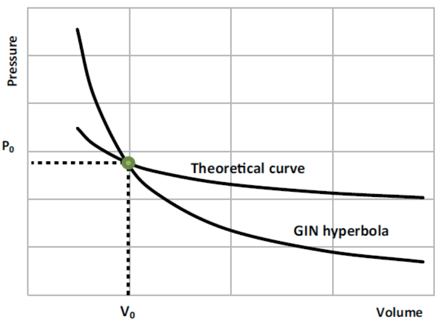

Figure 3.15 Grouting is completed at the intersection of the GIN hyperbola and the theoretical curve, where the injected volume of V_0 confirms a sufficient spread of grout and the pressure of P_0 limits the fracture deformation to the permitted level (after Rafi and Stille[2], 2015).

With this background, the GIN can be formulated based on the distance of the grout spread.

$$P \cdot V < GIN = \rho g h \pi (b + \delta_{acc})(khI + I^2) = \rho g h \pi I^2 (b + \delta_{acc})(k\frac{h}{I} + 1) \qquad (3.17)$$

Where

$$k = \frac{3}{4} \cdot \frac{E}{(1-\vartheta^2)} \cdot \frac{\delta_{acc}}{\rho g h^2} \cdot \frac{P_g}{P_e} \qquad (3.18)$$

Rafi and Stille[2] (2015) emphasis that defining the GIN based on the spread of grout can explain some of the ambiguities connected to this method. As the optimum procedure, they suggested to decrease the pumping pressure once the hyperbola was intersected, and stop grouting at a pressure smaller than the critical pressure in order to bring the fracture back to its initial size. It was concluded that although the GIN method has been successful under many conditions, it should be used with caution when grouting shallow fractures and/ or fractures that have to be highly sealed.

3.4. Application of the theoretical approach in tunneling projects

The Real Time Grouting Control method has been used to examine the performance of the grouting work in both tunneling and dam construction projects (e.g. see Tsuji, et al., 2012 and Rafi, et al., 2012). Furthermore, the advantages and disadvantage of the elastic jacking have been discussed considering a single horizontal fracture under a dam structure. However, consequences of this fracture deformation around an underground excavation are not known. Considering joint sets around the opening, only some of the fractures are perpendicular to the opening face and can lead the water into the excavation. Furthermore, boreholes in the grouting fan may only intersect some of these fractures. It means that the effect of a high applied pressure, and consequently the effect from elastic jacking of the fractures on the inflow of water into the excavation is difficult to be analyzed by only considering the theory described above. In this case, there is a need to compare the inflow data as well considering the order of grouting holes in order to be able to draw any conclusions on this problem.

4. METHODOLOGY

A theoretical approach, discussed in chapter 3, has been applied in two similar tunnel projects, one in Sweden and one in Norway. In the Swedish project (Stockholm City Line project), a lower grouting pressure has been used compared to the Norwegian project (Holm-Nykirke). The consequences, according to the applied theory, of the variation in grouting pressure have been compared. Based on the results from this comparison, a discussion concerning an optimal applicable grouting pressure is performed.

The methodology for the analysis of each project, before the comparison between the projects are performed, has been divided into five steps: (1) The geology and the requirements of the project are first evaluated; (2) It is examined if the theoretical approach is applicable in the projects, (3) The theoretical approach is used to calculate the effect of the grout pressure, (4) A statistical study of the results is performed; (5) The effect of the grouting pressure is evaluated. In this section, these steps are described shortly (Figure 4.1.). In the end of the chapter, a general comparison between requirements and grouting philosophy between Sweden and Norway is performed, and the reasons for performing a comparison between two grouting projects in these countries, based on the theories presented in chapter 3, is presented.

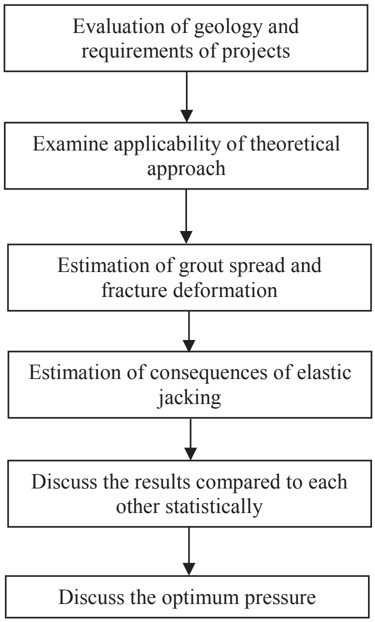

Figure 4.1 Methodology used in this research work

4.1 Geology and requirements of the project

One of the major issues in the determination of the design parameters for the grouting is the geology and the prevailing in-situ stresses. As mentioned before, the application of the Real Time Grouting Control (RTGC) method needs to be adjusted to the characteristics of the fracture system and the mechanical properties of the rock mass, since these elements affect the flow of the grout and the applicable grouting pressure. That is, the aperture size, the persistence, the orientation of the fractures, the depth that the fractures is situated in, the acting in-situ stresses and the strength of the rock mass all affect the results of the grouting work. Thus, the first step in this study is examining the geology of the studied projects in order to understand the similarities and the differences between the projects, i.e. if and in what way the final results are, or are not, comparable. In the next steps, the functional and environmental requirements of the projects, especially the sealing requirement, is defined. This makes it possible to evaluate the performance of the grouting work quantitatively. Also, the need for using a more precise method is distinguished by using Table 4.1.

Table 4.1 Degree of difficulty as a function of required sealing effect and conductivity of the grouted zone (after Stille 2012)

Required sealing efficiency *Required conductivity*	*< 90%*	*90-99 %*	*>99%*
>10-7 m/s	Uncomplicated grouting	Fair grouting	Difficult grouting
10-7 to 10-8 m/s	Fair grouting	Difficult grouting	Very difficult grouting
< 10-8 m/s	Difficult grouting	Very difficult grouting	Very difficult grouting

4.2 Applicability of the Real Time Grouting Control method

Besides the similarities in geology and the requirements of the chosen projects, it is important to examine if the Real Time Grouting Control Method (the method that estimates the flow of grout and distance of grout penetration in real time) is applicable in the chosen projects, since without this method the comparison would not be meaningful. As mentioned before, this method simplifies the geology of the area to one horizontal fracture that most of the grout flow through. The validity of this assumption, i.e. the applicability of this method given the specific geological conditions in the projects, is investigated by calculating the grout flow in this fracture (by using Eq.3.5) and comparing it with the recorded flow data.

4.3 Design with the theoretical approach

The theoretical approach is used to examine the effect of the grout pressure on the efficiency of the grouting process as well as the final outcome by using the outcome of RTGC, i.e. the estimated distance of the grout spread. Two sets of laboratory data (rheological and penetrability properties of the grout mix) and field data (recorded flow, grout pressure and measured water flow) are used as inputs in the application of the RTGC-method. The aperture size of the representative fracture calculated based on the flow of the grout, the flow of the grout, and the distance of the grout penetration in this fracture are given as output data. The calculated distance of the grout penetration in combination with the grout pressure gives an estimate about the state of the fracture with respect to jacking and the amount of fracture deformation that occur.

4.4 Statistical analysis of the results

Analyzing the results statistically is a useful tool for discussing the performance of the initial design and the changes required in order to optimize it. Scattered data of the amount of injected volume and the calculated spread of the grout in the boreholes of a section of the tunnel indicates if the fractures were possible to grout and if the grout spread was sufficient. The corresponding grouting time to obtain the required distance of the grout spread in most of the boreholes (the chosen frequency depends on the requirements of the project) is compared with the grouting time. Furthermore, the ingress of water into the tunnel before and after grouting can be compared statistically to examine the efficiency of the sealing in the whole section. Other statistics that are analyzed are the distribution of the estimated representative fracture and the maximum applied pressure when grouting.

4.5 Evaluation of effect of the grouting pressure

The effect of the applied pressure is evaluated theoretically and/or by examining the function of the final product. In this project, based on a theoretical approach, the positive and negative consequences of the applied pressure in each of the projects are estimated quantitatively by calculations. Increase of the penetrability due to fracture deformation is considered as the positive effect of using high pressure, which is evaluated by considering the decrease in grouting time due to this deformation. However, this deformation may affect the grouting process negatively since it enlarges the aperture size of the fracture that is needed to be filled with grout, a factor that is also taken into consideration with the applied theory. Another possible negative consequence of this deformation, which is calculated and discussed in this work, is the increase of the transmissivity due to the opening of the previously grouted fractures, i.e. the sealing efficiency of the grout applied in previously grouted boreholes are disturbed.

Discussing these theoretical results with respect to the measured inflow of water into the tunnel after grouting and the grouting time in the boreholes can confirm if the proper grouting pressure has been used in the project. By comparing the results between the two projects, a better understanding in the choice of an optimal grouting pressure in similar projects might be possible.

4.6 General comparison between Swedish and Norwegian projects

Grov et al. (2014) performed a comparative study between Norwegian and Swedish tunnel projects with the focus on the use of grouting pressure. They found that several similarities exist in the tunneling projects in these countries. In both countries, the inflow ahead of the face is controlled by pre-grouting through a 360º hole setting. In these projects the aim is achieving the required sealing efficiency in one round of grouting. However, if the flow from control holes exceeds the initial criteria, another round of grouting is performed. In these systematic pre-grouting projects, the records of drilling and grouting are kept to be used in decision makings. Furthermore, the effort is remaining loyal to the grouting strategies in Norwegian projects, while the observational method is used in Swedish projects, where new decisions can be made during the execution of the grouting. Another difference in Norwegian projects compared to Swedish projects is that they tend to refine the grouting strategy by using probe holes ahead of the face to obtain additional information. Furthermore, the spacing of the boreholes varies depending on the complexity of the hydrogeological conditions and the groundwater inflow rate criteria. However, in Norwegian projects the spacing of the boreholes is seldom varied (3-4 m spacing and a packer mounted 1 m inside the borehole). Regarding the grout mix, the usage of micro cement with water cement ratio of 0.5 to 0.9 is the usual choice in Norway, while in Sweden; Portland cement with grain sizes around 30μm with a water cement ratio of 0.8 to 1 is common. Regarding stop criteria, the main stop criterion in Norwegian projects is injected grout volume. In recent Swedish projects, however there are limitations on the grout pressure and grouting time that are the main stop criteria.

4.6.1 Geology:

Most of these two countries are covered by Precambrian rocks (older than 545 million years). The most common rock type of these old rocks is the gneiss. Other rock types from this era are granite, gabbro and quartzite. This type of rock covers two third of Norway and almost 9/10 of Sweden. The Precambrian shield is heavily fractured due to the long period of geological and tectonic events and through a stretch of some 100 meters it is common to find at least a few larger fracture zones, sometimes with clay fillings making the progress of a tunnel difficult, both mechanically and hydro geologically. The rock mass is consequently a very typical jointed aquifer where water are present along the most permeable discontinuities. With high ground water level, tunneling works are normally taking place in saturated conditions with the risk of disturbing the ground water balance imposing consequential damages (Grov et al. 2014).

4.6.2 Requirements

Grov et al. (2014) categorized tunnels into three different types; (1) Mountain tunnel, where there are horizontal fractures since the vertical stresses are much higher than the horizontal ones due to a large overburden that also minimize the risk of grout overspread, (2) Forest subsea tunnels, where the stresses are relatively lower compared to the mountain tunnels (lower overburden) which could increase the risk of grout leakage and (3) urban tunnels, where the small amount of overburden increase the risk of uplift and large horizontal stresses increase the risk of grout spread to the surface. The target sealing requirement for these three types of tunnels according to Grov et al. (2014) is suggested in Table 4.2.

Table 4.2 Different types of tunnels with respect to sealing requirements according to Grov et al. (2014).

Mountain tunnel		**Subsea Tunnel**		**Urban Tunnel**	
Norway	Sweden[1]	Norway	Sweden	Norway[2]	Sweden[3]
30 l/min/100m of tunnel length	About 10 l/min/ 100m of tunnel length	10 – 30 l/min/100m of tunnel length	2-10 l/min/ 100m of tunnel length	2 – 10 l/min/100m of tunnel length	0.5-2 l/min/ 100m of tunnel length

1 This case is not common

2 Systematic pre-grouting is performed in case of inflow less than 15 liter/min/100m

3 Pre-grouting s performed in all urban tunnels

4.6.3 Grouting philosophy

The different philosophies of grouting, demonstrated by Grov et al. (2014), clarify the reason for the differences in the determination of the grouting parameters in the Norwegian and the Swedish projects. According to them, the Swedish practice is mainly based on a theoretical approach, which has been directly applied to urban projects accompanied with extensive experience from extremely tight grouting in radioactive waste storages. However, in the Norwegian projects, empirical methods are used and the strategy has been developed through an extensive practical experience from previous tunneling projects. Based on this, the Norwegian practice suggests a high pressure and a thin mix (lower yield stress and viscosity) with the aim of producing a thick zone around the tunnel. On the contrary, the Swedish practice focus on achieving a higher sealing by using a lower grouting pressure and a thicker mix (higher yield stress and viscosity).

In this report, a Swedish urban train tunnel (the Stockholm City Line project) and a Norwegian tunnel (Holme-Nykirke) are examined and compared against each other. There are both similarities and differences between these projects that make a comparison between them interesting and meaningful. First of all, both of the projects are situated in similar geological formations that generally consist of hard crystalline rock. From the statistics provided in the previous chapter, the geology of both projects is quite similar with a smallest hydraulic aperture size of 70µm, that needs to be sealed and not any apertures larger than 200µm. Furthermore, these tunnels are intended to be used for public transportation and thus the sealing requirement is of the same order in both of the projects. In addition, a systematic monitoring system is used in both projects and recorded data of pressure and volume of injected grout are available. However, due to the differences in grouting methods, different design parameters (material properties, grouting pressure and stop criteria) were used. Therefore, it would be interesting to compare the results from both of the projects with the theoretical approach described in chapter 3. In next chapters, each of the projects is examined and compared separately in order to examine the effect of different design parameters, especially grouting pressure, on the sealing efficiency.

5. THE STOCKHOLM CITY LINE PROJECT

The Stockholm City Line project is a 6 km long commuter train tunnel, which starts in the north at Tomteboda, passing the Odenplan station and the City station and continues to Södermalm in the south. This project started in 2006 and the tunnel is expected to be in operation in 2017. Trafikverket, as the employer and the owner of the project, divided the project into different sub projects.

5.1. Geology

The geological situation is mostly hard discontinuous rock. The amount of water flow in every second (conductivity) can show how fractured the rock mass is in each region (Table 5.1)

Table 5.1 Conductivity of the rock mass without grouting at the different sub projects in the Stockholm City Line project (after Brynjolfsson, 2014)

Sub project	Median Conductivity (m/s)	Highest Conductivity(m/s)
Vasatunneln	4.6×10^{-7}	1.6×10^{-6}
Odenplan	5×10^{-8}	1.6×10^{-6}
Norrmalm	1.3×10^{-7}	1×10^{-6}
Norrströmstunnel	5×10^{-7}	1×10^{-6}
Södermalm	3.5×10^{-8}	5×10^{-6}

5.2 Requirements of the project

The environmental requirements were set to decrease the leakage to around 25% lower than the in-leakage value presented in the documentation from the environmental court. In this respect the allowed in-leakage varies from 5 to 20 liter per minute per 100 meters. However, in some of the small sections it can be increased to 40 l/min/100m. The requirements of the different subsections are depicted in Table 5.2.

Table 5.2 Sealing requirement of grouting process in Stockholm City Line sub projects. (after Brynjolfsson, 2014)

Sub project	Flow without grouting l/min/100m	Allowed inflow l/min/100m	Required sealing efficiency % (Eq2.1)
Vasatunneln	40.8	3	93
Odenplan	10.5	3	71
Norrmalm	24	3	88
Norrströmstunnel	11	5	54
södermalm	11.1	4	64

Based on the categorization made by Stille (2012) regarding the difficulty of grouting, the degree of difficulty of the grouting work was expected to be uncomplicated or fair (See table 4.1).

5.3 Design Strategy

Certain strategies were taken into consideration in the design of the grouting work at the Stockholm City Line project. According to (Brynjolfsson, 2014) the main attention was given to meet the functional and the environmental demands, being robust, straightforward, flexible, cost and time effective and have the possibility of easy verification.

The stop criteria for the sub-projects Vasatunneln and Odenplan in the north were set to limit the injection volume, the grouting time and the grout flow. The volume was limited to avoid an overflow of grout, and was set to 500 liter in holes with a depth of 21 m, i.e. 24 liter/meter. This volume corresponds to the maximum required grout spread in the largest fracture. On the other hand, to confirm an adequate spread in the smallest fractures that should be grouted, it was proposed to stop grouting after 40 minutes, which is approximately the time it takes for the grout to spread to the minimum required distance in the smallest fractures. It should be noted that the sealing requirements determine the largest fracture that can remain unsealed. In this design, the stop criterion with a flow of 5 liter/5 minutes indicates that the rock mass is tight enough and that continuation of the grouting process is a waste of time.

The strategy is to perform grouting with (mostly) constant pressure at a maximum value of 2 MPa to inject the grout mix with a water cement ratio of 0.8. The process was stopped as soon as one of the limiting criteria was achieved. The mix could be thickened if the pressure did not reach 50% of the stop pressure (the 2 MPa) after injecting around half of the maximum volume (300 liters), since this indicates that a fracture larger than expected exists.

In Norrmalmstunneln, which is situated adjacent to the City Station, the grouting time limit was reduced to 30 minutes. Furthermore, the allowed grouting pressure was increased to 3 MPa where there was a rock cover larger than 20 m. Modifications made for grouting in the Norrström tunnel, based on trial grouting in the Bangård access tunnel, were also made in order to perform the grouting in a more economical way. The main variation in the design of this section was reducing the grouting time to 20 minutes. This new value was the outcome of an extensive case study with the application of the Real Time Grouting Control method (see Tsuji, et al., 2012). This study showed that in 85% of the holes, the grouting was expected to be completed in 20 minutes. In the next section, the trial test which led to this decrease in grouting time is discussed.

5.3.1 Trial test in the Bangård access tunnel

A trial test has been performed in the Bangård tunnel and the results were evaluated by the RTGC method in order to adjust the grouting design parameters (Tsuji, et al., 2012). The geology in this region is rated as good to very good rock with RMR values in the range of 70-100. The rock cover is approximately 15 to 30 meters. The number of boreholes in each fan varies from 22-27 depending on the permeability of the grouted zone. The length of these boreholes is 21 m. The maximum applicable pressure was set to 2 MPa, however, lower pressure was used in grouting of the roof holes. The position of the Bangård tunnel has been depicted in Figure 5.1.

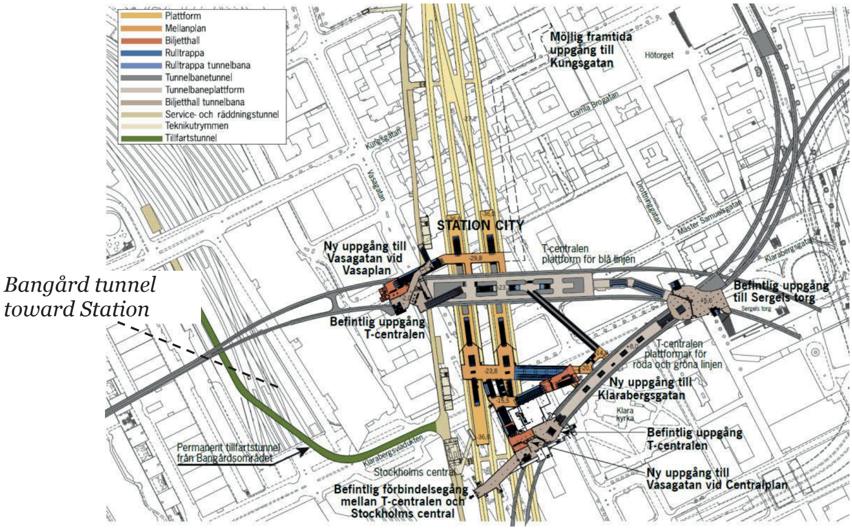

Figure 5.1 Layout of the Stockholm City station including the Bangård tunnel (after Tsuji et al. 2012)

In most of the fans, the first borehole to be grouted was the one in the centre of the tunnel bottom, since the conductivity of all holes were not known. However, for some fans the most conductive boreholes, based on hydraulic test, were grouted first and then the grouting proceeded in the regular order from the ones in the bottom and continued upwards, i.e. the last grouted boreholes were the centre hole in the roof.

In the study by Tsuji et al. (2012), grouting in 14 fans at different sections along this tunnel was evaluated. Calculations with the RTGC method showed that the grout could penetrate in fractures down to 70μm (the smallest fracture that should be sealed) at a distance of 4-8 meters from the borehole. The conductivity of the rock mass was estimated with both water pressure test, where the volume of the injected water under a certain pressure was measured, and with inflow measurement (Figure 5.2).

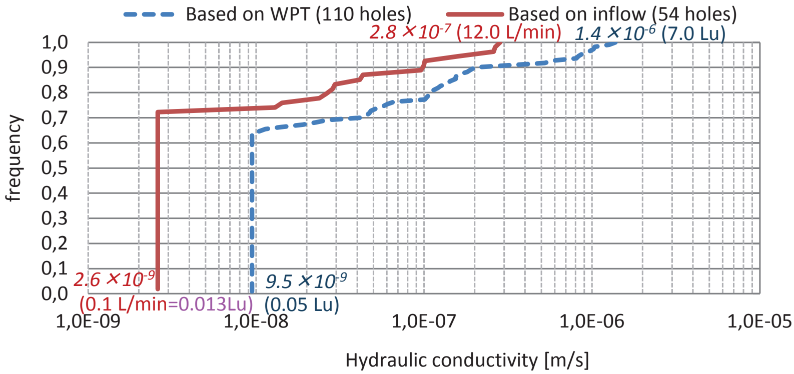

Figure 5.2 Cumulative function of transmissivity in each hole based on water pressure test results compared with that based on inflow measurements (after Tsuji, et al. 2012).

The aperture sizes in the 164 holes were estimated with the aid of both the cubic law (based on the estimated transmissivity) and the RTGC (based on the flow of grout). The smallest estimated aperture size was 41µm and few of the fractures were larger than 200µm. Tsuji et al. (2012) mentioned that the estimated apertures were 2-3 times larger than the results from water pressure test (Figure 5.3). In that study, the estimation of the distance of the grout spread by RTGC was performed for a minimum and a maximum limits of 70 (the smallest fracture to be sealed to achieve required sealing efficiency) and 200 µm respectively, since it was believed that not many fractures larger than that existed.

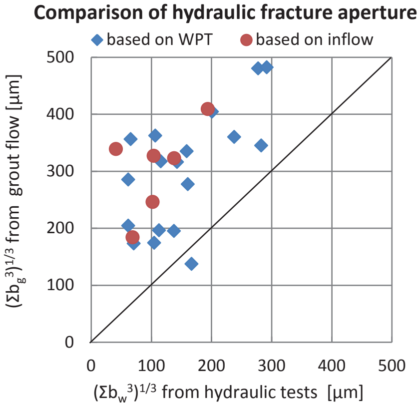

Figure 5.3 Comparison between the hydraulic fracture apertures based on hydraulic test $(\Sigma b_w^3)^{1/3}$ and that based on the grout flow $(\Sigma b_g^3)^{1/3}$ for 27 boreholes (after Tsuji, et al., 2012)

5.4 Application of the RTGC

Theoretical analysis is feasible in this project since a systematic monitoring and registration of data during the grouting were performed (data of pressure, grout flow and volume of injected grout were available in real time). These data make it possible to theoretically analyze and discuss the results with the RTGC methodology from the performed grouting. Data of grouting pressure, grout flow and injected volume combined with the material properties of the grout are the main inputs of the theory of the RTGC depicted in chapter 3. However, due to the lack of some of the data, specifically in-situ stresses, there could be some discrepancies between the results and the real process. In this project fractures are oriented close to vertical and this is favorable in case of using a higher pressure than the overburden since it provides higher resistance to the deformation due to higher in situ stresses.

The complete result of the RTGC analyses have been performed in previous studies by Tsuji et al. (2012). The grouting time to obtain the required penetration length of the grout has been implemented as one of the main stop criterion in the grouting process at the Bangård project. This criterion is based on the result of extensive analyses where the required spread was estimated theoretically to be achieved in less than 20 minutes. Tsuji et al. (2012) determined the requirement of a successful grouting to a minimum of 2 m grout spread with a b_{seal} of 70µm and a maximum of 5 m penetration with a b_{max} of 200µm (Figure 5.4), by considering the required grouting in the largest fractures and the fact that smaller fractures have smaller persistence and may not be connected to other fractures, i.e. do not have any major effect on the sealing efficiency. The results from analysing 326 holes shows that for approximately 85% of the cases, the spread criteria is fulfilled in 20 minutes of grouting, excluding hole filling time (Figure 5.5).

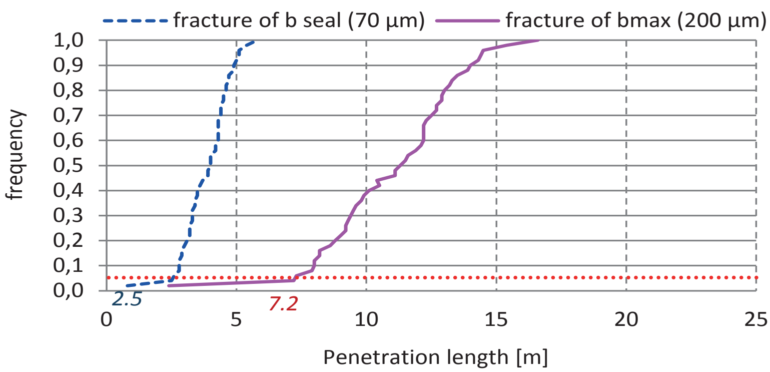

Figure 5.4 Cumulative function of grout spread for 50 holes with 2D grout flow at the Bangård tunnel in the Stockholm City line project (After Tsuji et al. 2012). The distance of grout spread has been estimated in smallest fracture to be sealed (70µm) and largest existing fracture (200µm).

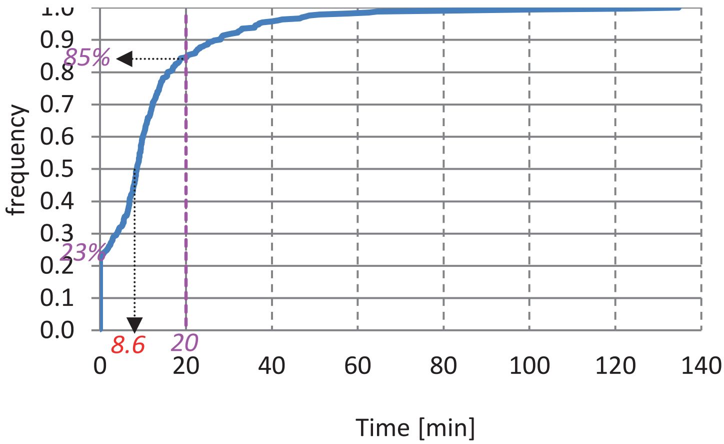

Figure 5.5 Cumulative function of actual grouting time for all 326 holes at the Bangård tunnel in the Stockholm City Line project (After Tsuji et al. 2012).

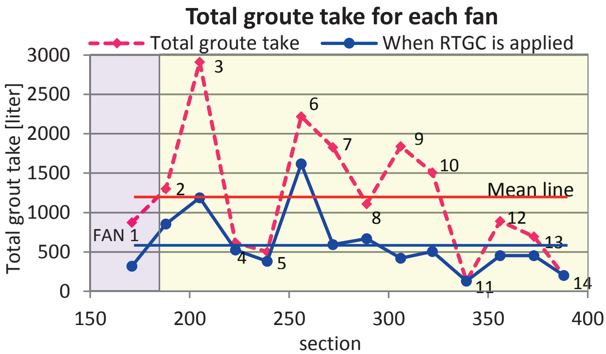

Figure 5.6 Comparison of the measured injected volume and the estimated grout volume based on the estimated penetration length. The aperture for each hole has been estimated based on recorded flow of grout (RTGC)

The total injected volume in some of the fans is larger than the estimated volume needed to fill the estimated fractures, showed in Figure. 5.3. This has been depicted in Figure. 5.6.

The grouting process with the time as stop criterion at the Bangård tunnel was considered successful, since the transmissivity of the rock mass decreased to the desired amount and in average, a sealing efficiency of 90% was obtained (see Figure 5.7).

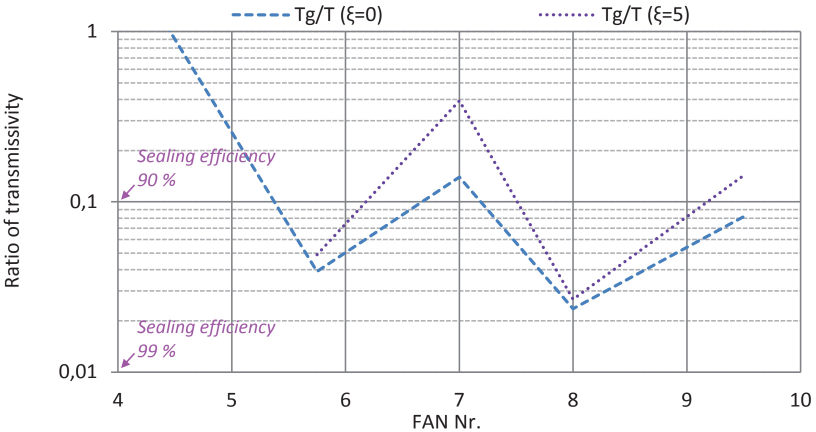

Figure 5.7 Reduction of transmissivity by grouting at the Bangård access tunnel in the Stockholm City Line project (After Tsuji et al. 2012).

5.5 Jacking of fractures

As previously discussed, large penetration in combination with relatively high-applied pressure may lead to jacking and fracture deformations. In order to analyze if jacking has occurred, the registered pressure and flow data during grouting needs to be analyzed. Almost no jacking could be distinguished in grouting performed in the Bangård access tunnel in the Stockholm City Line project. The main reason for this is the relatively low pressure used. One of the possible cases of jacking have been discussed by Rafi and Stille (2015), in which the deviation of recorded and estimated flows (onset of jacking) occurs after 4 minutes of grouting. The tunnel at this section is situated at a relatively low depth (around 20 m) and thus the estimated data indicates the occurrence of elastic jacking. However, according to Rafi and Stille (2015), this deformation is “manageable” since the grouting time was short enough, which limited the spread of the grout to 17 m. The probable deformation caused by the force due to the combination of this penetration length together with the relatively low applied pressure of 15 bar may increase the grouting time by less than 5% in order to achieve a grout spread of 17 meters. Furthermore, it may increase the transmissivity 1.6 times due to the extent of deformations in the un-grouted zone. It should be mentioned that this deformation might be much smaller than the estimated one, since the fractures in this region mainly consists of vertical ones. Due to the high requirements on tightness in this project, even those small deformations should have been unacceptable. However, it seems the small probable voids caused by the increase of fracture aperture were filled up with grouting from nearby holes, since the measured inflow in this zone is even smaller than the defined inflow requirements of the project.

6 THE HOLM-NYKIRKE PROJECT

The Holm-Nykirke project is a 14.3 km double track railway, where 12.3 km of that runs underground in tunnels. The railway stretch out along Holmstrand in Norway, see Figure 6.1. The project started in 2010 with a budget of 5.7 billion Norwegian Krona, and it is expected to be in operation by the end of 2016. The new double track between Holm and Nykirke will provide benefits in terms of reduced travel time, increased travel capacity and improved punctuality for the regional traffic. In addition, there will be lower maintenance requirements related to rock cuts and poor soil conditions.

Figure 6.1 Plan of Holm-Nykrike Project

6.1 Geology

In the study performed by (Nikolaev, 2015) the grouting performed in a section of around 4 km of the tunnel, between chainage 80185 to 84200, were examined. The rock cover over the tunnel in this area is between 50-170 m and the geology consists of basaltic solidified lava streams of rock with intermediate layers of sedimentary rock, such as silt, sandstone, tuff, agglomerate and lava conglomerate. This section was divided into three subsections with different geological characteristics, which have been summarized in Table 6.1. In this table, the highest grouting pressured used, the rock cover, and the number of grout holes are also presented (see also Figure 6.2).

Table 6.1 Summary of the characteristics for the analysed regions in the Holm-Nykirke project (Nikolaev 2015).

	Region 1 (fans 80472 and 80483)	Region 2 (fans 80994 and 81010)	Region 3 (fans 83429 and 83446)
Q-value	8	20	50
Geology	Sandstone	Basaltic igneous rock	Basaltic igneous rock
Highest grouting pressure [bar]	88	83	82
Approximate depth beneath the ground surface [m]	130	160	70
Total number of grout holes	106	94	104

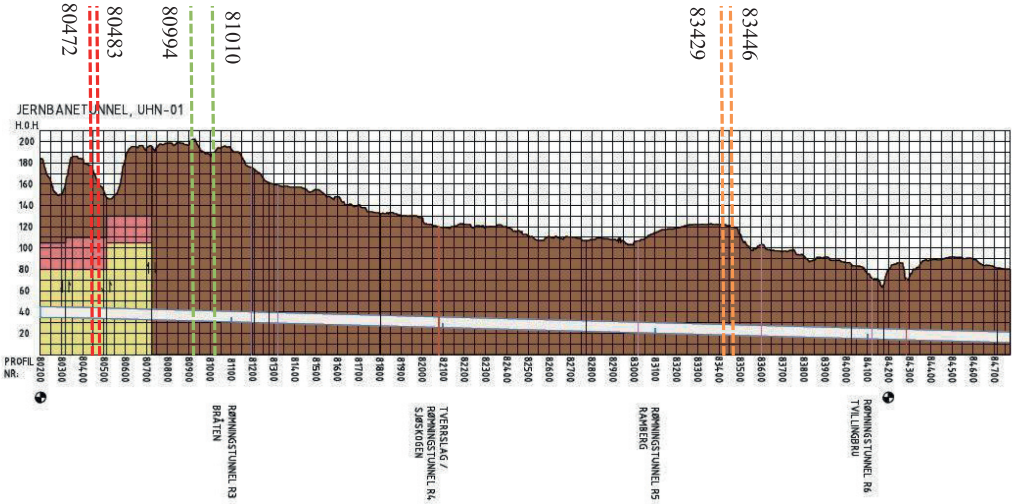

Figure 6.2 Illustration of the analysed sections of the tunnel (After Nikolaev, 2015).

6.2 Requirements of the project

The requirement of this urban traffic tunnel under the sea is not specifically mentioned in documents from the employer or contractor. However, it can be expected to be in the range of similar projects as shown in Table 2.2. Due to the similarity with the Stockholm City Line project (with respect to function and in certain aspects also the geology), we may expect that filling fractures with apertures larger than 70µm to be required in order to fulfil the requirements (see section 5.2). If the distribution of aperture sizes along the whole area is considered, around 20% of the fractures are smaller than 70 µm (Nikolaev, 2015). This means that 80 percent of the fractures should be sealed. Furthermore, the maximum size of the fractures is small and around 95% of all of the fractures are smaller than 125 µm (Figure. 6.3).

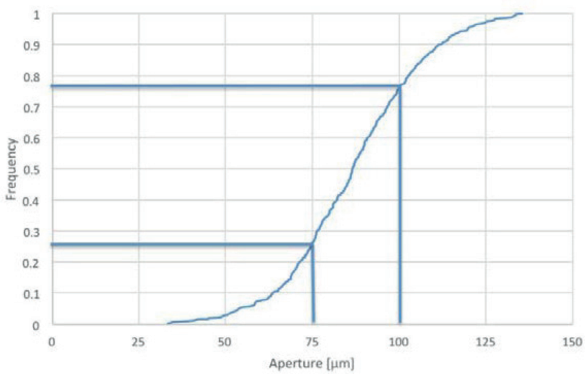

Figure 6.3 Cumulative function of the estimated aperture size by considering the flow of the grout inside the fracture using the RTGC (After Nikolaev, 2015). This Figure covers all of the fractures in the 3 regions depicted in Table 6.1.

6.3 Design strategy

In the Holm-Nykirke project, a relatively thin mix (yields value of 0.36 Pa) in combination with a relatively high pressure (8-9 MPa) has been used. From the registered flow and pressure data, the stop criteria seems to be achieving a certain maximum pressure at a certain minimum flow, and to obtain that, the grout mix has been allowed to be thickened by reducing the water cement ratio.

In order to evaluate performed grouting work, the injected volume of grout was compared with the volume corresponding to a 10 m radial grout spread around the tunnel. With the estimated apertures, the results show that a clear over spread of grout is obtained, see Figure 6.4. The analysis by Nikolaev (2015) shows that 200 to 1600 liter of grout could be enough to obtain a 10 m radial spread of grout from the tunnel wall in a horizontal fracture, depending on the size of the aperture and the pressure. However, in some cases, a much larger amount of grout mix was consumed. The injected grout volume in more than half of the grout-able boreholes was less than 500 liter but in boreholes situated in zones with poor rock mass quality, more than 3500 liter of grout was injected in a single hole (Figure 6.5).

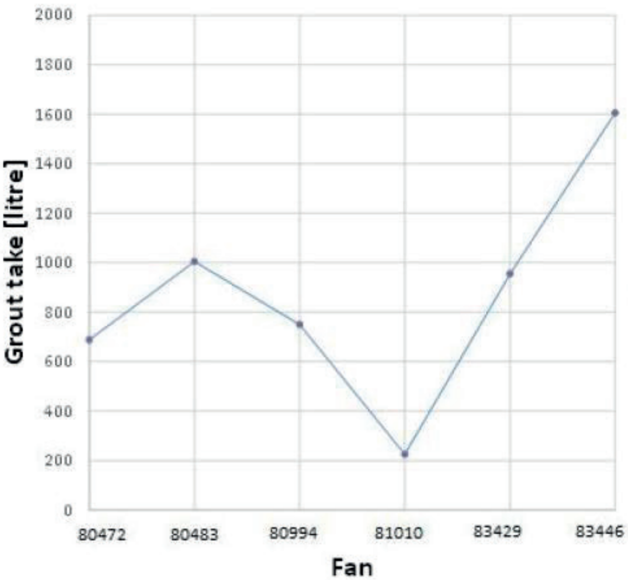

Figure 6.4 The estimated injected volume corresponding to 10 meter of radial grout spread in different fans (after Nikolaev, 2015)

The large injected volume can be the result from a long grouting time. From Figure 6.6, it can be seen that only around 10% of the holes are grouted in less than 20 minutes, which indicates a relatively longer grouting time in most of grout holes compared with the Bangård tunnel. However, as shown in Figure 6.6, the grouting time for around 80% of the boreholes is less than two hours.

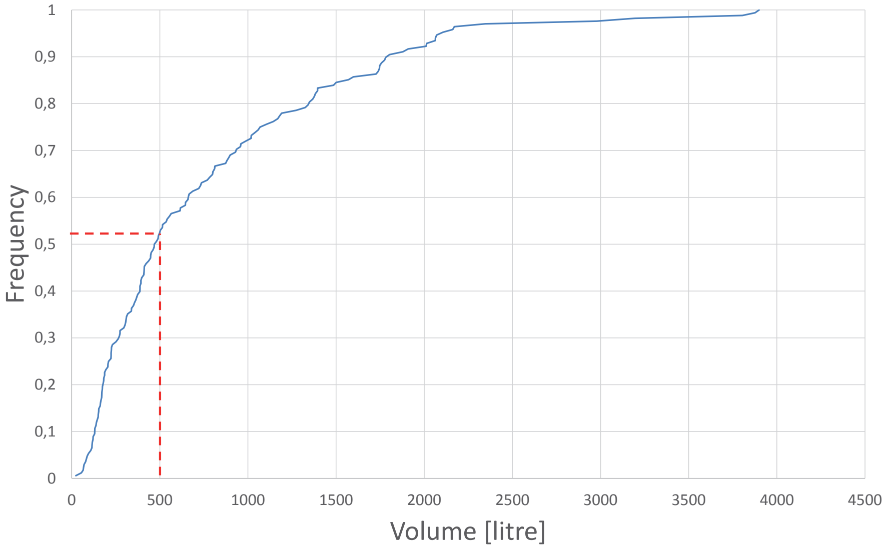

Figure 6.5 Cumulative measured injected grout volume in different boreholes (after Nikolaev, 2015)

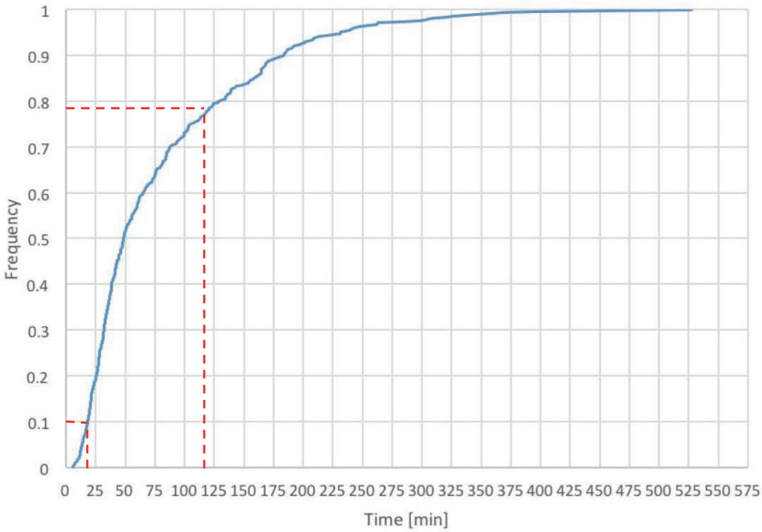

Figure 6.6 Cumulative recorded grouting time in each hole in the Holm-Nykirke project (After Nikolaev, 2015). Around 10% of the grouting work was finalized in less than 20 minutes and around 80% of the holes have been grouted in less than 2 hours.

6.4 Application of the RTGC

Through the analyses of different cases in the Holm-Nykirke project, Nikolaev (2015) showed that the recorded flow and the estimated flow with the Real Time Grouting Control method in many of the studied cases are in acceptable agreement (deviation is less than 5%). With this knowledge, the theoretical approach was assumed to be applicable in this project and the grouting process might be possible to be analyzed with this methodology.

Through the examined data, the shortest distance of radial grout spread from the borehole is estimated to be 20 m and in 50% of studied cases, the spread distance exceeds 50 m in this project, with a maximum spread of 120 m, see Figure 6.7. The spectrum of the apertures in the sandstone zone is depicted in Figure 6.8. When these values are analyzed, it should be noted that the theoretically estimated values with the RTGC methodology for the spread of the grout in this section are based on an assumed horizontal infinite fracture.

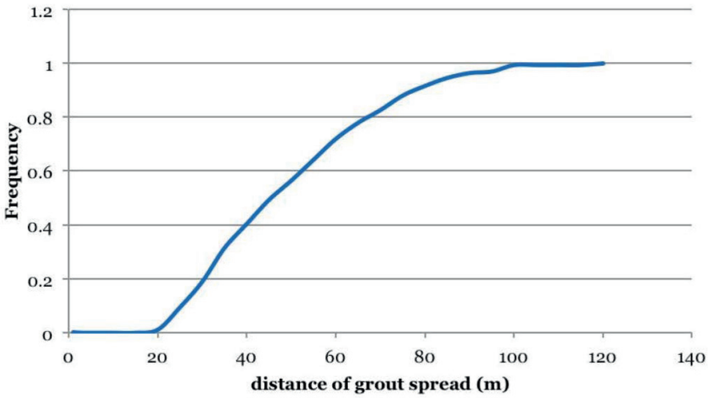

Figure 6.7 Cumulative function of the distance of the grout spread in the sandstone zone

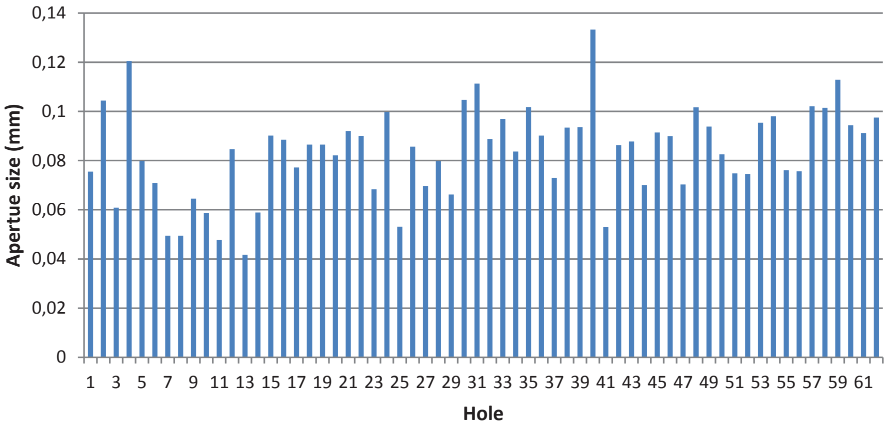

Figure 6.8 The size of the hydraulic apertures in the sandstone zone

The other interesting result is the effect of the grouting time and the size of the aperture on the spread of the grout. Figure 6.9 shows that in the studied cases, applying an almost similar pressure implies that the variation in the spread of the grout depends mainly on the estimated aperture size and the grouting time. This indicates the importance of limiting the grouting time to the required spread in the smallest and the largest fractures.

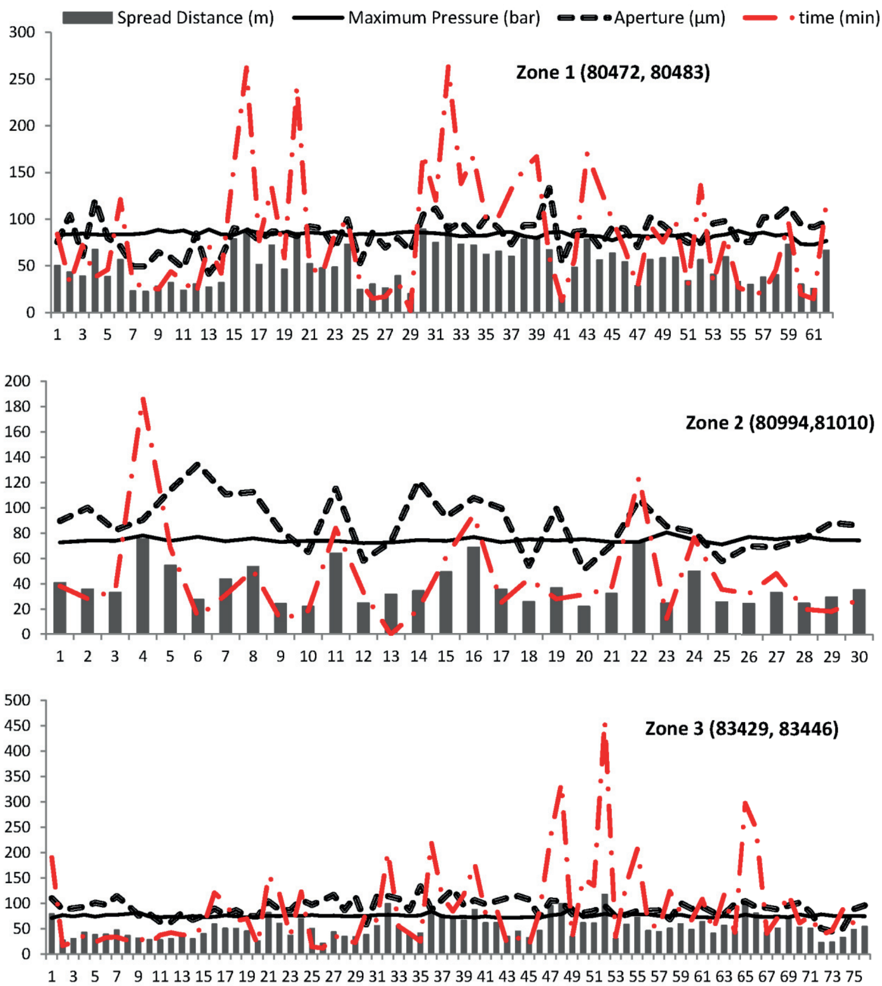

Figure 6.9 The effect of pressure, time and aperture size on the distance of the grout spread (5 fans) in the Holm-Nykirke project.

6.5 Jacking of fractures

Nikolaev (2015) performed an extensive case study of the Holm-Nykirke project in which probable deformations as well as the penetration length of the grout, with and without the deformations due to jacking, were estimated. That study used the output of RTGC, i.e. estimated distance of grout spread and estimated grout flow, through a theoretical approach. With this methodology, clear signs of jacking were observed in 15 out of 62 holes in a section of the tunnel, with a rock cover of 130 meter located in sandstone. In these cases, there was a clear deviation between recorded and estimated grout flow. The aperture of the fractures in this zone where jacking occurred is estimated to open up 40 times the initial aperture size, see Figure 6.10. However, the state of the fracture is far below the ultimate state, where a unrecoverable deformation of the ground may occur, i.e. the deformations are elastic and can be recovered if grout can be pushed out.

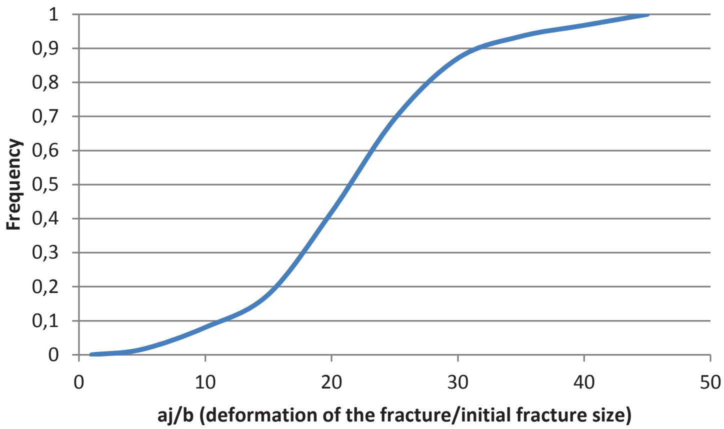

Figure 6.10 Cumulative function of the estimated increase in fracture aperture size in the zone with sand stone (62 holes). "aj" is the amount of deformation. "b" is the initial fracture aperture size (as depicted in Figure 6.8.)

In the analysis by Nikolaev (2015) it was shown that the used pressure in the Holm-Nykirke project is high and is quite similar in different holes. Thus, the variation of the calculated deformation is directly correlated to the variation in the distance of the grout spread. The increasing trend of the estimated deformation due to the estimated grout spread in the holes situated in the sandstone section is shown in Figure 6.11.

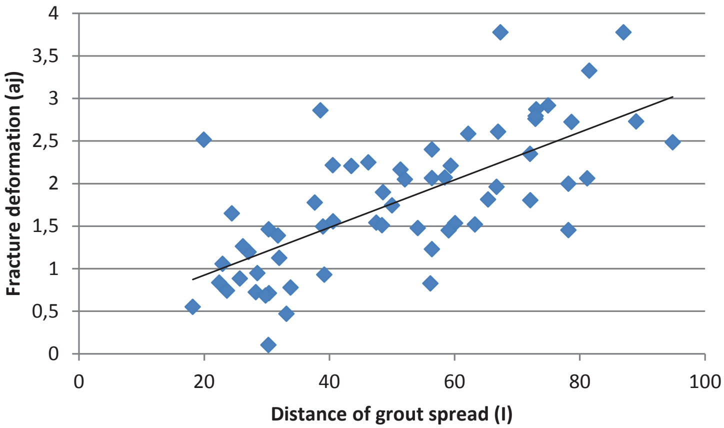

Figure 6.11 Correlation between the distances of grout spread with fracture deformation. Deformation is in millimetre and distance of grout spread is in meter.

The distance of the spread of grout in deformed fracture is smaller than the estimated one in initial fracture, see Figure 6.12, which means longer grouting time is needed if a specified penetration should be obtained. In other word, longer grouting time is needed to achieve the same distance of grout spread in jacked fractures. Thus, the long grouting time might be acceptable, since a spread of 17 meters in average are expected, see Figure 6.13, if such large deformations as depicted in Figure 6.10 occur. This means that the optimum grouting time should be determined considering the applied pressure.

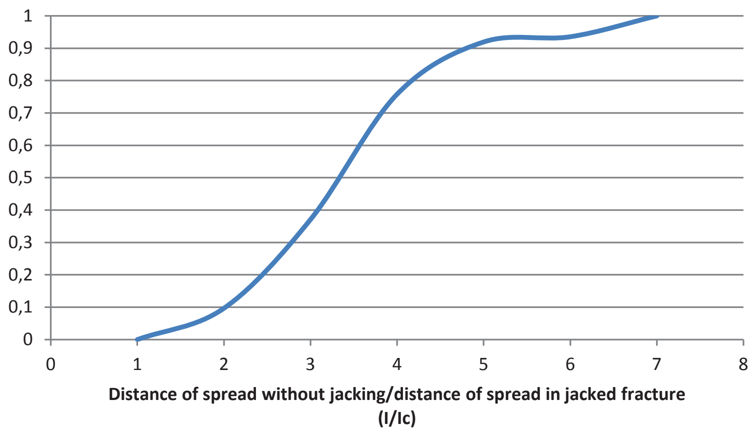

Figure 6.12 Cumulative function of the ratio of distance of grout spread in initial fracture of "b" (I) to the deformed fracture of "aj+b" (Ic) in sand stone zone.

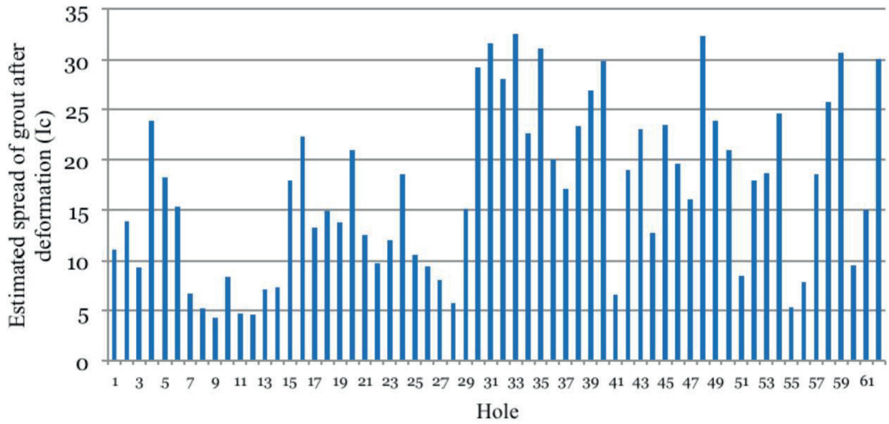

Figure 6.13 Distance of grout spread in deformed fracture of "aj+b" (Ic) in the zone with sandstone.

The other negative aspect of fracture deformation is the increase of transmissivity due to opening of previously grouted fractures. The significant increase in transmissivity occurs since it is a function of the fracture aperture size to the power of three (the cubic law).

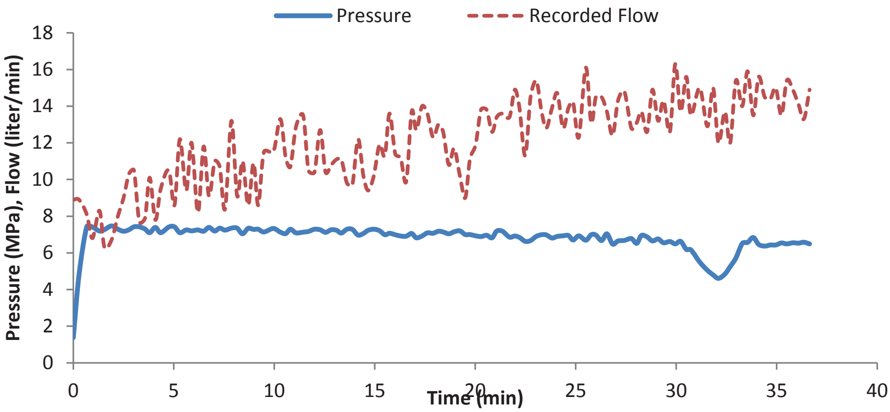

Figure 6.14 Registered pressure and flow in hole 10 fan 80472.

In the analyses of hole 10 for fan 80472, the focus is on the first 35 minutes of grouting where in average, a pressure of 70 bar has been applied (Figure. 6.14). A grout mix with a yield stress of 1 Pa and a viscosity of 0.02 Pa·s has been used. The aperture size is estimated to be 80 µm considering the recorded flow (Eq. 4.5). This is the average aperture size during the whole period of grouting. However, if jacking occurs the estimated average size will be larger than the initial fracture aperture size, and the estimated flow will become larger than the recorded flow as long as the aperture size is smaller than the estimated average aperture size (this occurs during first 10 minutes as shown in Figure. 6.15). However, due to the high applied pressure of 70 bar in this hole, the expectation is occurrence of jacking of the fracture from the onset of grouting. Thus it would be correct to only consider an initial recorded flow value to estimate the initial aperture size, which will be 62μm. This will result in a curve with decreasing trend that deviates from the recorded flow at the initial moment (Figure 6.16). As grouting continues, the fracture deformation will increase, and as a consequence a larger amount of grout can flow into this larger aperture.

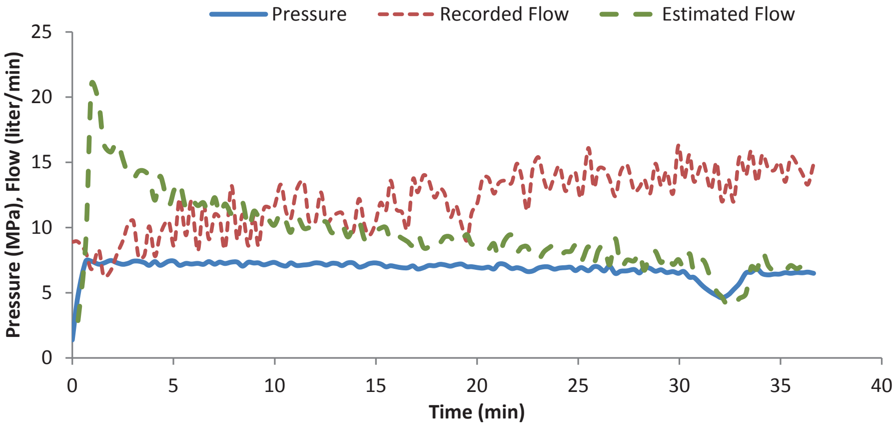

Figure 6.15 Estimated flow of grout in the fracture with average estimated aperture size of 80μm during grouting.

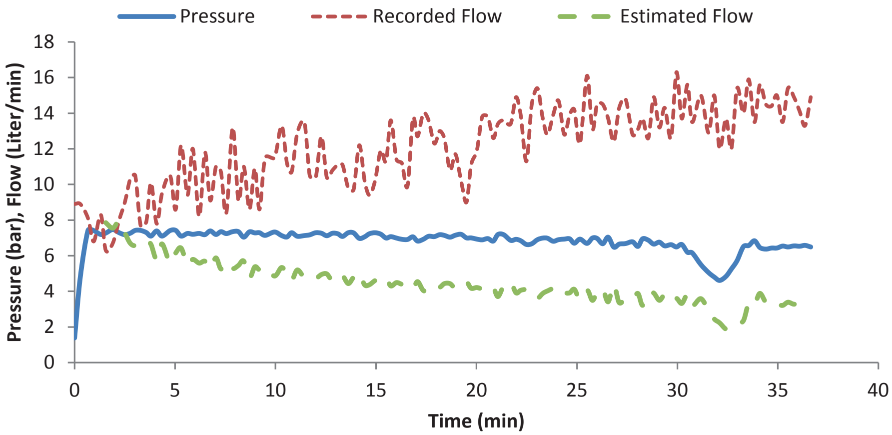

Figure 6.16 Estimated flow of grouting considering the initial estimated aperture size of 62μm.

The probability of jacking can also be closely examined by establishing an elastic jacking limit to confirm this deformation as well as determining its nature and amount. Figure 6.17 shows that jacking of the fracture probably started from the onset of grouting and the P_n data are 4 times larger than what is suggested as an elastic jacking limit based on the overburden above a horizontal fracture, which is P_n=0.3. The recorded data are lower than the ultimate limit, where unrecoverable deformations may occur. the estimated distance of the grout spread is 30 meters. The fracture aperture might open up to 2 mm according to the applied theory, which results in a decrease of the distance of grout spread with a factor of approximately 2.7 of the estimated distance with the initial aperture (Figure 6.18). Comparing to Figure 6.12, this sample is among 20% of the smallest variation in estimated spread after jacking. In 80 % of the cases, the estimated distance of grout spread is much smaller that the estimated spread in initial aperture.
Besides the effect on distance of grout spread, this deformation might open up the grouted fractures in the vicinity. This could affect the sealing efficiency of the grouting process if these induced voids are not filled during grouting from other holes and remain unsealed.

This example shows also the importance of geological investigations to determine the correct aperture size of the representative fractures during theoretical analyses. Furthermore, it should be noticed that the assumed fracture in this theoretical discussion is horizontal with an assumed infinite persistence. However, if the media is highly fractured, this simplification might affect the results, since the estimated distance would not be horizontal anymore. A consistent design and providing scenarios for variation of the design parameters at certain events would bring a systematic grouting process that can improve the results. In the discussed example, there is no information about the reason for applying a variation in pressure and why the grouting process was stopped.

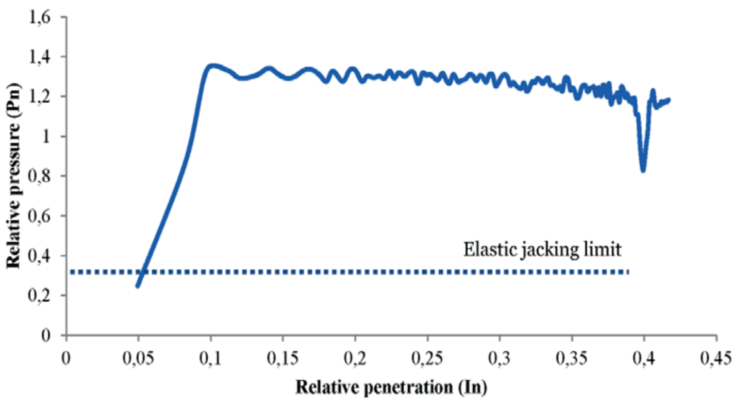

Figure 6.17 Pn-In compared with the elastic and ultimate jacking limits in hole 10 of fan 80472. The grout spread up to 30 meters from the boreholes

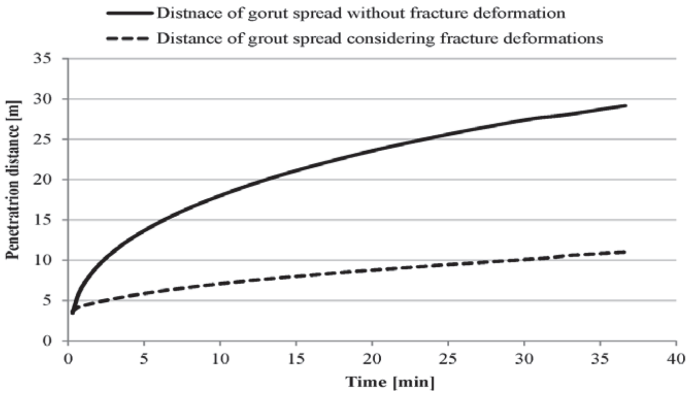

Figure 6.18 Comparison of the distance of grout spread with an initial aperture of 62µm (30m) and the deformed fracture that opens up to 2 mm, (11 m). This means that the ratio of I/Ic is around 2.7.

Another issue that might occur during the grouting is that in some of the holes, only a limited amount of grout is possible to be injected. Recorded data from some cases in the Holm-Nykirke project show that despite applying a high pressure, only a small amount of grout has been injected. For instance, in the hole beside the one examined in the previous example, pressure has been increased up to 80 bars and grouting is continued for around 30 minutes, however, the amount of flow barely exceeds 5 liter/min (Figure 6.19). The initial physical aperture size is estimated to be 45µm based on the injected grout volume (Eq. 3.4). Grout mix with yield stress of 1 Pa and viscosity of 0.02 Pa·s is estimated to spread around the borehole at a distance of 25 m in this fracture during 30 minutes. However, according to Eklund and Stille (2004), the aperture size should be at least 3 times the cement grain size otherwise the grout mix cannot penetrate into that fracture. This means that this fracture can be categorized as a tight fracture and the grout most probably could not penetrate into this fracture or has limited grout take. Furthermore, the high-applied pressure was not enough to open the fracture to an extent to have a larger volume injected. In addition, the size of the fracture is smaller than the smallest fracture needed to be grouted to fulfill sealing requirement of this project. With this information, the possible benefit with fracture deformation is unknown.

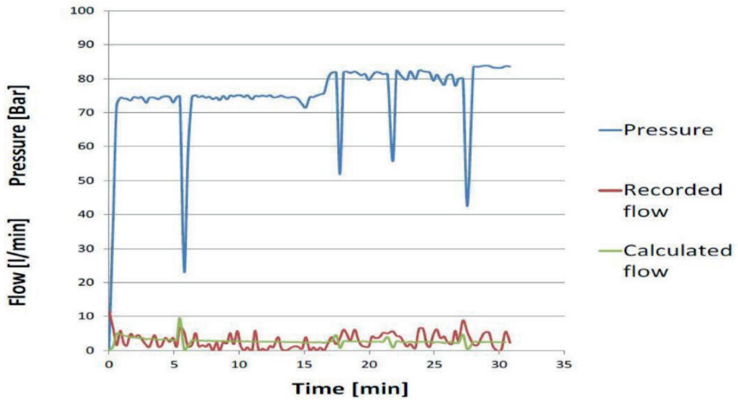

Figure 6.19 Example of a hole with limited grout take in fan 80472 (hole number 10) in the Holm-Nykirke project (After Nikolaev, 2015)

When the holes close to the studied example (hole number 10) are examined, it seems that the rock mass at this zone is thigh and it might be beneficial to investigate the geological situation in order to decrease the number of grout holes in this region for the next fans. Statistical data confirms that there have been a large number of holes that had almost no grout take (Table 6.2). Defining a logical sequence of grouting can provide a valuable information about the media and would be beneficial in revising the number of boreholes.

Table 6.2 Number of holes with no take compared with all the holes in the analyzed regions (After Nikolaev, 2015)

Holm project	Region 1 (fans 80472 & 80483)	Region 2 (fans 80994 & 81010)	Region 3 (fans 83429 & 83446)
Total number of grout holes	106	94	104
Number of grout holes with no take	44	64	28

From the examples discussed above, it can be concluded that investigated geological data, laboratory test results that shows the properties of the grout mix, and records of the variations in pressure and flow during the grouting process are essential information required for analyzing the cases theoretically.

7. DISCUSSION OF THE RESULTS

7.1 City Line project vs Holm-Nykirke project

The application of the RTGC method in the Stockholm City Line project and the Holm-Nykirke project is shown to be applicable in the previous chapters. The main difference of the grouting process between the two projects was determination of the design parameters (Pressure, mix properties and stop criteria). In comparison, it should be noted that the design parameters in the Stockholm City Line project and the Holm-Nykirke project are different. The grout mix prepared with INJ 30 cement in the Stockholm City Line project has higher yield stress than the mix used in the Norwegian project. With respect to the spread of the grout, under the same pressure, it may take shorter time for the mix with lower yield stress to reach the same penetration, which is the case in Holm-Nykirke project. In the Holm-Nykirke project, in contrast with the Bangård access tunnel in the Stockholm City Line project, grouting time was not limited which may lead to a relatively longer grout spread. Combination of a higher pressure and a thinner grout mix in this project make the condition even worse. As shown previously, it has on the average taken one hour to grout each hole and it takes even longer time for around half of the boreholes, which indicates possible overspread of grout at the Holm-Nykirke project. This long spread distance, i.e. large injected volume, in combination with the relatively higher pressure, is the main factors causing the more frequently observed jacking in the Holm-Nykirke project compared to the Stockholm City Line. It should also be noted that in the Holm-Nykirke project, fractures are not oriented close to vertical (as in the Stockholm City Line project).

7.2 Designing grouting in similar projects

The grouting works at both discussed projects were performed based on empirical approaches and the design parameters were determined by expert judgment and benchmarking similar project. However, it is possible to improve the design with more control on the outcome of the project by applying the RTGC method. This was implemented in the Bangård access tunnel (Tsuji et al. 2012). In order to do that, there is a need for adjusting the initial set ups accordingly to be able to interpret the results and taking correct action. Among the initial set ups, the determination of pumping pressure and stop criteria are points of interest. Discussion about which properties of the grout mix that is most suitable to use is, however, beyond the scope of this report.

Prior to grouting, thorough geological investigations in order to examine the in-situ stresses and the size of the rock fractures would be useful in obtaining more precise results. However, the economy and requirements of the project should be considered before performing such investigations, i.e. they may not always be necessary.

In the next step, acceptable elastic deformations and required spread of the grout should be determined. With this information and by establishing the corresponding elastic jacking curve, the applicable pressure should be decided. With this pressure, the required time to achieve the required spread can be calculated, including the difference in the distance of the grout spread in the deformed fracture. This grouting time should be used as one of the stop criterion in order to avoid an overspread of grout. Besides, a minimum grout flow should be defined to avoid a waste of grouting time in fractures with apertures close to the minimum aperture.

8. SUMMARY AND CONCLUSIONS

In this report, two Scandinavian tunneling projects were compared in terms of the applied grouting pressure. One of the main issues discussed was the importance of the geological investigations when the design parameters of the grouting should be determined. It could be observed that by interpreting the results in combination with the recorded pressure and flow data, the accuracy of the initial geological assumptions could be validated. This is especially useful for validating the simplification of the fractured rock mass to a rock with a large representative fracture. This validation is done by examining the agreement of the recorded grout flow through the rock mass with the estimated grout flow through that representative fracture.

In addition, it was shown that the estimated values are valuable if they are examined against the recorded and observed data and events. In offline analyses, where the RTGC is used to evaluate the previous recorded data, it was shown that RTGC application provides a tool that gives essential information about the spread of the grout and the state of deformation of the fracture. However, the estimated values must be analyzed and discussed against recorded data and registered observations, since these values at the present time are not available during grouting to make direct on-site changes to design parameters based on them.

Online application, that is expected to be developed on grouting rigs in a near future in order to control the grouting during the process in real time can overcome these difficulties since the analyses are performed at every time step and design parameters can be adjusted in real time accordingly. The case study by Nikolaev (2015) showed that the extended application for the calculation of the consequences of elastic jacking works well, and it can be implemented in an online application.

In the comparison between the two projects it was shown that a combination of high pressure and long grouting time in the Norwegian project could lead to unnecessarily long spread of grout and injection of a large volume of grout. Furthermore, a combination of this long distance of grout spread and high grout pressure could also lead to jacking and large deformations of the fractures. It seems that possible hidden consequences of these deformations can have a significant impact on the sealing requirements as well as on the economy of the project. However, fulfillment of sealing requirements was reported in both projects, despite the large estimated fracture deformations in the Norwegian project. One reason might be that the induced voids due to jacking are filled by grouting of fractures in nearby located holes. The analyses also indicate that the relatively long grouting time was not necessary in some of the studied holes of Norwegian project.

Based on this work, it can be concluded that the theoretical approach can provide essential information by quantifying consequences of elastic jacking that can be used for an optimizing of the injection pressure and stop criteria in the design phase.

For future work, it would be interesting to establish different grouting scenarios with respect to design parameters. By doing so, initially estimated design parameters can be justified base on these scenarios during grouting. This would be an important step towards establishing an online application of this theory.

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