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Thesis for the Degree of
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Trondheim, October 2019

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## Abstract

Due to growth of the population in cities and limited space there is an increasing trend to move transportation systems, such as roads and railroads underground. Furthermore, Norway is a rugged country with deep fjords, high mountains and numerous islands, where tunnelling is an essential measure to ensure effective transportation connecting the different parts of the country.

If no measures are performed to reduce water inflow into tunnels, they will work as very efficient drains of the groundwater. Lowering of the groundwater table has many adverse consequences, such as damage on infrastructure and buildings due to ground surface settlement, drainage of lakes and desiccation of vegetation. High ingress of water into a tunnel can also be unfavourable for the work during construction and water seepage in tunnels under operation could lead to corrosion of installations, slippery roads or formation of icicles during the winter. For these reasons, tunnels have different requirements regarding allowable water inflow, depending on the use and location.

The most common method for reducing groundwater ingress into tunnels in Norwegian tunnelling, is cement grouting ahead of the tunnel work face, termed pre-grouting (short for pre-excavation grouting). In Norwegian rock mass pre-grouting practice the grouting pressure is significantly higher than in other countries worldwide. The use of high pressure originates from decades of practical experience in grouting of rock caverns, subsea tunnels and hydropower projects. Furthermore, the grouting pressure was slightly increased and standardised by the research project “Tunnels for the citizens” (finished in 2004), but there is little research to justify the usage of high pressure. Despite a wide empirical basis there are unresolved questions related to the use of high grouting pressure.

True Improvement in Grouting High pressure Technology for tunnelling (TIGHT) is a competence building project (KPN) in user defined innovation arena (BIA), carried out in the period of 2014 to 2018. This PhD is written in connection with TIGHT. The main goal of the project was to increase the understanding of rock mass grouting to such extent that it would lead to a significant differentiation on use of materials and pressure pending on the actual circumstances for any given project.

The fundamental objective for this PhD is to achieve an increased understanding of rock mass grouting with the use of high grouting pressures and to investigate if the current practice of using high grouting pressure is optimal regarding reduction of water inflow in tunnels, in line with good economy and caretaking environmental aspects. The thesis is comprised of two

different methods to investigate pre-grouting: analysis of data from the grouting process and investigation and assessment of pre-grouted rock mass.

The principal conclusions based on this PhD study are that pre-grouting with the use of high grouting pressure in general is successful regarding the reduction of water ingress into tunnels. When it comes to economy and environmental aspects there is potential for improvement. The study showed a high prevalence of hydraulic jacking of fractures during pre-grouting and the grout and time consumption in holes where hydraulic jacking was indicated in general were considerably increased. It is concluded that by adjusting the grouting strategy after the occurrence of hydraulic jacking, the grout consumption could be decreased without inflicting negatively on the reduction of water ingress into the tunnel.

The thesis highlights the importance of understanding how cement based grout spreads in rock mass fractures with different grouting strategies, types of grout and grouting pressure. Furthermore, it discusses how to perform successful pre-grouting with respect to the reduction of water inflow into tunnels, economy and environmental aspects.

## Preface

This PhD study was conducted at the Department of Geoscience and Petroleum at the Norwegian University of Science and Technology (NTNU) from September 2015 to May 2019. The thesis is presented as a collection of papers. The research was funded by the Research Council of Norway and linked to the research project True Improvement in Grouting High pressure Technology for tunnelling (TIGHT), project no. 236676/O30, a competence building project (KPN) in user defined innovation arena (BIA), in cooperation with Statens Vegvesen, Bane NOR, and industrial partners BASF, Mapei, Geovita, LNS, ITS, Normet, Bever Control, AMV and Veidekke. Research partners in TIGHT are NGI, NTNU and SINTEF, whilst KIGAM of Korea and Nanyang University in Singapore together with BeFo of Sweden are associated with TIGHT. Additional contributions were made by Bane NOR; availability of the construction site at Åsland and funding of the tests conducted at this site.

## Acknowledgements

At first, I would like to express my gratitude to my supervisor Eivind Grøv, for his never ending optimism and availability. Throughout this PhD he has shared his expertise and connections with the tunnelling industry in both Norway and Sweden. I am also grateful to my cosupervisor, Professor Bjørn Nilsen, for constructive feedback and proofreading. Thanks to SINTEF for giving me a place at their office during the PhD work. I am sure I would not have been able to finish my PhD on time if it has not been for the support from Stein, Nghia, Javier, Dirk, Mario, Torun, Yared, Anatoly and Ivan at the office, with special thanks to Kristin Hilde Holmøy for professional assistance throughout the PhD. I also appreciate the assistance from my cosupervisor Ulf Håkansson for helping me with the understanding of grout properties.

Bjørnar Gammelsæter from Bane NOR has been the key and vital assistance for making the testing at Åsland possible, always answering my e-mails and helping me with information and sharing his expertise.

I would like to thank my colleagues in the TIGHT project, with special thanks to Thorvald Wetlesen Sr., Claudia Querbach and Silje Hatlø Hagen at Bever Control AS, for collaborating with me and expressing their engagement in my work. Also, thanks to Lloyd Tunbridge (NGI) for including me in his work for the TIGHT project.

I greatly appreciate the guidance from Karl Gunnar Holter in regard to the execution of core drilling and water injection tests at Åsland.

I would also express my appreciation to the PhD colleagues at NTNU for unwinding activities, such as IGP retreat, Tuesday quizzes and lunches. I would like to give a special tank to Anette for being my private colour consultant for graphics and dinner dates to get the PhD work at arm's length. I would also like to thank Veena for consulting me in the field of statistics. Towards the end of the PhD I received irreplaceable help from Solveig, letting me take part of her PhD dissemination and guidance towards the PhD defence.

Finally, I would express my gratitude to my amazing family. Emil who is patient, supporting and makes every day more likable. Energic little Oskar who was a welcoming contribution to the family during the PhD. Thanks to my parents for excellent dinners and weekend trips, Randi and Åge for welcoming family dinners. I have a great appreciation to the whole family for spending time with Oskar. Without this help it would not have been possible to finish the PhD.

## List of content

**Paper II** **The significance of hydraulic jacking for grout consumption during high pressure pre-grouting in Norwegian tunnelling**

**Paper III** **Investigation and assessment of pre-grouted rock mass.**

**Appendix A** **Flowchart of algorithm to detect hydraulic jacking in grouting logs**

## List of abbreviations and acronyms

| | |
|---|---|
| A | Amphibolite |
| D&B | Drilling and blasting |
| GA | Garnet amphibolite |
| GG | Granitic gneiss |
| GIN | Grout Intensity Number |
| HJ | Hydraulic jacking |
| JRC | Joint roughness coefficient |
| LRIR | Allowable inflow rate per 100 metres of tunnel |
| MFC | Microfine cement grout |
| MWD | Measurements while drilling |
| OPC | Ordinary Portland cement / standard cement grout |
| OTV | Optical Televiewer |
| PF index | Pressure Flow index |
| PG | Pegmatite |
| PP | Poor pegmatite |
| RTGC | Real Time Grouting Control method |
| SCG | Supracrustal gneiss |
| TIGHT | True Improvement in Grouting High pressure Technology for tunnelling |
| TTG | Tonalitic gneiss |
| w/c ratio | Water cement ratio in kg |

# 1 Introduction

## 1.1 Background

Tunnels have different requirements regarding allowable water inflow, depending on the use and location. Often, the most important factor when determining the allowable water inflow is safeguarding the environment above the tunnel. In Scandinavian tunnelling, most of the tunnels are excavated beneath the groundwater level. If the area above has infrastructure, residential buildings, agriculture, vegetation or lakes, it is important to ensure that the groundwater level is not lowered to a level that could result in damage on infrastructure and buildings due to ground surface settlement, drainage of lakes and desiccation of vegetation.

The most common method for reducing groundwater ingress into tunnels in Norwegian tunnelling, is cement grouting ahead of the tunnel work face, termed pre-grouting (short for pre-excavation grouting). In Norwegian rock mass pre-grouting practice the grouting pressure is significantly higher than in other countries worldwide. The use of high pressure originates from decades of practical experience in grouting of rock caverns, subsea tunnels and hydropower projects. Furthermore, the grouting pressure was slightly increased and standardised by the research project "Tunnels for the citizens" (finished in 2004), but there is little research to justify the usage of high pressure. Despite a wide empirical basis there are unresolved questions related to the use of high grouting pressure.

Some questions related to the use of high grouting pressure are as following:

1. Which pressure is optimal to achieve a successful grouting, regarding restriction of water ingress, grout consumption and usage of time?
2. What happens in the rock mass when grouting with high pressures? This question especially implies to situations where the grouting pressure is higher than the minor principal stress in the rock mass.
3. Is the grout spread in the rock mass as expected, regarding the use of high pressure, type of grout and fracture apertures?

True Improvement in Grouting High pressure Technology for tunnelling (TIGHT) was a competence building project carried out in the period of 2014 to 2018. This PhD is written in connection with TIGHT. The research project had the objective of gaining in-depth competence on how the rock mass is affected by various grouting parameters, in particular high pressure and by this conclude how to design for optimal water controlling of tunnels. The main goal of the project was to increase the understanding of rock mass grouting to such extent that it would lead to a significant differentiation on use of materials and pressure depending on the actual circumstances for any given project. Improved knowledge of grout materials, grout equipment and grout strategies lead to cost- and time effective work and reduce the risk of constructional or operational requirements not being met. The research approach for TIGHT is illustrated in Figure 1.

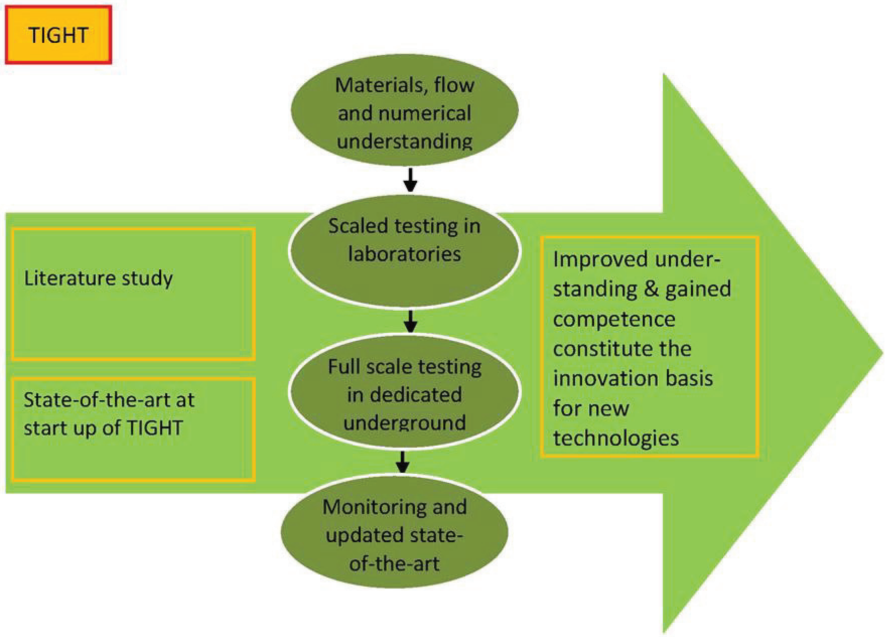


Figure 1: Research approach for TIGHT (Grøv, 2013)

## 1.2 Objectives of the thesis and research design

The fundamental objective for this PhD was to achieve an increased understanding of rock mass grouting with the use of high grouting pressures and to investigate if the current practice of using high grouting pressure is optimal regarding reduction of inflow into tunnels, and in line with good economy and environmental aspects.

To establish the most suitable method for investigating these questions and increase the general understanding of Norwegian rock mass grouting, a pre-study of grouting works was conducted. In the pre-study a general overview of available data from grouting works was gathered and studied. Through the TIGHT project, a large quantity of data from finished tunnels and tunnels under construction was made available by the project participants. Bane NOR, Statens Vegvesen, LNS, Veidekke and Bever Control shared all information asked for.

Examples of such data are:

- Grouting logs from grouting rigs.
- Geological mapping in the tunnels.
- MWD data (Measurement While Drilling).
- Geological reports from pre-study.
- Final reports from finished tunnels.

As different approaches were considered to best reach the goals for the PhD, substantial time was spent to attain a general understanding of the available data from finished tunnel projects, and tunnels under construction were visited for studying pre-grouting on-site.

One of the first approaches was to evaluate grout consumption in regard to MWD data. MWD data is data acquired during drilling, such as rotational speed, torque and load on the drill bit. Interpretation of parameters acquired through MWD is among other things used for rock mass characterisation. After approximately three months it was chosen to discard this method. The main reason for this was that the MWD data could not reveal large water conductive fractures in generally good rock. Larger zones of weak or fractured rock could be revealed, but these zones were not necessary the most water conductive, neither not always the zones that consumed most grout. In general, poor correlation was found between grout consumption and MWD data, and it was therefore chosen to use other approaches to assess the grout consumption during rock mass grouting.

The second approach was to evaluate grouted rock mass by documenting grout spread in rock fractures and measurement of hydraulic transmissivity. This approach turned out to be costly and beyond the funding of this PhD. Substantial time was spent on finding funding and a tunnel project that would allow drilling through pre-grouted rock mass. Bane NOR generously decided to cover the costs for the testing and dispositioned their network of tunnels at Åsland construction site (Follo Line project). The test holes could not be drilled before the tunnels were available, because the testing would block the tunnels for long periods of time. The tests were performed during the spring of 2017, 18 months after the PhD started. Because of repeatedly erroneous analyses by the company that conducted the OTV measurements, the final and correct dataset was not received before the fall of 2018. The test plan turned out to be complicated and time-consuming within the scope of a PhD, and the PhD could not rely on the results from this study. Therefore, this study became only a supplement to the main study area of this PhD.

The third approach was to analyse time series of grout pressure and flow logged by grouting rigs, which turned out to become the main study area of the PhD. During grouting, the grouting rig logs pressure, flow rate and grouted volume. Each grouting hole has a unique pressure and flow response, dependent on many different factors, such as grout properties and fracture characteristics. During the initial study of the logs from the grouting, it was discovered frequent occurrences with distinct changes in the grouting behaviour. In most cases the change was interpreted to origin from hydraulic jacking (HJ) of fractures in the rock mass and the occurrences appeared to have a significant effect on the grout consumption. Thus, it was chosen to find a method to interpret HJ in the grouting logs and collect data related to these events. The purpose of this was to reveal the consequences of HJ and if HJ could be predicted. No examples of noteworthy research based on field data were found in literature, only theoretical approaches to the prediction of HJ and consequences, such as Rafi and Stille (2014), Warner (2004) and Lombardi (1985).

As result, this thesis covers two main approaches for investigating pre-grouting: analysis of data from the grouting process and investigation and assessment of pre-grouted rock mass. In the thesis, interpretation of grout pressure and flow rate data from grouting rigs will be addressed first, followed by the assessment of pre-grouted rock mass, as illustrated in the flow chart in Figure 2.

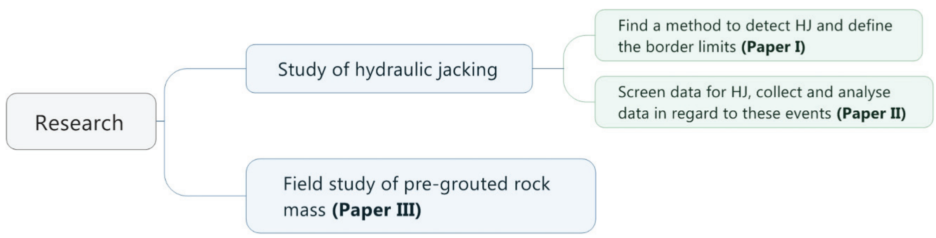


Figure 2: Flow chart illustrating the two different studies and publications.

In general, the PhD study has investigated pre-grouting under "normal" (most common) Norwegian tunnelling conditions, which means that no extreme cases regarding high overburden (>200m), high groundwater pressures or challenging geological conditions were included in the study. Weakness zones which are typical to encounter in Norwegian tunnelling are however included.

Throughout this PhD the term fracture is used for all discontinuities in the rock mass. As described by Palmström (2015), various types of discontinuities are termed according to size and composition, e.g. joints, fissures and cracks. During pre-grouting there are no knowledge regarding size and composition of the discontinuities in the rock mass, and fractures is therefore used as a general term.

### 1.3 Structure of the thesis

This thesis is comprised of three parts; the first part consisting of seven chapters, the second part of three research papers and the last part of one appendix.

Part 1 includes the following chapters:

**Chapter 1: Background and the objective of the thesis**, describes the objective and motivation of the PhD work, the way it was included in a major research project and the important reasons of rock mass grouting in Norwegian tunnelling projects.

**Chapter 2: Theoretical framework for the field of study**, gives a theoretical basis for the field of study, with the purpose of providing an understanding of the performed research and aiding the discussions of the results.

**Chapter 3: Interpretation of hydraulic jacking (paper I and II)**, gives a summary of a developed methodology for detecting hydraulic jacking of rock fractures, published in Paper I, followed by a study where the developed methodology is used for the assessment of grout consumption in regard to hydraulic jacking of rock fractures (Paper II).

**Chapter 4: Investigation and assessment of pre-grouted rock mass (paper III)**, contains a summary of the study presented in Paper III, where pre-grouted rock mass is investigated to document grout spread in fractures and transmissivity of the rock mass in the close surrounding to the tunnel.

**Chapter 5: Prediction of hydraulic jacking**, gives a brief presentation of hydraulic jacking in regard to overburden and grout consumption, with the purpose to establish if hydraulic jacking can be predicted by these parameters. There is not written a paper related to these findings.

**Chapter 6: PhD findings in regard to the practical work of pre-grouting in Norway and the use of the PF index**, provides some practical recommendations to the tunnelling industry based on the findings of the PhD study. The practical use of the developed PF index for evaluation of the relationship between grouting pressure and flow rate during grouting is also presented.

**Chapter 7: Main conclusions of the PhD research**, presents a short summary for the major findings of the PhD research.

## 1.4 List of publications and declaration of authorship

**Paper I:**

Strømsvik, Helene; Morud, John Christian; Grøv, Eivind, 2018. *Development of an algorithm to detect hydraulic jacking in high pressure rock mass grouting and introduction of the PF index*. Tunnelling and Underground Space Technology 81, pp.16-25, https://doi.org/10.1016/j.tust.2018.06.027

*The research and writing were performed by Helene Strømsvik. John C. Morud contributed with the development of the numerical model to illustrate pressure and flow behaviour when grouting fractures of different geometries. Eivind Grøv was a discussion partner during the interpretation of the data and assisting with the outline of the paper and proofreading.*

**Paper II:**

Strømsvik, Helene. *The significance of hydraulic jacking for grout consumption during high pressure pre-grouting in Norwegian tunnelling*.
Tunnelling and Underground Space Technology 90, pp. 357-368, https://doi.org/10.1016/j.tust.2019.05.014

*No co-authors*

**Paper III:**

Strømsvik, Helene; Gammelsæter, Bjørnar. *Investigation and assessment of pre-grouted rock mass*. Submitted April 8$^{th}$ 2019 to: Bulletin of Engineering Geology and the Environment. Current status: under review.

*The idea for the research, the main part of the data interpretation and writing of the paper were performed by Helene Strømsvik. Bjørnar Gammelsæter contributed with practical assistance of the field work, discussion of the results and proofreading of the paper.*

# 2 Theoretical framework for the field of study

This chapter contains a theoretical basis for the field of study, with the purpose of providing an understanding of the performed research and aiding the discussions of the results.

## 2.1 Water ingress in tunnels

If no measures are performed to reduce water inflow into tunnels, they will work as very efficient drains of the groundwater. Lowering of the groundwater table has many adverse consequences, such as damage on infrastructure and buildings due to ground surface settlement, drainage of lakes and desiccation of vegetation. When tunnelling in urban areas it is therefore of great importance to reduce the water ingress to an acceptable level. In sensitive areas, it is also important that the groundwater table is stable during construction of the tunnels. High ingress of water into a tunnel can also be unfavourable for the work during construction and water seepage in tunnels under operation could lead to corrosion of installations, slippery roads or formation of icicles during the winter.

## 2.2 Water inflow restrictions

Tunnels have different requirements regarding allowable water inflow, depending on the use and location. In Norwegian tunnelling projects the hydrogeology and sensitivity for fluctuations in the groundwater table in the area are evaluated in a pre-study. Based on these investigations an inflow limit per 100 metres of tunnel is estimated. Table 1 presents common inflow limits.

Table 1: Common inflow limits in Norwegian tunnels (modified from Aarset et al. (2011))

| | **Strict requirement** | **Immediate requirement** | **Moderate requirement** |
|---|---|---|---|
| **Allowable inflow** | 5 l/min/100m | 10 l/min/100m | 20 l/min/100m |
| **Functional requirements** | Sensitive surroundings | Moderately sensitive | Site-dependent |

In sensitive areas with strict requirements, the groundwater level is monitored during construction and the limits for water ingress are adjusted continuously to meet the demands. Buildings founded on soil, particularly clay are very prone to settlement with even small reductions in the groundwater table. When tunnelling in the rock mass beneath such buildings it is necessary to monitor the buildings in regard to settlement and the requirements for water inflow could be modified according to these measurements.

## 2.3 Pre-excavation rock mass grouting

Based on Norwegian experience gained from several projects during the last 2-3 decades, the best method for reducing groundwater ingress in tunnels has been found to be cement grouting ahead of the tunnel work face, termed pre-excavation grouting, or pre-grouting. This is the common measure for reducing water ingress into tunnels and underground spaces in Norwegian tunnelling.

### *2.3.1 Method description*

Pre-grouting is performed from inside the tunnel before the next excavation step and can be used for any excavation method. In this thesis pre-grouting using cement based grouts performed in connection with drilling and blasting (D&B) is investigated. The cause for this limitation is the availability of data, as the most common grout types are cement based and D&B is by far the most common excavation method in Norwegian tunnelling.

The typical setup for pre-grouting when excavating with D&B is drilling 25-70 holes of 15-30 metres length at the face of the tunnel, depending on the tunnel face area and geology. Packers attached to grouting rods are placed approximately 2 metres into the drill holes. The grouting rigs commonly have 3-4 grout lines which can operate simultaneously. The grouting is first performed in the bottom holes, moving upwards. This setup is illustrated in Figure 3.

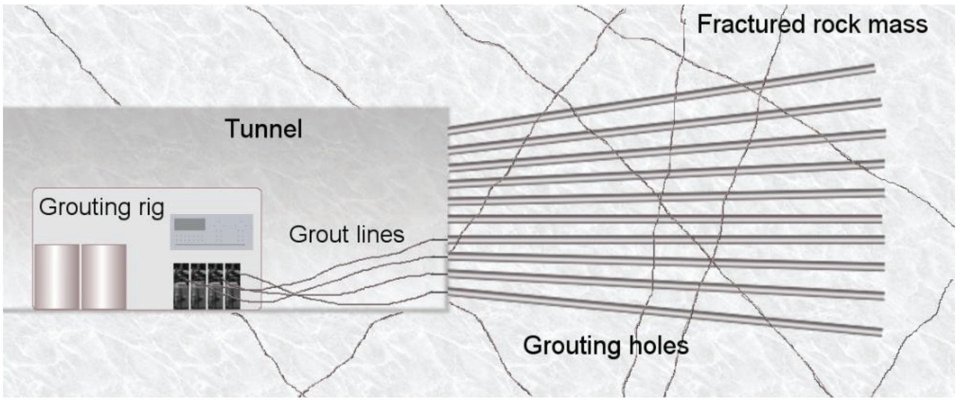


Figure 3: Illustration of pre-grouting performed in a tunnel (Strømsvik, 2019).

The decision basis to conclude whether or not the grouting is sufficient is defined by stop criteria, which are discussed in subchapter 2.3.6.

In areas with strict requirements regarding water inflow, rounds of pre-grouting are performed with overlap, termed continuous or systematic pre-grouting. During continuous pre-grouting when excavating with D&B, it is common to perform one round of grouting every third blast (in Norwegian tunnelling, each round of blasting is 5 metres under normal conditions). When the requirements for water inflow is less strict probe drilling is continuously performed at the tunnel face and grouting performed when the water ingress from the probe holes is higher than a pre-determined limit, this method is termed grouting when deemed necessary.

### 2.3.2 Types of grouts in Norwegian pre-grouting

According to Aarset et al. (2011) the most common grout types in Norwegian tunnelling are cement based grouts with different fineness and additives. Non-cement based grouts are most frequently used in post-grouting, which is performed if the residual leakage after pre-grouting is too large.

Table 2 presents examples of cement-based grouts on the market in Norway. According to the Norwegian road authorities ordinary Portland cement, or standard cement (OPC) is defined as a cement with $D_{95}$ between 25 µm and 40 µm, while microfine cement (MFC) is defined as a cement with $D_{95}$ less than 25 µm.

Table 2: Examples of cements for rock mass grouting on the Norwegian market (modified from Holmøy et al. (2015)).

| **Cement** | **Blaine ($m^2$/kg)** | **Specific surface ($m^2$/kg)** | **$D_{95}$ (µm)** |
|---|---|---|---|
| *OPC* | | | |
| Norcem Industry cement | 550 | | 40 |
| Injisering 30 (Cementa) | | 1300 | 30 |
| *MFC* | | | |
| Microfine 20 (Norcem) | 750 | 2550 | 20 |
| Microfin 20 (Maipei) | | 2650 | 20 |
| MasterRoc MP 650 (BASF) | 650 | | 20 |
| Ultrafin 16 (Norcem) | | 1600 | 16 |
| MasterRoc MP 800 (BASF) | 800 | | 15 |
| Ultrafin 12 (Norcem) | | 220 | 12 |

During the field research in the PhD study it was observed that common additives in the OPC were silica and superplasticizer in the MFC.

### 2.3.3 Environment and costs

As described in subchapter 2.3.2, the most common grout used for pre-grouting in Norway is cement based, made from Portland clinker. According to Andrew (2018), decomposition of carbonates is one of the three largest contributors to anthropogenic emissions of carbon dioxide to the atmosphere, the other two are fossil fuels and deforestation. Cement production is the largest source of carbon dioxide emission from the decomposition of carbonates. This reflects that excessive use of cement during pre-grouting is not only undesirable for the project economy, but also for the global carbon footprint.

Pre-grouting is an expensive and time-consuming process in Norwegian tunnelling. Figure 4 shows estimated costs for grouting per metres of tunnel related to the allowable inflow rate per 100 metres of tunnel (LRIR) in 2012.

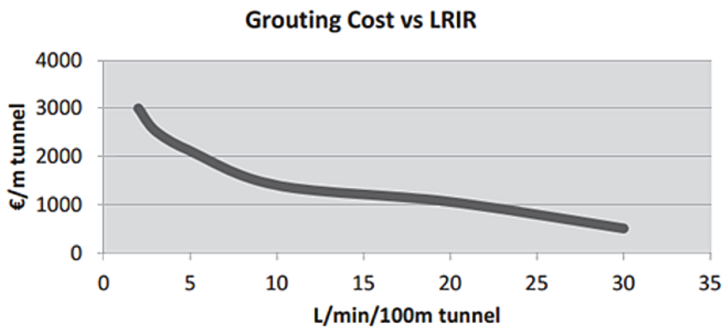


Figure 4: Costs per metres of tunnel relative to allowable inflow rate per 100 metres of tunnel (LRIR), (Grøv and Woldmo, 2012).

The effect cement production has on the global environment and pre-grouting costs in regard to project costs both illustrates the importance of understanding the grouting process and applying the most optimal grouting procedure to achieve an optimal result, which is a tunnel that is tight enough for its purpose without excessive use of grout or time.

### *2.3.4 Grout spread in rock mass and grout properties*

When describing grout spread through rock fractures, many of the main principles are similar with waterflow through rock mass. The grout flows along the paths of least resistance, i.e. where the aperture is greatest, resulting in different flow distribution within each fracture. The main difference is that cement-based grout is a non-Newtonian fluid, which according to Stille (2015) can be described as a particle-based Bingham fluid. A Bingham fluid is a viscous fluid that has a yield strength that needs to be exceeded before it will flow. The most significant difference between water and grout is that when pumping grout into fractures, the frictional forces in the fluid are significantly higher, resulting in pressure increase. Also, the grout is not able to penetrate the same fracture volume as water.

In Norwegian projects one of the goals during pre-grouting is to fill fractures 5-6 metres beyond the profile of the tunnel (Aarset et al., 2011). During and after the grouting it is not possible to evaluate if this criterion is met or determine how the grout is distributed, regarding apertures of the fractures. The result of pre-grouting is determined by the degree of tightness after construction. This limits the knowledge and learning of where the grout is spreading in the rock mass and how the pre-grouting can be optimized regarding tightness, grout consumption and usage of time.

According to Stille (2015) the grouts ability to penetrate fine fractures (penetrability) depends on the relationship between the size of the grains and fracture apertures. In fine-grained cement this relationship is complex, mainly due to an increase in specific surface area, resulting in greater surface activity. The penetrability is also affected by the water/cement ratio (w/c ratio), cement quality, type of mixer and temperature. Stille (2015) presents laboratory tests showing that INJ30 cement, was found to have a critical aperture of 90-157 μm, dependent on w/c ratio, type of mixer and temperatures. The critical aperture is defined as the aperture sufficiently large for free grout flow, with no filtering. Draganović and Stille (2011) compared a cement with $d_{95}$ of 32 μm with two finer cements with $d_{95}$ of 12 μm and 16 μm, where the finer cements were found to have a considerably poorer penetrability. According to Stille (2012), studies has shown that the penetrability of cements is decreasing with $d_{95}$ smaller than 20 μm.

A study presented by Håkansson et al. (1992) found an increase in both yield stress and plastic viscosity with an increase in specific surface of the particles in cement based grouts. Studies presented by Skjølsvold and Justnes (2016) in connection with the TIGHT project, found that in some microfine cements the setting started after approximately 30 minutes. According to Grøv and Woldmo (2012) the short setting time of microfine cement can give less consumption of grout compared to standard grout cement.

#### *2.3.5 Use of high grouting pressure and hydraulic jacking*

When grout is pumped from the grouting rig into the grouting hole in the rock mass there is a pressure development in the grout. This pressure is dependent on different factors and varies throughout the grouting course. The two main factors contributing to the development of pressure is the flow rate of grout and the ability of the rock mass to consume the grout. As described in subchapter 2.3.3, cement based grout is a particle-based fluid that can be described as a Bingham material. Thus, the frictional resistance in the grout material will also be a major contributor to the pressure build-up in the grout.

To achieve a spread of grout in the fractures a certain amount of pressure is required and high pressure is considered to be optimal to ensure best possible penetration into fine fractures.

Draganović and Stille (2011) concluded that high pressure could also improve penetrability by eroding partially-built plugs along edges of fractures and hold the fractures open for a long time. Also, a higher penetration rate is achieved by using high pressure.

The use of high grouting pressure does not only have positive effects on the grouting results. If the grouting pressure inside the fracture exceeds the normal pressure acting on the fracture, the aperture of the fracture will increase. This process is called hydraulic jacking (HJ). A description of this process is found in Rafi and Stille (2015) and is discussed in Paper I.

Stille (2015) presents the following negative consequences of HJ during pre-grouting:

1. Higher consumption of grout, due to higher flow rate and increased volume of fractures.
2. Uplift of the overburden, if the fractures subjected to HJ are close to horizonal oriented.
3. Increased transmissivity outside the grouted zone, due to increased apertures of fractures.
4. Finer factures can be exposed to compression during grouting, making them more difficult to grout.

In Norwegian rock mass grouting it is considered that in some conditions HJ could be beneficial and improve the effectiveness of grouting. (Aarset et al., 2011). This could for example be in conditions with clay rich fracture fillings where the connectivity between asperities in the fractures is high, resulting in water transport in ungrouted cannels along the asperities, as described by Pusch et al. (1991), and discussed in Paper II. By analysing the pressure and flow rate of grout during rock mass grouting it is possible to detect occurrences of HJ. This is further discussed in subchapter 2.3.8 and Paper I.

One of the main challenges and a disputed subject in rock mass grouting is which pressure to apply. Pre-excavation grouting in Norway typically operates with grouting pressures of 60-80 bar, but ranges between 20-100 bar, as shown in Table 3. This is significantly higher than most other places in the world. The high pressure to some extent originates from the research project "Tunnels for the citizens" finished in 2004, where it was concluded that such high pressures would increase the penetration of grout and grouting capacity (Klüver and Kveen, 2004).

Table 3: Grout pressure applied for Norwegian infrastructure tunnels in urban areas (modified from Grøv et al. (2014)).

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

Figure 5 presents examples of "rules of thumb" for grouting pressure relative to overburden, used as guidance for selecting grouting pressure outside of Norway. The purpose of the rules of thumb is to give guidance to select a grouting pressure that give minimum risk of uplift of the overburden due to HJ of fractures. It can be observed that the Swedish rule of thumb for "good rock" is the only one close to Norwegian practice.

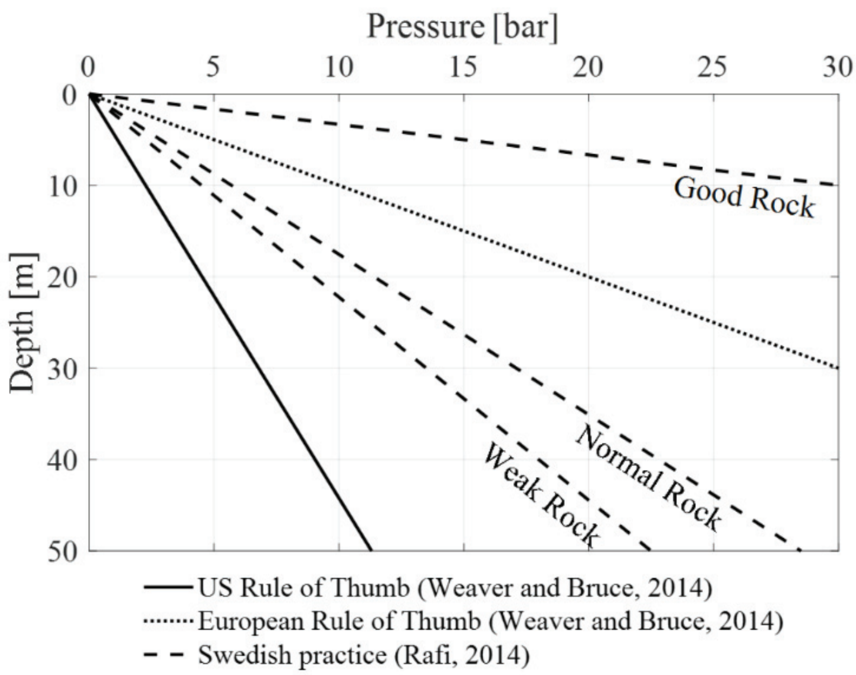


Figure 5: Rules of thumb for grouting pressure relative to overburden (with data from Rafi (2014) and Weaver and Bruce (2014))

### 2.3.6 *Stop criteria for pre-grouting*

One of the most difficult decisions during pre-grouting is determining when to decide that the grout spread is sufficient to ensure the required tightness and consequently stop the grouting. At present time there is no absolute method for confirming the spread of the grout in the rock mass fractures during the grouting works. This is the main reason for the challenges of defining the stop criteria. As a result of this, the stop criteria are often based on empiricism. This is the case also in Norwegian grouting practice.

In pre-grouting in Norwegian tunnelling, the following stop criteria are most common:

1. Reaching a pre-determined grouting pressure, at the same time as the grout flow should be small, or close to zero. The pre-determined pressure is often 80 bar, but lower if the grouting is performed with low overburden, or close to the tunnel entrance.

2. After a pre-determined volume is grouted, with insufficient decrease in the grout flow and/or insufficient increase in pressure, the w/c ratio of the grout is reduced.

3. After a pre-determined maximum volume is grouted, with insufficient decrease in the grout flow and/or insufficient increase in pressure, the grouting is ended.

It can be noted that during field observations of pre-grouting in this study, it was apparent that different grouting rig operators solved their given stop criteria by using different methods. Some operators have a "soft" start with slow increase in flow, while others start with high flow and some operators have more frequent pauses during the grouting of a hole. All observed operators had reasons for their approach, and it was apparent that post-grouting was undesirable among the tunnel workers.

Examples of more theoretical based methods for deciding when to end the grouting are the GIN principle and the Real Time Grouting Control method (RTGC method). The GIN principle is presented in detail by Lombardi and Deere (1993) and Lombardi (1997). The principle is developed from laboratory studies on grout, theoretical studies and monitoring of grout pressure and grout volume in dam grouting. The main idea in this grouting principle is to choose a GIN limiting curve (also termed limiting envelope), related to the grouting pressure and the accumulated grouted volume, to find the most appropriate moment to end the grouting. Examples of proposed limiting curves are presented in Figure 6. If the grouting exceeds the limiting curve, there is a high risk for the occurrence of HJ. This principle is strongly based on the theory that HJ is more prone to happen if the grout spread in the fracture is large.

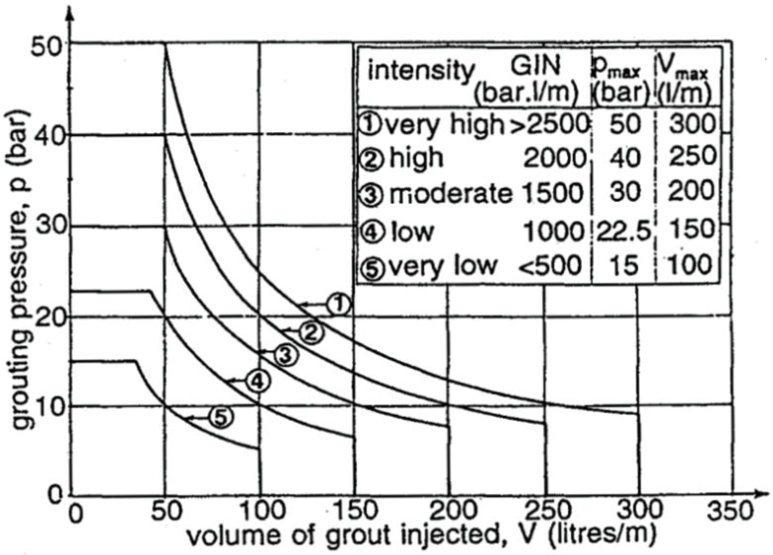


Figure 6: Proposed limiting curves for grouting (Lombardi, 1997)

Eq. 1 presents a formula for estimating the splitting force (F) during grouting of a rock joint, presented by Lombardi (1985). Since the publication discusses the grouting of a joint it is reasonable to assume that the splitting force in this context is referring to the hydraulic force in the fracture that could lead to HJ.

$$F = \frac{V^{2/3}\, P^{5/3}}{C^{2/3}} \tag{1}$$

where $V$ is grouted volume, $P$ is grouting pressure and $C$ is cohesion of the grout.

The grouting intensity should be chosen according to site conditions. It can be observed that the limiting curves are within a pressure range far lower than the grouting pressures used in Norway. In this regard it is important to understand that the GIN principle is developed for dam grouting, where the grout holes are drilled from the surface and the grouting is performed relatively close to the ground surface.

The RTGC method is a result from decades of research on grout rheology and grout spread in fractures, performed in Sweden. A thorough description and the foundation for the method can be found in Stille (2015). In short, the developed equations for grout spread are reversed. Instead of predicting the grout course based on data from the fracture geometry, these values are estimated based on the actual grout flow and rheological properties of the grout. This way the grout spread in the smallest and largest apertures can be estimated. According to Kobayashi et al. (2008) the grouting is completed when the grout spread in the smallest fracture that needs to be sealed is sufficient, or before the grout spread in the largest fracture aperture has reached a maximum limit. This method requires close monitoring of the grout properties and continuous advanced calculations must be available at the grouting site during the grouting procedure. How this could be applicable in real life is still an ongoing research in Sweden and the results of this is very interesting to follow.

#### *2.3.7 Data collected from grouting rigs during grouting*

The grouting rig in the tunnel displays pressure and the flow rate during the grouting works, which helps the operators to follow the grouting progress regarding the given stop criteria. Furthermore, the computer system on the rig collect the data. How the data is collected differs, depending on the equipment supplier. Some sample data every $10^{th}$ second, while others sample data according to the pump strokes of one of the grouting lines. Also, the types of sampled data in regard to estimation of pumped volume differs.

When performing analyses with data from different types of grouting rigs it is therefore important to understand how the data is logged and make adapted algorithms and interpretations according to the rig data. In some cases, the logging interval is too sparse to perform detailed analyses on the collected data, this issue is discussed in Paper I.

#### *2.3.8 Detection of hydraulic jacking during pre-grouting*

There are not many publications which discuss detailed and quantified definition of HJ during rock mass grouting. Some authors however discuss how HJ can be discovered by using the logged data.

Lombardi and Deere (1993) suggest how HJ can be detected by looking at the ratio between the flow rate (Q) and the pressure (P), as illustrated in Figure 7. The curve in graph (d) is showing a decrease in the Q/P ratio over time, as the resistance to grout is increasing. The sharp peak in the graph is referred to as HJ of fractures, or hydraulic fracturing, which is the formation of new fractures. It can be observed that just before the pronounced peak in the Q/P ratio, the pressure is dropping (a) and the flow rate is increasing (b).

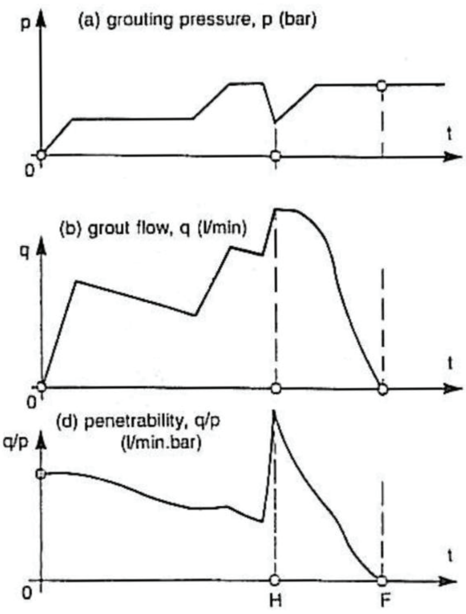


Figure 7: a) Grouting pressure during hydraulic jacking/fracturing (H), b) flow during H and d) Q/P ratio during H (Lombardi and Deere, 1993).

Rafi (2014) used Figure 8 as an example on how one can recognise HJ of a rock fracture by the use of the rheological properties of the grout in addition to the grouting log. The figure is based on the RTGC method, where the grouting data is evaluated with back analysis to estimate the expected flow. In this case the predicted flow rate is estimated to decrease while the pressure is stabilizing, as expected when cement based grout is flowing as a 1D (channel) or 2D (disc) flow in a fracture with a constant aperture. The recorded flow is deviating from the estimated flow behaviour and is steady, not decreasing as expected. If the recorded flow behaviour is deviating from the predicted flow it is, according to Rafi (2014), either due to wrong assumption of the dimensionality, the aperture size or, HJ of the fracture.

Figure 9 shows a graph of the recorded flow and pressure from a grouting project in Laos presented by Rafi and Stille (2014), where predicted grout flow is estimated by using the RTGC-method. The onset of HJ is assumed to be when the recorded flow is deviating from the predicted flow, which in this case is when the pressure is stabilizing at a steady flow rate. There might be other explanations for this type of behaviour of the flow and pressure, as there are several plausible reasons for wrong assumption of the dimensionality and change in aperture size in a fracture, which could represent a pressure and flow behaviour similar to events representing hydraulic jacking or fracturing. This is discussed in Paper I.

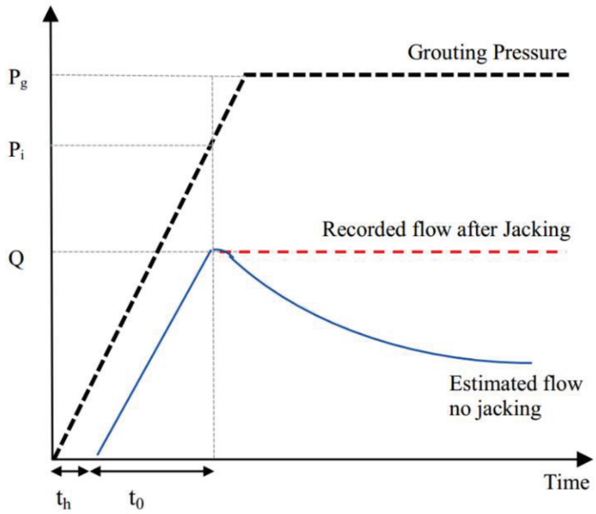


Figure 8: Recorded flow and estimated flow using the RTGC method (Rafi, 2014).

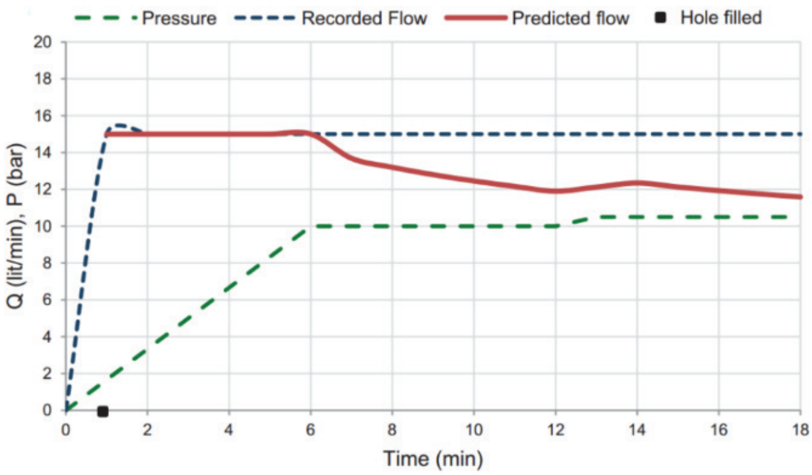


Figure 9: Recorded flow and estimated flow using the RTGC method on real data (Rafi and Stille, 2014).

Warner (2004) emphasized the pressure drop at a constant flowrate as an indicator of HJ, but also listed numerous events that could cause a pressure drop at a constant flow rate:

1. Hydraulic fracturing.
2. Grout loss into a concealed pipe or other substructure.
3. Outward displacement of a downslope or retaining wall.
4. Grout entering much larger fractures or voids.
5. Grout encountering a softer or more permeable formation.
6. Thinning the grout or other rheological change that increases mobility of the grout.
7. Leakage of grout and pump malfunction.

Warner (2004) also states that reduction in the pressure increase during the pressure build-up at a constant flow rate, could indicate a significant event in the grouting process.

The technique of hydraulic fracturing for estimating the minimum in-situ stress component in rock mass, is also a potential source for learning about the behaviour of flow and pressure in HJ during rock mass grouting. The method is performed by isolating a section of a borehole by pressurizing two inflatable packers. The section is placed in intact rock with no prior fractures. Water is pumped into the sealed off section until a fracture is generated, known as hydraulic fracturing. The procedure is repeated at least two times, resulting in a reopening of the generated fracture, known as hydraulic jacking (HJ). The procedure is described in detail in ASTM International (2004). The method is well established and a common method for stress measurements in rock mass in Norway. In this regard it is important to consider the significant difference in rheology between water and cement based grouts.

The presented theory in this subchapter (2.3.8) is the base for the algorithm for detecting HJ during grouting in the grouting rig data, which is presented in Paper I and chapter 3 in the thesis.

#### *2.3.9 General opinions regarding grouting pressure and hydraulic jacking*

An issue which does not clearly become visible by studying research literature, is the ongoing discussion regarding which grouting pressure to apply and if HJ should be avoided or not.

During the Nordic Grouting Symposium in Oslo in 2017, it was held a workshop with the following questions:

1. Is HJ wanted or unwanted?
2. How can we balance high pressure as something required for good quality grouting with the risk of jacking?

These questions were discussed in 9 groups of approximately 10 members, reflecting a good representation of the grouting community in the Nordic countries. The statistics presented in Figure 10 were extracted from the group answers, which was retrieved from the chairman of the scientific committee of the conference, Eivind Grøv.

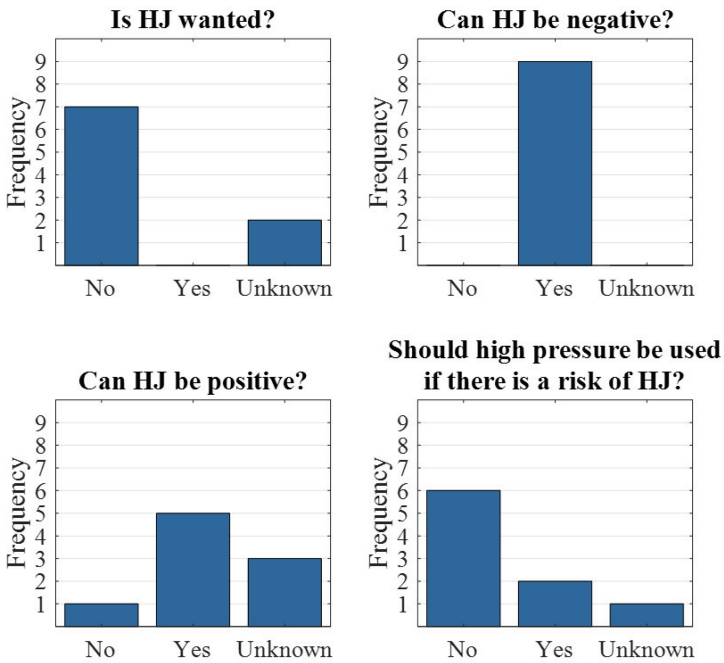


Figure 10: Frequency distribution plots showing group opinions during the workshop in Nordic Grouting Symposium 2017.

In general, there seems to be a consensus that HJ is not a desired occurrence, which have negative impact on the grouting. Some degree of high pressure seems to be accepted as a necessity to achieve a good penetration of grout, but the occurrence of HJ should be avoided if possible. Five out of the nine groups also meant that HJ could in some cases have a positive impact on the grouting.

These results from the workshop are not based on scientific research but should be regarded as general opinions from persons who joined the workshop. It is nevertheless interesting to have this “common opinion” about grouting with high pressure and HJ.

# 3 Interpretation of hydraulic jacking (paper I and II)

This chapter contains a summary of a developed methodology for detecting HJ of rock fractures, published in Paper I, followed by description of a study where the developed methodology is used for the assessment of grout consumption in regard to HJ of rock fractures (Paper II).

## 3.1 Method for detection of hydraulic jacking

### *3.1.1 Data acquisition and collaboration with WP 7 in TIGHT*

The data collected for this study was mainly:

1. Log files from grouting rigs supplied by Veidekke, LNS and Bever Control.
2. Geological mapping results supplied by Statens Vegvesen and Bane NOR.

Table 4 gives an overview of tunnels where data were collected and used for the development of a method for detecting HJ during pre-grouting.

Table 4: Tunnels and number of rounds of pre-grouting used as database for the development of the algorithm to detect HJ.

| **Project** | **Rounds of pre-grouting** |
|---|---|
| Kongsberg tunnel | 21 |
| Åsland AN | 20 |
| Åsland AS | 35 |
| Skogafjell tunnel | 37 |
| Lyshorn tunnel | 23 |
| Nøklegård tunnel | 22 |
| Sjøskogen tunnel | 7 |

The data was primarily retrieved from finished tunnelling projects. Two of the tunnels were under construction during the study and were visited several times; the Kongsberg tunnel and Lyshorn tunnel. Åsland was also visited, but the observed rounds of pre-grouting were not included in the analysis, as other more suitable pre-grouted tunnel sections were selected from this site.

The visits at tunnel sites and the observation of pre-grouting were connected to work package 7 (WP 7) in the TIGHT project. WP 7 was a study where pressure loggers were designed to log the grouting pressure inside the grouting holes during pre-grouting, as described in Tunbridge et al. (2016b).

The participation in WP 7 had the following purposes:

1. Learn how the logging system on the grouting rig represents the actual pressure in the grouting hole.
2. The interpretation of the relationship between grout pressure and flow rate was considerable easier after being on-site during pre-grouting. Many of the unclear and unexplainable occurrences in the grouting log had practical explanations from the work in the tunnel, such as washing of equipment, tight grouting rods, pump malfunction and grout leakage into the tunnel.
3. WP 7 created a great opportunity to establish contact with project owners and contractors, including workers and grouting rig operators. The work in the tunnels gave plenty of time for deliberations about the rig operators strategies for meeting the criteria given in the contracts, combined with their own opinion of how to meet the inflow requirements.
4. It was also a desire to help with WP 7 with retrieving good quality data from suitable sites.

The main contribution to WP 7 was finding suitable tunnel projects for testing, communication and arrangement with contractors, assembly and testing of equipment and transport of equipment to the construction site. Furthermore, instructions for equipment installation and overseeing the grouting progress were performed at the construction site. Some time was also spent on evaluation of the test results and they were discussed with Lloyd Tunbridge, which was responsible for the work performed in this WP. The results from the testing in WP 7 were not planned to be published as a part of the PhD-work, therefore limited contribution to the interpretation of the results in WP 7 was offered. The following publications were made with data from WP 7 during the TIGHT project: Tunbridge et al. (2016a), Tunbridge et al. (2016b) and Tunbridge et al. (2017). After the project was finished some of the data retrieved from the field tests in WP 7 was assigned to a master student at NTNU. The MSc thesis is scheduled to be completed in June 2019.

The participation in the WP 7 work gave a unique opportunity to get first-hand knowledge about the Norwegian grouting practice and was vital in the understanding and interpretation of grouting logs in this PhD.

### *3.1.2 Interpretation of hydraulic jacking from grouting logs*

Paper I has focus on how the collected data is interpreted to screen for HJ in grouting logs. HJ of fractures in a rock mass is very difficult to document, because it is not possible to observe the actual process of HJ and there are several other processes in rock mass grouting that can display similar pressure and flow behaviour as for HJ. For this reason, it was seen as necessary to give a thorough description of how HJ jacking was interpreted in this study, to ensure reproducibility of the results and allowing other researchers to understand the decision basis for detecting HJ. The developed method for detecting HJ is based on theory, which is adapted to the collected data in this study. The subsequent text gives a a summary of the development of the algorithm. The full description is found in Paper I.

The first step to interpret HJ in grouting logs was visual inspection of graphs made from the grouting logs combined with a literature study, followed by trial and error approach with data from the grouting projects presented in Table 4. Theories and material published by Lombardi and Deere (1993), Stille (2015), Warner (2004) and ASTM International (2004), presented in subchapter 2.3.8, were used as theoretical basis.

To obtain a better understanding of the behaviour of grout flowing through sections of changing geometry, a 2D numerical model was created to illustrate the behaviour of a Bingham fluid trough changing geometries. The model is presented in Paper I.

It was concluded that the following pressure and flow behaviour could indicate HJ; decreasing or stable pressure while the flow is increasing, or alternatively increasing or stable flow while the pressure is decreasing. Other events during the grouting could also exhibit the same pressure and flow behaviour, such as grout set into motion after stoppage, grout bursting through joint fillings, increase in flow while grouting an open void and rapid adjustments made by the operator. Paper I explains how some of these events can be excluded, when screening for HJ.

Interpretation of grouting pressure and flow rate as two separate variables, which are affected by oscillations by the piston/plunger pump and low sampling frequency, turned out to be a challenging task. It was soon discovered that it would be helpful to find a parameter that could represent the relation between grouting pressure and flow rate. Such a parameter could illustrate the grouting process and emphasize changes in the relationship between pressure and flow, that could indicate the occurrence of significant events, e.g. HJ and be helpful when defining boundary conditions for detecting HJ in the data.

It was found that by simply subtracting the grouting pressure from the flow rate the pressure build-up is represented in a distinct way, as well as the general relationship between pressure and flow. This value was named the PF index (Pressure Flow index). The PF index was first introduced as the QP index in Strømsvik and Grøv (2017), Eq. 2. To create a dimensionless value with a practical scale, some adjustments were made to the formula. Addition of 90, creates a positive value and multiplication with 0.9, adjusts the range of the scale. The was modified to make the index dimensionless, Eq. 3.

$$QP\ index = \ (Q_v - P + 90)\ \cdot 0.9 \tag{2}$$

$$PF\ index = 0.9 \,{}^{min}\!/_{l} \cdot Q_v - \frac{0.9 \cdot P}{1\ bar} + 81 \tag{3}$$

where $Q_v$ is flow [l/min] and $P$ is the grouting pressure [bar].

To be able to use the PF index as a tool for computerized interpretation of HJ on data logs from the Norwegian projects it was necessary to perform further adjustments by filtering the data, as described in Paper I. The PF index in combination with logged pressure, flow rate, pumped volume of grout and time were further used to create an algorithm which can indicate the onset of events plausible to be HJ. The algorithm was developed and tuned through the analysis of a great number of grouting rounds from varying geology, overburden, w/c ratios, grouting rigs, operators and projects, to ensure that the algorithm would work acceptably for the different tunnelling conditions.

The main attribute of the algorithm is that the PF index should increase by a given value within a limited timeframe, at the same time as the pressure is stable or reduced while the flow is stable or increasing. In addition, the algorithm can identify and exclude the following scenarios in the grouting course:

- Start of grouting, or restart after temporary stoppage.
- Rapid reduction of flow by operator.
- Rapid increase of flow by the operator.
- Repeated rhythmic pressure drops caused at the pump by the pumping cycle.

Figure 11 shows a flow chart illustrating the structure of the algorithm created for detecting events indicating HJ. A detailed flow chart of the algorithm is presented in Appendix A.

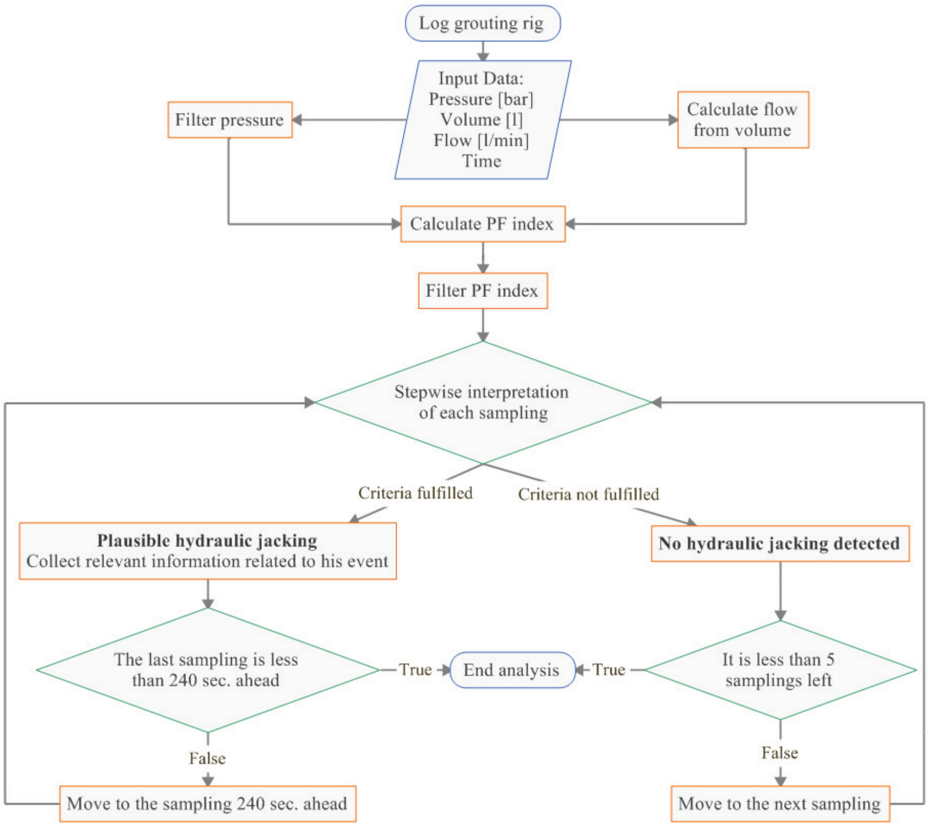


Figure 11: Flow chart showing the structure of the algorithm created to detect HJ in data logs from grouting rigs (Strømsvik et al., 2018).

When the algorithm had located potential HJ, data related to the event was collected, such as grouted volume, time, pressure and type of grout. Furthermore, data related to the grout round was collected, such as geological classification for the grouted rock mass and overburden.

Figure 12, examples a) and b) show a graphic view of two plausible events where the onset of HJ is identified by the algorithm. In example a) there is a pressure drop with a relative constant flow, in example b) there is both increase in flow and decrease in pressure where the algorithm indicates the plausible HJ. The case shown in b) is slightly more complicated to interpret because of several events with an increase in the PF index and relatively high fluctuations in the pressure, but the boundary conditions given for HJ appear to be set at an appropriate level, as the algorithm succeed to identify what appears to be the most likely event of HJ, by visual inspection of the data.

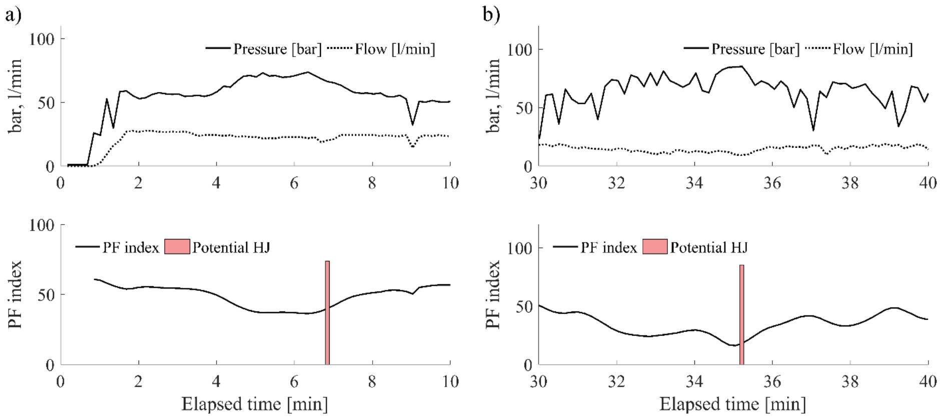


Figure 12: a) and b) show pressure and flow over time with the accompanying PF index beneath. The bar on the PF index is marking the onset of HJ (Strømsvik et al., 2018).

### *3.1.3 General comments on the developed algorithm*

A major challenge during the interpretation of data from the grouting rigs was low sampling rate in combination with oscillations in pressure and flow caused by the piston/plunger pump. The sampling rate was approximately every $10^{th}$ second on some grouting rigs, whilst on other grouting rigs the sampling rate was at every pump stroke on one of the grouting lines. Both infrequent and irregular sampling causes aliasing, and the behaviour of the flow and pressure between each sampling is therefore unknown. Handling data with aliasing is complex and can lead to misinterpretation; data from grouting rigs with a sampling frequency lower than every $10^{th}$ second therefore was decided to be eliminated from the study.

If the method is to be used for other sampling frequencies, it must be carefully adapted. As discussed in the description of the method development, there are several factors that need to be considered, such as adjustments made by the operator, fluctuations caused by the pumping cycle on the rig and rheological attributes when the grout is pushed from standstill to flowing. Trials of the algorithm show that in most cases these events are not interpreted to be HJ, but it has not been possible to eliminate all the different varieties of disturbance, that might lead to erroneous interpretation of HJ. The most challenging aspect of this study has been interpreting HJ at relatively low grouting pressures with high flow rate. In such conditions the grout is often filling an open fracture and the pressure does not respond as pronounced to changes in flow, which could result in false positive HJ events.

Despite the discussed constraints, the testing of the method to detect the onset of HJ has been successful in regard to the presented theoretical assumption of when HJ can be assumed, and the method is considered well suited for analysis of HJ occurring in Norwegian tunnelling projects. Currently it is not possible to verify the method directly to provable occurrences of HJ during rock mass grouting, because there are no available methods to register these events. Therefore, the developed algorithm and the PF index is considered primarily to be a theoretical method.

### 3.2 Method for the investigation of grout consumption

Initially 7 tunnels, all excavated with drill and blast were selected for this research, as presented in Table 4 (subchapter 3.1.1). Unfortunately, a large part of the collected data had to be discarded, due to a sampling less frequent than every 10$^{th}$ second in the data logs from some of the grouting rigs, or malfunctions in the logging system. However, the data available represent 3391 grout holes distributed on 91 grouting rounds in 6 tunnels. A summary of the tunnels, which are anonymized in this study, is presented in Table 5. All tunnels were grouted with Portland cement with different degrees of fineness. In this study, standard cement (OPC) is cement with a $d_{95}$=<40 µm and >25 µm with silica as additive. Microfine cement (MFC) is defined as cement with a $d_{95}$<25 µm with plasticizer as additive. The target pressures presented in Table 5 are the chief stop criterion for the grouting of each grout hole. None of the grout rounds included in this study were grouted with both OPC and MFC.

Table 5: Overview of data from the 6 tunnels included in the study.

| | Rounds | Holes | Type cement | Overburden | Geology | Target Pressure |
|---|---|---|---|---|---|---|
| **A** | 31 | 1012 | OPC | 9-45 m | Gneiss with veins of diabase | 60-80 bar |
| **B** | 8 | 332 | MFC | 65-100 m | Gneiss with veins of amphibolite and pegmatite | 80 bar |
| **C** | 12 | 429 | 9 MFC<br>3 OPC | 24-86 m | Gneiss with veins of amphibolite and pegmatite | 60-80 bar |
| **D** | 16 | 581 | 8 MFC<br>8 OPC | 165-200 m | Banded gneiss | 80 bar |
| **E** | 6 | 227 | 1 MFC<br>5 OPC | 183-188 m | Amphibolite | 80 bar |
| **F** | 18 | 810 | OPC | 23-78 m | Monzonite | 80 bar |

After the grouting logs had been screened for HJ and data related to the events were collected, it was chosen to investigate the correlation between HJ during pre-grouting and grout consumption, as published in Paper II. Increased grout consumption is considered to be one of the most adverse aspects of HJ. The available data was found to be well suited for this type of study. The prediction of HJ was also investigated based on these data, but it was found that the available data was not suitable for this type of study. A detailed clarification regarding this matter is presented in chapter 5; Prediction of hydraulic jacking.

To be able to investigate the correlation between HJ and grout consumption it is important to establish the connection between all parameters which affect the grout consumption. It was assumed that the following factors in the available data could have an impact on the grout consumption in one round of pre-grouting:

1. Rock mass classification (in this study; the Q-system).
2. Percentage of holes with no grout take.
3. Percentage of holes with HJ.
4. Maximum grout pressure.
5. Average grout pressure over time.
6. Grouting time.
7. Number of holes in a round.
8. Length of holes.
9. Type of grout.
10. The chief stop criterium: reaching a target pressure.

A pairwise correlation analysis for bullet points 1 through 8 was performed by using Pearson's linear correlation and the statistical software R Studio (Dalgaard, 2002).

For the comparison of grout consumption using OPC as opposed to MFC, histograms showing frequency distributions were produced. This method was chosen because of large difference in sample size. OPC is the most common grout type in Norwegian pre-grouting practice and it was more available data for this type of grout.

In addition to grout consumption per round, the following comparisons between OPC and MFC were performed using histograms:

- Percentage of holes with no grout take.
- Percentage of holes with HJ.
- Grout consumption in non HJ holes.
- Grout consumption in HJ holes.
- Time consumption per metres hole in non HJ holes (including breaks).
- Time consumption per metres hole in HJ holes (including breaks).
- Percentage of increase in grout consumption in HJ versus non HJ holes.
- Percentage of increase in usage of time in HJ versus non HJ holes.

### 3.3 Results of the investigation of grout consumption

The data logs from the grouting rigs were screened for HJ, using the algorithm published in Paper I. The pie charts presented in Figure 13 show percentage of grouted holes where HJ was indicated. The dark fields marked “No grout” show the percentage of holes where the grout consumption was less, or similar to the volume of the grout hole. These initial results suggest that it is a high degree of HJ during rock mass grouting in Norwegian tunnelling projects. It also shows that a high percentage of the drilled holes are not groutable.

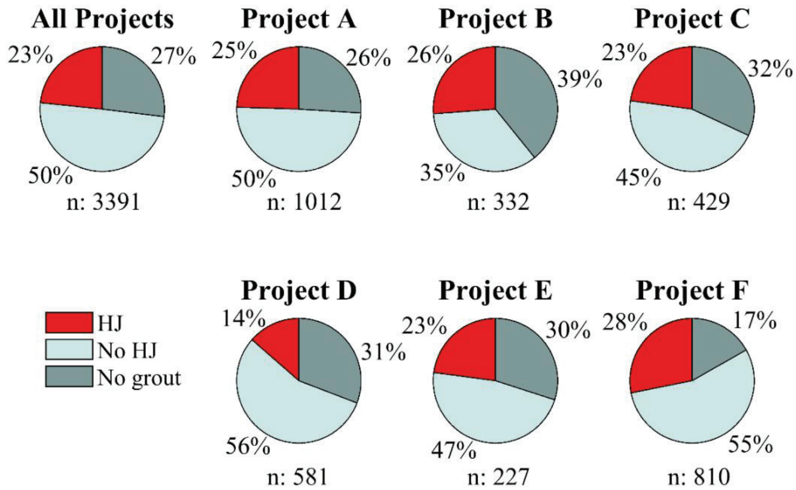


Figure 13: Pie charts showing percentage of grouted holes where HJ was indicated. The dark fields marked “No grout” represents the percentage of holes where the grout consumption was less, or similar to the volume of the drill hole (Strømsvik, 2019).

In 88% of the grouted rounds the $J_w$-value in the Q-system was reported to be 1, which according to NGI (2013) represents dry excavations or minor inflow. For 10% of the rounds the $J_w$ was reported to be 0.66, which represents medium inflow and for 2 % the $J_w$ was reported to be 0.5, which represents jet inflow in competent rock. The rock mass classification in the tunnel was performed after blasting of the grouted area, and the results in general indicates successful grouting, regarding reduction of water ingress.

Figure 14 shows data from 2365 holes grouted with OPC, where the holes are sorted according to grout take. Holes with the highest grout take on the left side, with descending grout take towards right. The blue dotted line represents accumulated grout volume and the orange line represents percentage of holes where HJ is indicated. Approximately 10% of the holes contributes to 43% of the grout consumption and about 51% of these holes have a pressure and flow behaviour that indicate HJ during grouting. The plot indicates a correlation between grout consumption and HJ.

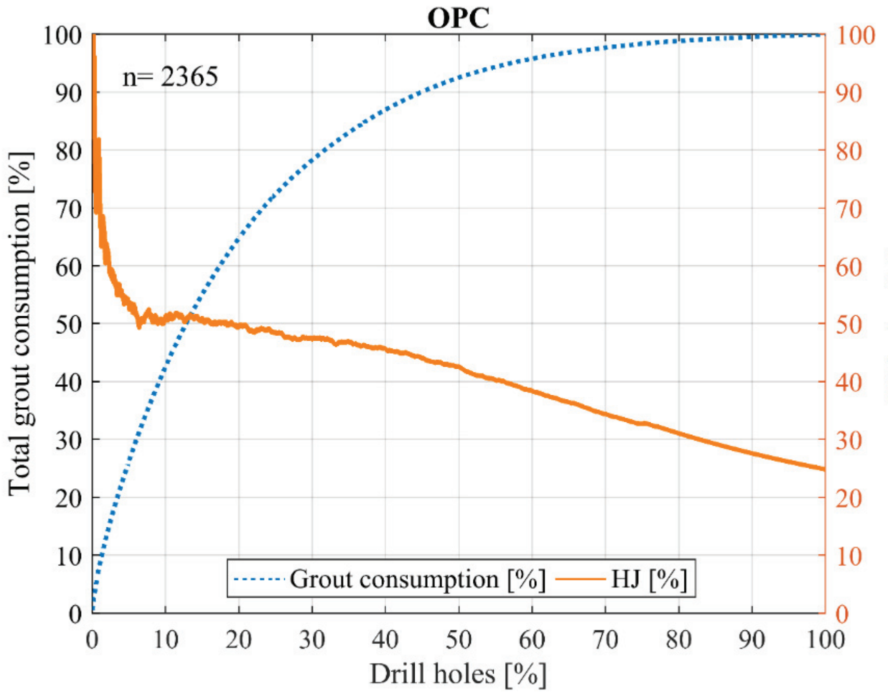


Figure 14: Accumulation of grout consumption and percentage of holes where HJ is indicated, for holes grouted with OPC (Strømsvik, 2019).

Figure 15 shows the same data projection as described above, for 1023 holes grouted with MFC. In this selection of data approximately 10% of the holes contributes to 57% of the grout consumption and it is indicated HJ in about 75% of these holes. Less holes account for a higher part of the total grout consumption with MFC, compared with OPC. It can also be observed that the correlation between HJ and grout consumption is considerable stronger for MFC than OPC.

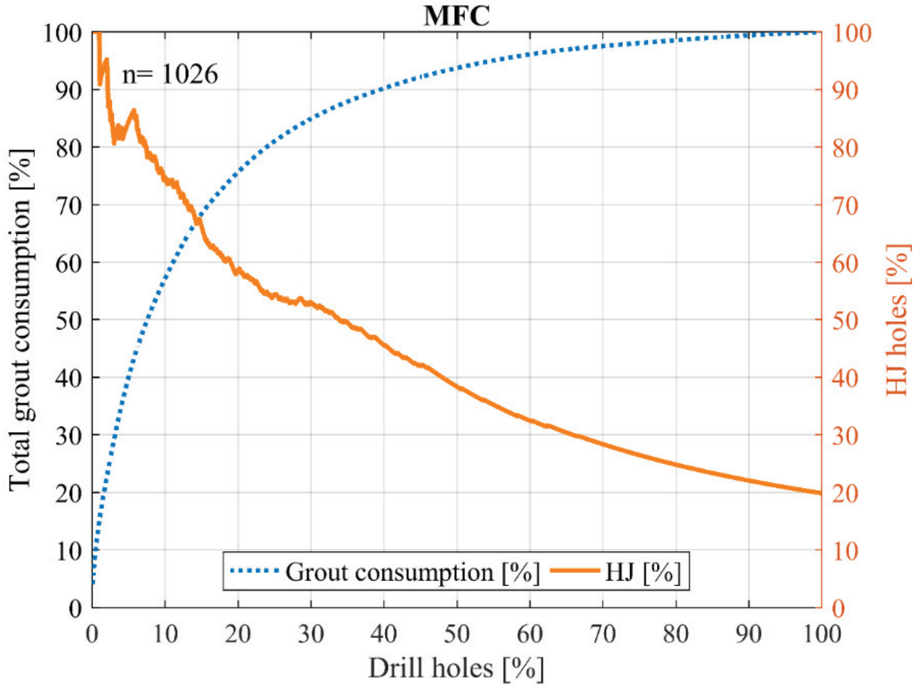


Figure 15: Accumulation of grout consumption and percentage of holes where HJ is indicated, for holes grouted with MFC (Strømsvik, 2019).

A correlation analysis was performed on the data from the geological mapping performed in the tunnel using the Q-system and grout consumption per round. The data regarding rock mass classification was considered not to be optimal for this study, because the classification was performed in pre-grouted rock mass. The data was still considered as useful and the correlation analysis was therefore conducted. The results from this analysis gave no clear correlation with grouted volume and Q-value, or any of the input parameters in this study. In this regard, it is important to mention that the rock mass classification was not performed for the purpose of this study. If the classification using the Q-system had been performed in grout holes prior to grouting, these results would most likely be different. This is issue is described in Paper II. The grout consumption in regard to rock mass properties was therefore not covered by this study.

A correlation analysis was also conducted with the following parameters per round of pre-grouting:

- Grouted volume.
- Percentage of HJ holes.
- Percentage of holes with no grout take.
- Average of the max pressure logged in the holes.
- Average of the average pressure logged in the holes.
- Average of the grouting time in the holes.

In general, statistically significant correlations were found between grouted volume and all the tested parameters. The best correlation was found between grouted volume and average grouting time of the holes and percentage of HJ holes, which also represent the only positive correlations. When grouting with MFC, the correlation with percentage of HJ holes was very strong. The full analysis is presented and discussed in Paper II.

Nine frequency histograms were produced with comparisons between OPC and MFC, published in Paper II. The medians of the results from the histograms are presented in Table 6. Holes with no grout take are not included in the category “holes with no HJ”.

Table 6: Median values from comparisons between OPC and MFC.

| | | OPC | MFC |
|---|---|---|---|
| 1 | Grout consumption per round | 26 000 litres | 13 000 litres |
| 2 | Holes with no grout take | 21% | 35% |
| 3 | HJ holes | 24% | 19% |
| 4 | Grout consumption per metre of drill hole, no HJ | 29 l/m | 17 l/m |
| 5 | Grout consumption per metre of drill hole, with HJ | 52 l/m | 41 l/m |
| 6 | Time usage per metre of drill hole, no HJ | 3.4 min | 2.1 min |
| 7 | Time usage per metre of drill hole, with HJ | 6.4 min | 4.7 min |
| 8 | Increase in grout consumption per round in HJ holes | 68% | 115% |
| 9 | Increase in time per round, in holes where HJ is detected | 79% | 90% |

### 3.4 Discussion of results

The study shows that a small share of the grout holes in a round of pre-grouting accounts for a large part of the grout consumption and a high prevalence of HJ has been found in holes with high grout consumption, particularly for holes grouted with MFC.

The positive correlation between grout consumption and usage of time and negative correlation with high grouting pressure in the correlation analysis is explained by the stop criteria, which is based on reaching a target pressure accompanied by a reduction in flow. If the resistance to grout take was high, the grouting pressure increased fast and the grouting was terminated after a short time of grouting.

Generally, less grout take was found by using MFC, compared to OPC. The resistance to MFC grout seems to be higher, since the target pressure in general was reached in a shorter time, resulting in less grout consumption. There were also more grout holes with no grout take using MFC, than with OPC. It is reasonable to expect that cement with high degree of fineness would be able to penetrate finer fractures and fill a larger portion of the void space in the fractures, resulting in higher consumption of grout and a slower pressure build-up. This appears not to be the case in this study. According to the theory presented in subchapter 2.3.4 there could be several reasons for this result;

1. Poorer penetrability of fine grained grout. Since the MFC grouts used in study had a $d_{95}$ close to 20 μm, it is not expected that this is the cause of the reduced grout consumption for MFC, but it could be the explanation for more holes with no grout take for MFC.
2. Higher yield stress and plastic viscosity in fine grained cements: this could be a contributing factor, since higher frictional resistance within the grout would require higher pressure to move the grout. This leads to a more rapid pressure build-up and the target pressure is reached faster, resulting in less grout consumption.
3. Shorter setting time for MFC. This would change the yield stress and plastic viscosity of the grout and furthermore affect the penetrability and the pressure build-up.
4. If the setting of MFC starts as early as 30 minutes after mixing it is also plausible that the grout in some cases is stored too long in the agitators on the grouting rig prior to pumping, resulting in decreased penetrability, higher yield stress and higher plastic viscosity

One of the most notable findings is the large difference between OPC and MFC regarding grout and time consumption in non HJ holes versus HJ holes. When using MFC, there is a considerable larger relative increase in both grout consumption and usage of time. Also, the increase in grout consumption for MFC is considerable higher than the increase in grouting time, whereas for OPC the increase in grout consumption for HJ holes is lower than the increase in grouting time. This indicates that the flow rate is increased in holes grouted with MFC, where HJ is detected. The reason for this is not known but could be related to increased pressure build up in MFC.

Despite the finding of a strong connection between HJ and grout consumption, this study has not been able to decipher to what extent HJ contributes to the increase in grout consumption. High prevalence of HJ in holes with high grout consumption does not evidence that HJ is the only cause of high grout consumption. As discussed in subchapter 2.3.6, HJ is assumed to be more prone to happen if the grouted volume is large. At the same time, there is little doubt that HJ increases the fracture volume, which also leads to an increase in grout consumption. To reveal to what degree HJ contributes to an increase in grout consumption it would be beneficial to study grouting with HJ of fractures in single drill holes in ungrouted rock mass. This type of study would require a test site, not a study of data from pre-grouting during tunnel construction.

The question regarding whether HJ should be avoided or not is not clearly answered by the results in this study, but some justified assumptions can be deducted. As discussed in section 2.3 and 2.4 in Paper II, rock fractures have a wide variety in characteristics, which result in diverse flow patterns and different grades of cement infilling. In fractures where asperities are isolated elements there is no need to use HJ as a tool to grout the narrow zones close to the asperities because these zones will not transport water due to the lack of connectivity. In fractures with high connectivity between the asperities, HJ could be beneficial to reduce the potential pathways for water. HJ of fractures during grouting increases the fracture volume, which can result in excessive grout consumption. Also, HJ of fractures during pre-grouting is irreversible due to the particles in the grout, which could lead to an increase in conductivity along the perimetry of the grout spread. By this line of reasoning it can be concluded that in most cases HJ does not appear to be necessary to achieve the designated inflow requirements.

### 3.5 Concluding remarks on the study of hydraulic jacking

Recognising HJ during rock mass grouting and the making of an algorithm to perform computerized detection of HJ in grouting logs proved to be a challenging task, because of low sampling frequencies in combination with piston/plunger pumps and unknown geometry of the fracture system. The study concluded that the best way of detecting events with HJ is focusing on increasing or stable flow while the pressure is decreasing, alternatively a decreasing or stable pressure while the flow is increasing. The method to detect the onset of HJ has been successful in regard to the presented theoretical assumption of when HJ can be assumed and the method was well suited for a large-scale analysis of HJ events occurring in Norwegian tunnelling projects.

During the development of the method a new parameter was created, called the PF index. The index is a dimensionless number, representing the relation between grouting pressure and flow rate during grouting. The index was found to be useful when screening for HJ, general interpretation of the grouting progress and could work as a target value in the stop criteria, if the grouting criteria is based on reaching a pre-determined pressure at a pre-determined flow rate. The use of the PF index in general interpretation of grouting during pre-grouting is discussed in chapter 6.

The study concludes that grouting a hole after HJ has occurred does not tend to be a good choice, since it is correlated to longer grouting time and higher grout consumption. This particularly applies when using MFC. The only reason for deliberately HJ of fractures during pre-grouting, should be when needed to achieve the designated inflow requirements, for example if there is high degree of connectivity between asperities in the fractures.

The main goal of pre-grouting in Norway is to reduce the water ingress, according to an allowable inflow rate, “tight enough for its purpose”. This should be performed in a cost- and time-effective way, with least possible impact on the local and global environment. The most solid arguments for avoiding HJ of fractures are related to its strong correlation to high consumption of grout and increased grouting time, which both affect project costs. In addition, high grout consumption has a significant negative effect on global carbon footprint.

HJ or not should be evaluated for each individual project and the project owners should be aware that HJ could lead to unnecessary usage of cement and time, especially when using MFC. HJ during pre-grouting in Norwegian projects ought to be closer monitored in the future, and more research related to this issue should be performed.

# 4 Investigation and assessment of pre-grouted rock mass (paper III)

This chapter gives a summary of the study presented in Paper III, where pre-grouted rock mass is investigated to document grout spread in fractures and transmissivity of the rock mass in the close surrounding to the tunnel. The following subchapters give a short description of the data acquisition and chosen methods for data interpretation, followed by a summary and discussion of the results. As described in chapter 1.2, due to practical reasons related to costs, time and practicability, this study differs from the research presented earlier in this PhD. On the other hand, the study gives a broader perspective of the investigation of the performance of high pressure grouting.

## 4.1 Research methodology

The initial motive for the investigation and assessment of pre-grouted rock mass was to document the grout spread in fractures regarding fracture apertures and type of grout, compared to what is expected for the grout from laboratory testing and to document the tightness of the rock mass surrounding the built tunnel.

It was found too expensive and demanding to establish a test site and the best way for conducting the field tests for this study was therefore to use a suitable built tunnel. As the study needed core drilling, Optical Televiewing (OTV) and water injection tests, the investigated tunnel sections would be blocked for long periods of time, which is problematic in most tunnels both under and after construction. Also, core drilling trough the grouted zone surrounding the built tunnel could impose water ingress into the tunnel, which in most cases would be highly undesirable. The construction site at Åsland have a network of tunnels excavated with D & B in connection with The Follo Line project. Bane NOR generously offered to provide areas at Åsland and fund the tests needed for the study. Three localities at Åsland were selected for this study, named Ch. 171, Ch. 129 and Ch. 21. Figure 16 gives an overview of the construction site and the position of the selected localities for testing.

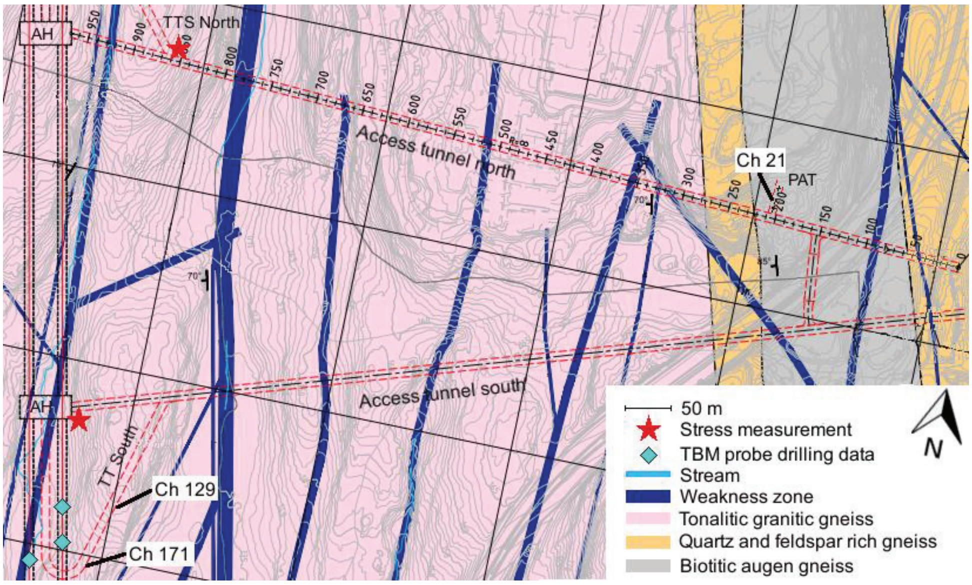


Figure 16: Overview of the construction site at Åsland showing geology, weakness zones, position of the test localities, stress measurements and TBM probe drilling data (modified from FPS AS (2014)).

The flow chart in Figure 17 shows which tests were performed and in what order they were performed. Detailed descriptions of all the performed tests are presented in Paper III.

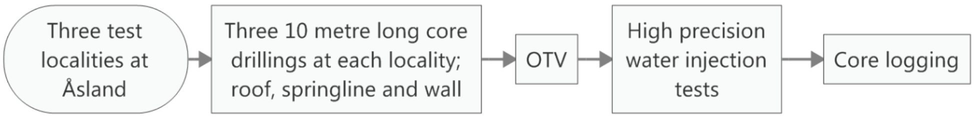


Figure 17: Flow chart illustrating the tests performed at each test location and in which order they were performed.

Figure 18 shows an example of the hole positioning at Ch. 171. All test holes were approximately perpendicular to the tunnel alignment.

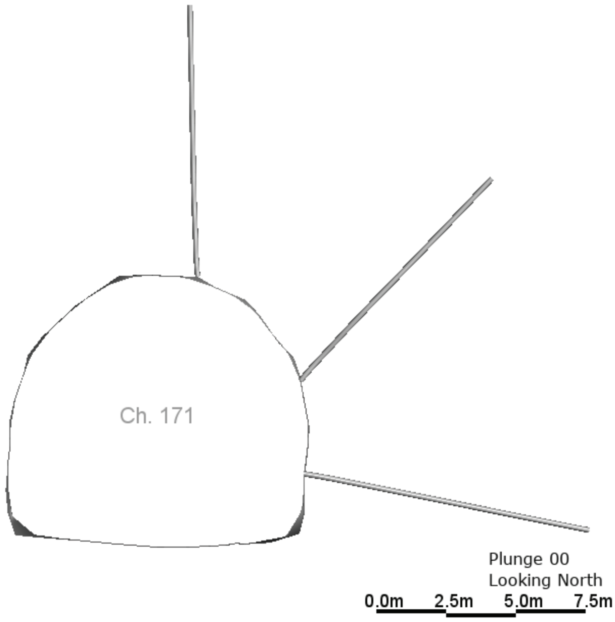


Figure 18: Positioning of test holes, example from the 3D model from Ch. 171 (Paper III).

The data retrieved from the OTV, water injection tests and core logging were implemented into a 3D model constructed in Leapfrog Works 2.2. The purpose of the model was to get a visual understanding of the test site and, combine findings from fractures, geology and water injection tests.

With the large construction activity in the area, the PhD-research had the advantage of investigations and tests performed in relation with the Follo line project. The following data was retrieved from the Follo line project:

- Stress measurements conducted by SINTEF.
- Grouting data from the test localities.
- Fracture distribution from probe drillings ahead of TBMs.
- Fracture distributions from probe drilling of well holes.
- Area maps with weakness zones and geology.

The localities for stress measurements and probe drillings ahead of TBMs are market on the map in Figure 16. More details are given in Paper III.

## 4.2 Results and discussion

### *4.2.1 Fracture distribution and grout spread*

A total of 103 fractures were identified in the test holes; 40 at Ch.171, 24 at Ch. 129 and 39 at Ch. 21. A total of 20 fractures were cemented (19%). Four of the cemented fractures had an approximate aperture of 1 mm, no smaller cemented fractures were found. Trace of cement was found in 6 fractures (6%), where four of the fractures with trace of cement were smaller than 1 mm, one fracture had an approximate aperture of 1 mm and one fracture had an approximate aperture of 2 mm. Table 7 shows percentage of grouted fractures at each location in regard to distance from the tunnel profile and type of cement.

Table 7: Percentage of grouted fractures at each location in regard to distance from the tunnel profile and type of cement.

| Test location | Cemented fractures | | Cement |
|---|---|---|---|
| | 0-5 metres | 5-10 metres | |
| Ch. 171 | 36% | 22% | MFC ($D_{95}$ < 25 µm) |
| Ch. 129 | 21% | 20% | MFC ($D_{95}$ < 25 µm) |
| Ch. 21 | 12% | 0% | OPC (25 µm < $D_{95}$ <40 µm) |

According to Stille (2015) one would expect that, with the types of cement used at the test locations in this study, smaller fractures than 1 mm, but larger than 0.157 mm, would easily be filled with cement. However, no fractures under 1 mm filled with cement were found.

Figure 19 shows an example of a cemented subhorizontal fracture found in the drill core in the roof hole at Ch. 171. It can be noticed that there are three layers of cement with different colour. The reason for this layering is unknown, but it could be due to several reasons; such as HJ of the fracture, pauses during grouting of a hole, or that the fracture is intersected by more than one grout hole.

Figure 19: Picture of the drill core showing a subhorizontal cemented fracture with an approximate aperture of 8 mm. The cement appears to have 3 layers (Paper III).

During the analysis of the result it was realised that the major joint set in the Åsland area was oriented with a dip close to vertical and strike direction of E-W which is approximately parallel with the direction of all the drilled test holes in this study. This implies that most likely there are fractures present at the investigated localities that are not represented by this study. The main reason for the orientation of the test holes in the study was an initial objective to reach 10 metres outside the grouted zone and it was most practical to drill perpendicular to the tunnel profile. The way of intersecting the major joint set would have been to drill holes parallel to the tunnel alignment, which was however not feasible for this study. In retrospective the test holes in the springline and wall should have been drilled with opposite angles from the tunnel profile, to better represent fractures with different orientations. Paper III contains stereoplots showing the orientation of the fractures intersected by the drill holes as well as for the general fracturing in the Åsland area.

#### *4.2.2 Transmissivity in the rock mass surrounding the tunnel*

In all test holes high precision water injection in test sections of 0.5 metres was performed. In total, 149 water injection tests were conducted. The methodology and execution of the tests and calculation of hydraulic transmissivity are presented in detail in paper III.

No measurable hydraulic transmissivity in any fractures filled with grout was found, which confirms that hardened cement grout is an effective barrier for waterflow in fractures. All sections with measurable transmissivity had fractures with no cement filling, or only trace of cement (termed open fractures). To illustrate these results, an excerpt from the 3D model of the roof hole at Ch. 171 is shown in Figure 20. It can be observed that all the sections with measurable transmissivity are directly linked to open fractures. One of the cemented fractures is positioned in a test section with measurable hydraulic conductivity, but an open fracture with trace of cement is also present in this section. The same open fracture is present in an adjacent water injection test, which had the same hydraulic conductivity, proving that the cemented fracture had no measurable hydraulic transmissivity.

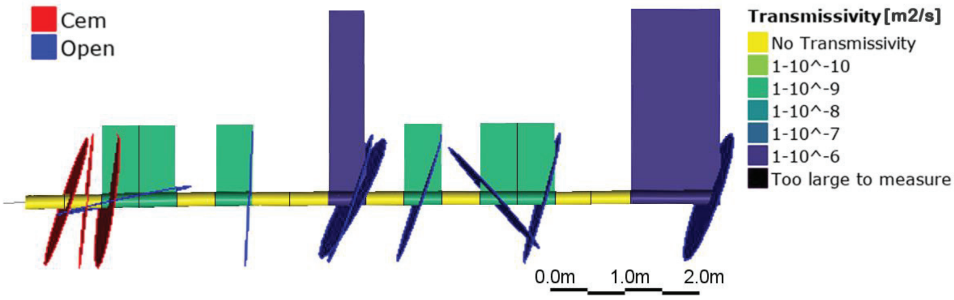


Figure 20: 3D model of the roof hole at Ch. 171, with sections of water injection tests and fractures. The bars represent measured hydraulic transmissivity in log scale (Paper III).

Figure 21 through Figure 23 show excerpts from the 3D model, illustrating the transmissivities along the test holes. Figure 21 represents the test location at Ch. 171, Figure 22 the test location at Ch. 129 and Figure 23 the test location at Ch. 21. In these illustrations the fractures are excluded due to the readability of the figures.

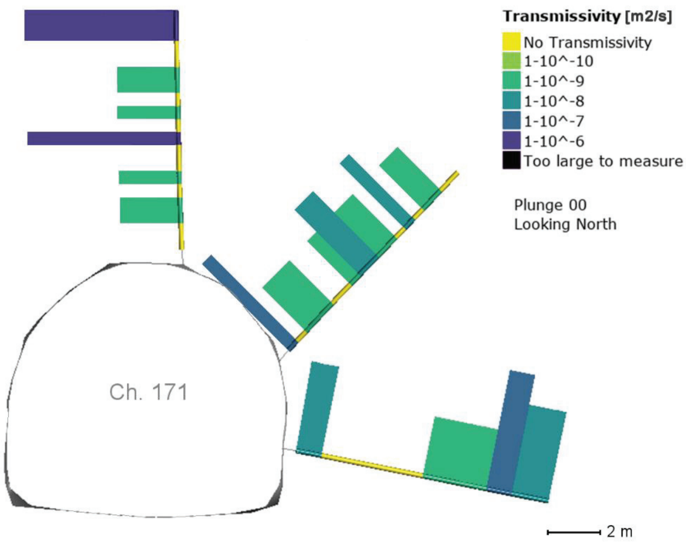


Figure 21: 3D model showing tunnel profile, hole positioning and hydraulic transmissivity at Ch. 171. The bar plots along the test holes are in log scale (Paper III).

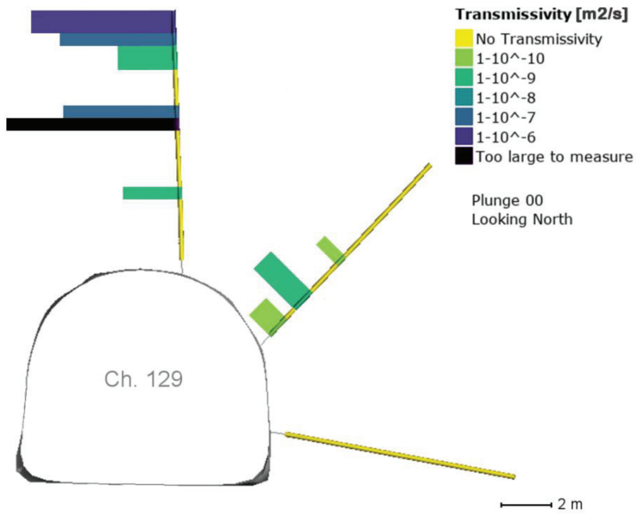


Figure 22: 3D model showing tunnel profile, hole positioning and hydraulic transmissivity at Ch. 129. The bar plots along the test holes are in log scale (Paper III).

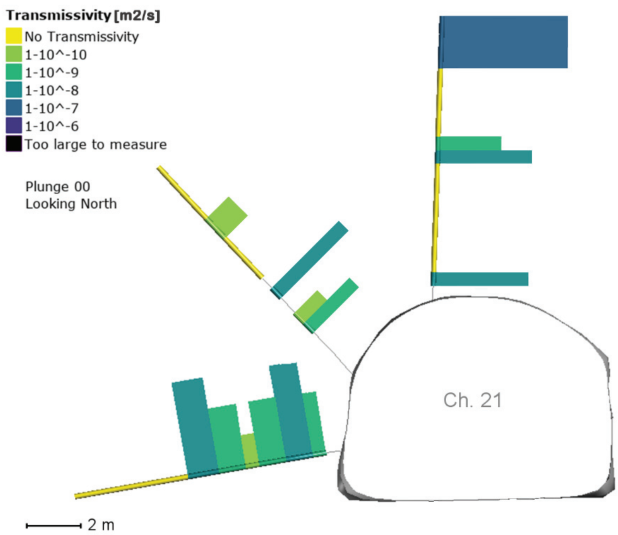


Figure 23: 3D model showing tunnel profile, hole positioning and hydraulic transmissivity at Ch. 21. The bar plots along the test holes are in log scale (Paper III).

It can be observed that in all the locations the fractures with highest transmissivity are found in the mid to deeper parts of the test holes placed in the roof. The reason for this is that the drill holes in the roof have intersected large ungrouted subhorizontal fractures, which were not within reach of the pre-grouting. At all three locations there is approximately 5 metres between subhorizontal fractures with high transmissivity. Grouted subhorizontal fractures were found close to the tunnel profile, which were assumed to be the major contributor to the large consumption of grout in the pre-grouting performed close to the test site. The grout spread in these fractures ensured that the grouting was successful in regard to sufficient reduction of the water ingress into the tunnel, despite the limited spread of grout in smaller fractures close to the tunnel profile.

#### *4.2.3 Hydraulic jacking*

Indication of HJ was found 18 times during the pre-grouting performed in areas overlapping Ch. 171 and Ch. 129. One example of such an event is shown in Figure 24, with pressure and flow behaviour that indicates HJ in a grout hole. It can be observed that there is a significant decrease in pressure and an increase in flow, as a result of the increased aperture of the fracture. It was not possible to screen for HJ at Ch. 21, due to lack of data.

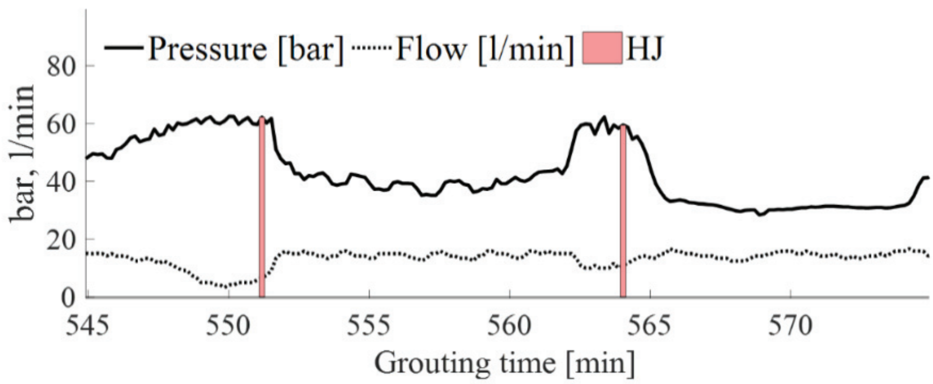


Figure 24: Example of pressure and flow behaviour that indicates HJ in a grout hole overlapping the test site at Ch. 171 (Paper III).

The subhorizontal fractures present at the test localities give an increased risk of HJ during grouting, especially since the minor principal stress in the area was found to be close to parallel with the direction of the overburden pressure. The minor principal stress in the area was approximately 10 MPa. Additionally, the major principal stress was measured to be 24 MPa, oriented N169º with a dip of 3º, which also favours the opening of subhorizontal fractures. The HJ started at 3.5 MPa and was mainly occurring at 4.0 to 4.5 MPa. This indicates that the HJ occurred at a grouting pressure approximately one third of the pressure in the direction of the overburden pressure. HJ at 1/3 of the pressure of the minor principal stress is a surprising result, since HJ of a fracture theoretically only can occur at a pressure similar or higher than the pressure acting perpendicular to the fracture surface. The reason for HJ at a significantly lower grouting pressure than the minor principal stress is not known, it can only be speculated if the high anisotropy in the stress might inflict, or the in-situ stress at the test locations are significantly different from the locations where the stress measurements are performed.

In this case the consequences of HJ can be critically discussed for several reasons:

1. The orientation of the most groutable fractures were favourable for uplift of the overburden.
2. The grout consumption is increased due to increased volume of the large fractures and grout travelling fast in fractures subjected to HJ.
3. HJ of large fractures could have resulted in decrease in the aperture of smaller fractures, reducing the penetrability in these fractures during grouting.

Grout consumption could have been reduced by using a different grouting strategy after the occurrence of HJ, without negative effect on the final results in regard to water inflow. This is further discussed in chapter 6.

#### *4.2.4 General remarks on grout consumption*

In the grout round at chainage 155 a total of 103 tonnes of cement was grouted, which corresponds to 116 097 litres of grout. The grouting works lasted for 64 hours. Assuming that the grout spread in large subhorizontal fractures, it can be speculated if this large amount of grout was necessary to achieve the required tightness around the tunnel profile. Presuming that three different subhorizontal fractures were intersected by the grouting holes and the fractures were smooth with a large average groutable aperture of 3 mm, this would give a disk shape distribution of the grout, with a spread radius of 55 metres in each of the 3 fractures. This is equivalent to an area of 1.8 soccer fields in each fracture. In real life it is possible that the grout spread would be even larger due to channelling and anisotropic spread of the grout. This suggests that the grouting performed in this area might have been excessive and it is likely that the tunnel would be tight enough with less grout.

In many cases it seems like the philosophy during rock mass grouting when tunnelling in sensitive areas is “better to be safe than sorry”. The difficulties and costs with performing post-grouting, or risking damage of surface structures due to drawdown of the groundwater table in many cases in excessive use of grout and time. With the available information during rock mass grouting in todays practice, this absolutely safe mentality is understandable. For the future, it would be beneficial to have a better understanding of hydrogeology, fracture distribution and stress condition in the rock mass before taking qualified decisions on-site regarding when the grouting should stop.

#### *4.2.5 Fracture distribution, roughness and transmissivity in different rock types*

In addition to the evaluation of grout spread in fractures and the transmissivity of water in the rock mass surrounding the tunnel it was found that the data was suitable for evaluation of transmissivity, the degree of fracturing and fracture roughness based on rock type. This is not directly related to the study of high pressure pre-grouting of rock mass, but relevant in regard to the prediction of water ingress in to tunnels, which is related to the issue of when pre-grouting is expected to be necessary. To achieve full exploration of the retrieved data in this project it was chosen to also analyse the data in regard to this matter.

The following rock types were encountered in the test holes: tonalitic gneiss (TTG), granitic gneiss (GG), supracrustal gneiss (SCG), amphibolite (A), garnet amphibolite (GA), pegmatite (PG) and poor pegmatite (PP). The term poor pegmatite was used because the upper part of the roof hole at Ch. 21 intersected a weakness zone and the pegmatite in this hole section was of poor quality and different from the other type of pegmatite encountered in the study.

The fracture roughness was evaluated using the joint roughness coefficient (JRC), which is an empirical index used for surface roughness characterisation. JRC was estimated for all the fractures during the core logging by using a contour gauge and a table for typical roughness profiles, published by Barton and Choubey (1977).

The median JRC for all the represented rock types is shown in Figure 25. Some of the fractures were in the transition (Tr) between two rock types. In general, high JRC values were found in tonalitic gneiss, granitic gneiss, pegmatite and garnet amphibolite and lower JRC values in amphibolite, supracrustal gneiss and fractures in the transition zones between two rock types. Most of the fractures found in the transition zones were between amphibolite and other rock types.

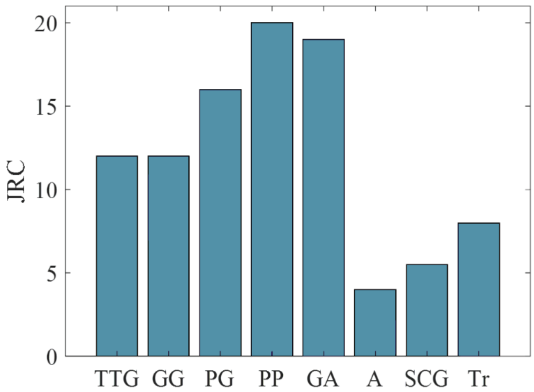


Figure 25: Median of JRC for rock types (Paper III).

The average grain size of the rock types was measured to be as following: granitic gneiss 4 mm, tonalitic gneiss: 3.5 mm, pegmatite: 10 mm, poor pegmatite: 15 mm, garnet amphibolite 10 mm (garnet)/0.3 mm (amphibolite), supracrustal gneiss: 0.5 mm and amphibolite: 0.3 mm.

These results indicate that fracture surfaces in rock types with coarse mineral grains are rougher than fracture surfaces on rock types with fine mineral grains.

30 out of 103 fractures found in the test holes were single fractures placed within one section of water injection with measurable transmissivity and for these 30 fractures the hydraulic aperture was calculated. The hydraulic aperture, i.e. the aperture of a fracture that would give the same mean flow as the actual aperture, was calculated according to a method presented in Paper III. It was also performed a pairwise correlation analysis between calculated hydraulic aperture and JRC, as described in the paper.

The results of the analysis are summarized in Figure 26. The correlation coefficient is relatively low, and there is no statistically significant correlation between hydraulic aperture and JRC. In this regard it is important to keep in mind that the surrounding rock mass is pre-grouted, and the transmissivity is most likely affected by this. By looking at the corresponding scatter plot, it can be observed that there is heteroscedasticity in the data. This means that the scatter has more spread in one end of the scale. When the JRC is low, which represents smoother fractures, the hydraulic apertures are generally in the smaller end of the scale. With increasing JRC the hydraulic apertures are in both ends of the scale, including both small and large hydraulic apertures. This effect is the cause of no statistical correlation, but it can still be concluded that there is a tendency towards smaller hydraulic apertures with low JRC.

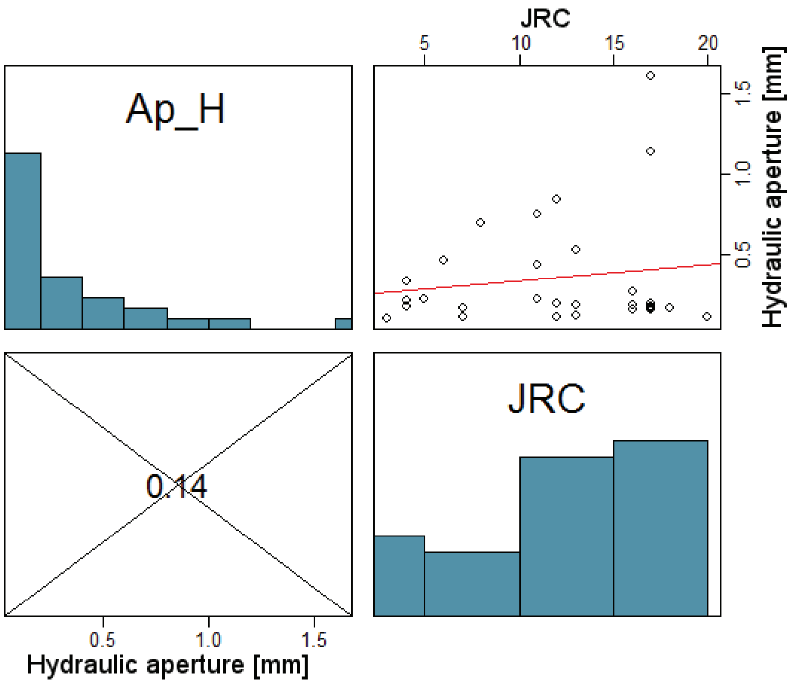


Figure 26: Results from the pairwise correlation analysis between hydraulic aperture (Ap_H) and JRC. The upper right shows scatterplot of the values and the lower left shows the correlation coefficient, where the cross indicates that there is no statistical correlation. The bar plots show the distribution of the two parameters (Paper III).

Figure 27 shows the distribution of rock types encountered in the drill holes at each test location, the distribution of fractures in each of these rock types and the distribution of calculated hydraulic apertures in each of the rock types.

At Ch. 171 the pie chart shows that the distribution of fractures between the rock types are roughly even, but there are relatively less fractures in the granitic gneiss, although the fractures in granitic gneiss have a larger hydraulic aperture than the fractures in the other rock types. Supracrustal gneiss and garnet amphibolite are more fractured than the average, but the present fractures have smaller hydraulic apertures. At this locality the amphibolite has approximately average number of fractures, with average hydraulic apertures.

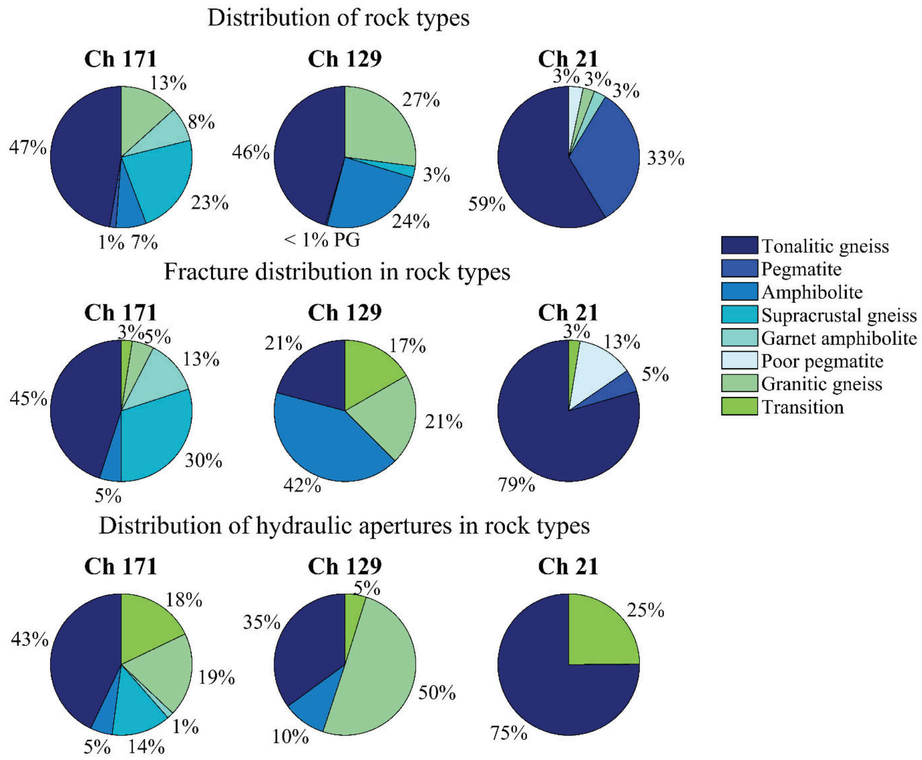


Figure 27: Pie charts showing distribution of rock types at each side, distribution of fractures in each rock type at each test location and distribution of hydraulic apertures in each rock type at each test location (Paper III).

At Ch. 129 the amphibolite is more fractured than the other rock types, but the fractures have considerable smaller hydraulic apertures than average. The granitic gneiss is slightly less fractured than average, but the present fractures have considerable larger hydraulic apertures than average. The tonalitic gneiss has less fractures than average, but the fractures has larger hydraulic apertures. 17% of the fractures are at the transition between two rock types and have smaller hydraulic apertures than average. All the fractures at a transition are between amphibolite and other rock types.

At Ch. 21 it can be observed that tonalitic gneiss and poor pegmatite are more fractured than the other rock types present. The hydraulic apertures in tonalitic gneiss are approximately average for this location. During the high precision water pumping test the transition zone and the poor pegmatite were in the same test section, which means that the fractures present in the transition zone and the poor pegmatite have considerable larger hydraulic apertures than average. The pegmatite at this location has considerably less fractures than average and the fractures present did not have measurable transmissivity.

These results indicate that the fractures in the coarse grained rock types have large or medium hydraulic apertures and low or medium degree of fracturing, while the fine grained rock types have smaller hydraulic apertures but higher degree of fracturing. Garnet amphibolite consists of amphibolite (fine grains) and garnet crystals (coarse grains). The JRC was generally measured to be high, but in this rock type the hydraulic apertures were smaller than average.

One matter which is not discussed in the paper is how the grouting has inflicted on the results. As presented in subchapter 4.2.2, the grouted fractures had no measurable permeability, which would inflict on the results, compared to the same study performed in an ungrouted rock mass. In the results presented in Figure 27, the grouted fractures are presented as fractures in the rock mass, but since these fractures have no measurable transmissivity, the estimated hydraulic apertures are conservative in sections where the grouted fractures are present.

The pie charts shown in Figure 28 presents an overview of how this affects the results. At Ch. 171 there are four cemented fractures in tonalitic gneiss, supracrustal gneiss and in the transition between two types of rock, which mean that the hydraulic apertures for these categories are underestimated. This confirms the earlier findings, indicating that coarse grained rock types generally have large or medium hydraulic apertures, despite low or medium degree of fracturing. At Ch. 129 there are two cemented fractures in tonalitic gneiss, two cemented fractures in amphibolite and one cemented fracture in the transition between two types of rock. The two cemented fractures in amphibolite do not have significant impact on the earlier findings, indicating coarse grained rock types, generally have large or medium hydraulic apertures, despite low or medium degree of fracturing. This is due to the generally high degree of fracturing in amphibolite at this site. At Ch. 21 there are three cemented fractures in tonalitic gneiss, which confirms the finding that coarse grained rocks have large or medium hydraulic apertures, despite low or medium degree of fracturing.

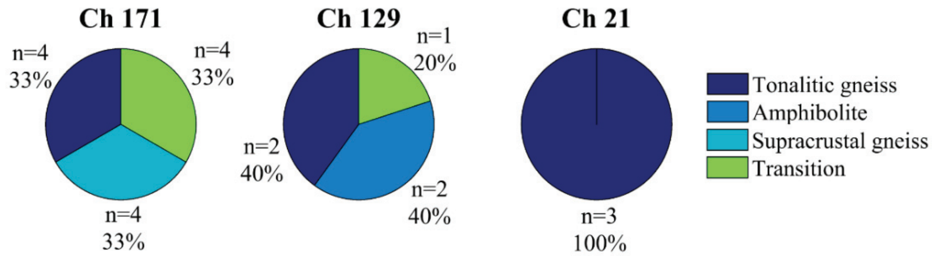


Figure 28: Pie charts showing distribution and number of cemented fractures in different rock types for each test site (Paper III).

The pie chart in Figure 28 will be included in the revised, final version of Paper III.

### 4.3 Concluding remarks

In this study pre-grouted rock mass has been investigated in regard to spread of grout and hydraulic transmissivity. Transmissivity, degree of fracturing and fracture roughness in regard to rock types ere also assessed. The following main conclusions could be drawn;

1. The grout penetration into small fractures was found to be less than expected. Only fractures that had a measured aperture of 1 mm or larger at the drill hole intersection, were found to be fully grouted. From laboratory studies the grout used at the test locations for this study should be able to penetrate fractures down to 0.16 mm. Overall, 20% of the fractures were filled with grout.

2. The grout consumption in the grout rounds at Ch. 171 and Ch. 129 seems to be excessive and it is concluded that the inflow requirements could have been met with less grout consumption. This especially implies to the grouting technique used after the occurrence of hydraulic jacking.

3. It was found a tendency towards smaller hydraulic apertures with low JRC values. With increasing JRC the hydraulic apertures were at both ends of the scale, including both small and large hydraulic apertures.

4. Generally higher JRC values were found in coarse grained rock types, such as granitic gneiss, tonalitic gneiss and pegmatite, and lower JRC values for fine grained rock types, such as amphibolite and supracrustal gneiss.

5. In fine grained rock types, such as amphibolite and supracrustal gneiss, the hydraulic apertures were smaller, even though these rock types were more fractured than average. Granitic gneiss was the rock type that was found to have the largest hydraulic apertures, although granitic gneiss was less fractured than average.

Tonalitic gneiss had relative average degree of both fracturing and hydraulic apertures.

# 5 Prediction of hydraulic jacking

This chapter contains a brief discussion of HJ with regard to overburden and grout consumption and how HJ may be predicted based on these parameters. There is no submitted paper related to this topic.

The data retrieved after screening for HJ, as presented in Paper I, was considered useful also for prediction of HJ. One possible approaches for this type of study is to consider the overburden in relation to the pressure at which HJ occurred. "Rules of thumb" for grouting pressure presented in subchapter 2.3.5, are examples of such approach. The purpose of the "rules of thumb" is to provide guidance for selecting a grouting pressure that gives minimum risk of uplift of the overburden due to HJ of fractures. The other approach is to study the grouted volume for evaluating the risk of HJ. As mentioned in subchapter 2.3.6, Lombardi (1985) suggests that the risk for HJ of fractures at lower grouting pressures increase with increased grouted volume.

## 5.1 Hydraulic jacking pressure compared to overburden

Figure 29 shows the results of data produced by using the algorithm presented in Paper I. The data basis is the tunnelling projects and rounds of grouting presented Table 5, chapter 3.2. Each plot represents the median of the grouting pressure immediately before the onset of HJ at each round of grouting versus the overburden. Data from 91 rounds of pre-grouting included in the plot. It can be observed that at lower overburdens the HJ is occurring at lower grouting pressures, but with increasing overburden the HJ is occurring at a generally higher grouting pressure. It can also be observed that it is some heteroscedasticity in the data, meaning that at one end the scatter has a larger spread. It is tempting to conclude that this is mainly due to HJ at a lower pressure with lower overburden, but this cannot be deducted from the data, because the stop criterion was based on pressure and when the overburden was low the stop criterion was 60 bar.

This figure clearly illustrates the challenges of using overburden as a measure of when HJ can be expected. This might not be the case in other parts of the world. In Norway the general stress distribution in the rock mass is not a result of the overburden only. As described in Paper III, the stress conditions in the rock mass in Fennoscandia is complex, with horizontal stress normally by far exceeding the vertical stress. The origin of the high horizontal stress is presumed to be a combination of ridge push from the Mid-Atlantic Ridge and rapid unloading of the surface due to erosion and deglaciation (the Holocene glacial retreat). The complex composition of factors contributing to the in-situ stress makes it very difficult to predict minor principal stress, without in-situ stress measurement.

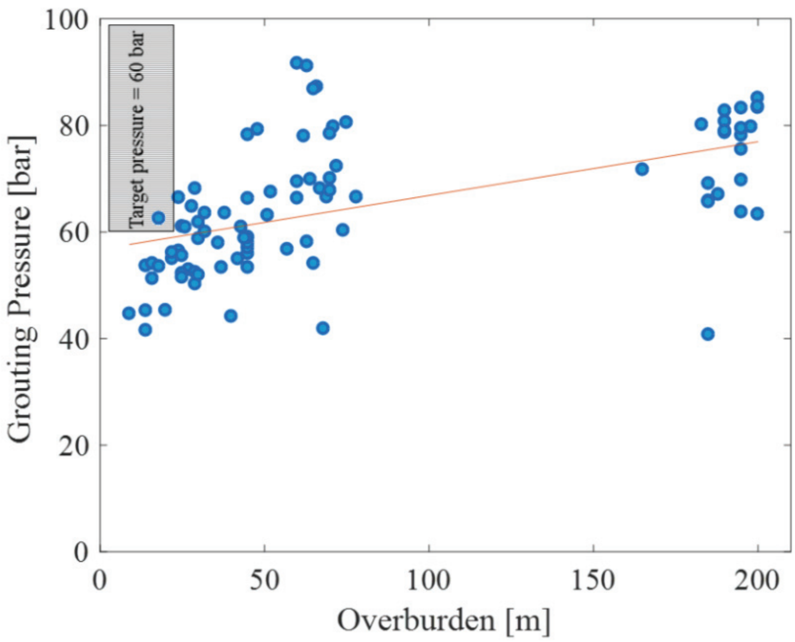


Figure 29: Scatter plot showing the median of the grouting pressures immediately before the onset of HJ at each round of grouting compared to the overburden. Data from 90 rounds of pre-grouting are included in the plot.

Even with measurement of in-situ stresses it can be difficult to predict occurrences of HJ during rock mass grouting. In the study presented in Paper III, HJ occurred at a pressure approximately 1/3 of the minor principal stress measured 120-160 metres away from the location of the pre-grouting. This is not as expected, since HJ of a fracture theoretically can occur only at a pressure similar or higher than the pressure acting perpendicular to the fracture surface. It was concluded that it was not possible to predict the pressure at which HJ is expected to occur based on the data available in this study only.

In this regard it is important to keep in mind that in areas with low overburden the stress is generally lower and more anisotropic, and it is expected that HJ would occur at lower grouting pressures. Additionally, HJ with low overburden has more negative consequences, such as lifting of the surface and higher risk for the grout to reach the ground surface.

## 5.2 Hydraulic jacking pressure compared to grouted volume

Figure 30 shows data produced by using the algorithm presented in Paper I in regard to HJ pressure and grouted volume. The data basis is the tunnelling projects and rounds of grouting presented in Table 5, chapter 3.2. Each marker represents grouted volume and grout pressure immediately before the onset of the first event of HJ in each hole where HJ has been indicated.

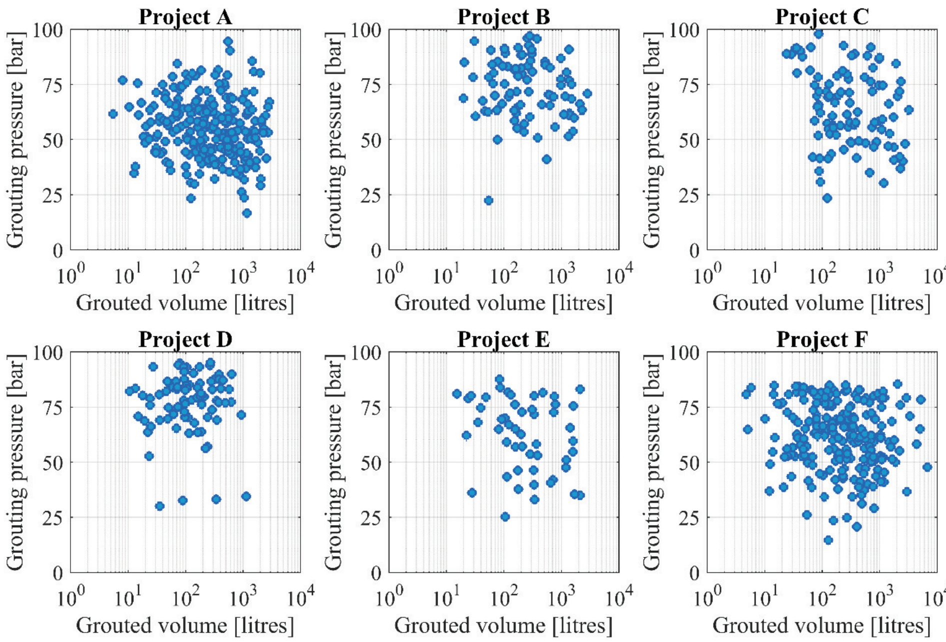


Figure 30: Scatter plots showing grouted volume and grout pressure immediately before the onset of the HJ of the first event of HJ in each hole where HJ has been indicated.

In this data interpretation it has been chosen to present data from each individual tunnel to limit the influence from variables such as regional stress distribution, overburden and geology. By visual inspection of the scatter plots it can be concluded that there is no clear correlation between grouted volume and the pressure at which HJ occurs for any of the 6 tunnels in this study.

## 5.3 Concluding comments on prediction of hydraulic jacking

No distinct trends of correlation between grouting pressure at HJ versus overburden or grout consumption have been revealed. It was therefore decided not to pursue these topics any further in the PhD research.

The results confirm the difficulty of predicting HJ and demonstrates that during high pressure grouting it is not possible to avoid occurrence of HJ. Therefore, the main focus should be on how to handle the grouting when such events occur and to reduce the unnecessary consumption of grout. This could lead to a significant decrease in cement consumption in tunnelling projects, without inflicting on the final result in regard to water inflow.

# 6 PhD findings in regard to the practical work of pre-grouting in Norway and the use of the PF index

This chapter provides some practical recommendations to the tunnelling industry based on the findings of the PhD study. The practical use of the developed PF index for evaluation of the relationship between grouting pressure and flow rate during grouting is also presented.

In general, it can be concluded from this PhD study that high pressure pre-grouting under normal conditions, is a successful method with respect to the reduction of water ingress. On the other hand, a successful grouting project also includes economy and environmental aspects. The findings from the PhD study indicate that the grout consumption in some cases appear to be excessive and needs to be further evaluated. The conclusion of excessive grout consumption is related to the grouting procedure after the occurrence of hydraulic jacking (HJ) of fractures in the rock mass. This chapter describes why HJ in many cases is negative and suggest how the grout consumption can be reduced by changing the grouting strategy after the occurrence of HJ.

## 6.1 Significance and limitations of high grouting pressure and consequences of hydraulic jacking

The main purposes of using high pressure are:

1. Better penetration of grout in fractures with small apertures.
2. Eroding joint-filling and plugs in fractures and hold the fractures open during grouting.
3. Larger spread of grout in shorter time.

The main objective for pre-grouting is to ensure that fractures relatively close to the tunnel profile are grouted. Grouting during HJ of fractures does not coincide with this main objective. There are several reasons for why HJ is undesirable and have negative impact on the grout spread in the rock mass close to the tunnel:

1. In many cases the grouting pressure is drastically reduced after HJ, this results in less penetration of grout into small fractures intersected by the grout hole.
2. The flow rate is often increased during HJ, but the increase in grouted volume is only filling the fracture subjected to HJ.
3. The aperture increase in the jacked fracture could lead to confinement of smaller fractures, resulting in less grout spread in smaller fractures.

In one way one could describe HJ of fractures during pre-grouting with “popping a balloon”. The grouting can go on for a very long time, but the grout spreads far away from the place where it is intended to be. The result is high consumption of grout, long grouting time and which does not contribute significantly to the reduction of water ingress in the area where the pre-grouting is performed.

In the study presented in Paper II, it was indicated HJ in 23% of 3391 analysed grout holes. When grouting with standard cement the grout holes with HJ had 68% larger grout consumption and 79% longer grouting time than holes with no HJ. When grouting with microfine cement the grout holes with HJ had 115% larger grout consumption and 90% longer grouting time than holes with no HJ. Holes with grout take similar or less than the volume of the drill hole are not included in these figures. The increased grout volume cannot be alleged to origin only from the occurrence of HJ, but by considering the physical changes in the fracture geometry in addition to the pressure and flow behaviour of the grouting it is clear that HJ contributes to the increase in grout consumption. Given the frequency of HJ, it is little doubt that there is a large potential for reduction of grout consumption during pre-grouting in Norwegian tunnelling projects.

Some may argue that HJ should be avoided under all circumstances and that the changes in HJ grout holes are unrecoverable. As described in chapter 5, it was found to be very difficult to predict HJ before and during pre-grouting in Norwegian rock mass conditions and it was concluded that when performing high pressure pre-grouting, one has to accept that HJ will occur. What can be controlled is how the grouting is performed after HJ has occurred.

### 6.2 Use of the PF index and grouting procedure to reduce grout consumption after hydraulic jacking

The PF index was created as a tool in computerized screening for HJ, but it was discovered that the index could be useful in other applications as well, considering the chief stop criterion in Norway; reaching a pre-determined grouting pressure, at the same time as the grout flow should be small or close to zero. The PF index prominently illustrates the pressure build-up and could be used to monitor the progress of the pressure build-up during the grouting as well as visual identification of HJ during grouting.

By using the pre-determined target pressure and flow rate, the PF index representative for the stop criterion can be calculated, using Formula 3 presented in subchapter 3.1.2. As an alternative to a stop criterion based on pressure and flow, a stop criterion represented by the calculated PF index could be given. An example of this is illustrated in Figure 31, where the target pressure is set to 80 bar and the flow rate should be 5 l/min, or less. This would give an PF index of 13.5. In Figure 31, this stop criterion is marked in grey. When the flow is high, and the pressure is low the PF index is high. As the pressure increases the PF index decreases, and also if the flow decreases the PF index decrease. When the value of the PF index reaches the grey area, the grouting should be stopped, or kept stable close to the borderline of the grey area a few minutes before stopping.

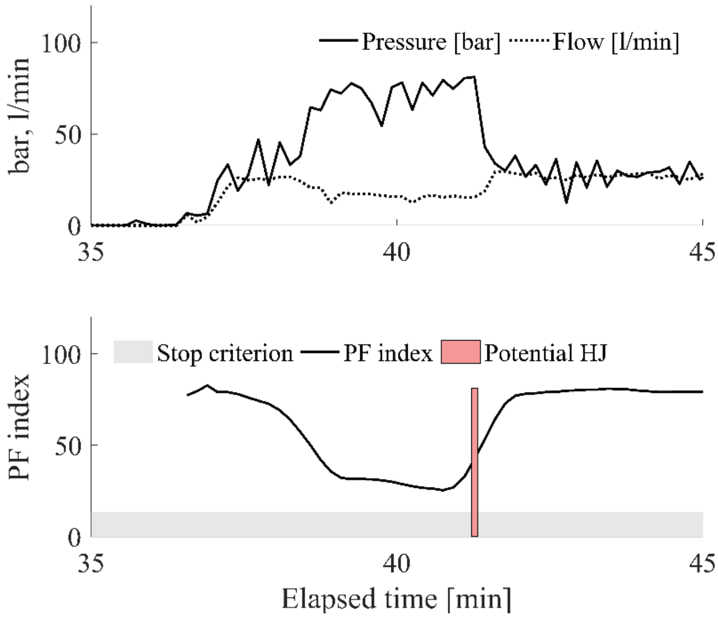


Figure 31: Pressure and flow over time with the accompanying PF index beneath. The bar on the PF index is marking onset of HJ (Strømsvik et al., 2018).

If the PF index has a significant increase during grouting it is a good indication that the grouting of that specific hole is not as expected and a new strategy for the grouting of this hole needs to be planned. If the occurrence of HJ is suspected, the grouting should be paused and resumed shortly after stoppage, allowing the HJ fracture to close. When the grouting is resumed the grouting pressure should be held lower than the pressure at which the HJ occurred and the decision for when the grouting should be stopped should be based on grout consumption. In the example in Figure 31, the grout is “lost” in the HJ fracture, with high grout flow, combined with low pressure, which does not help the penetration of grout into the fractures close to the tunnel. Grouting at a lower pressure without HJ would be more beneficial to the grouting of fractures close to the tunnel. There are three good reasons why the grouting should not be continued during HJ in the given example:

1. Penetration of grout into small fractures need high pressure, but during HJ the pressure is often lost. By pausing the grouting for å short time, allowing the fracture to close, the grouting could be continued with a pressure higher than during HJ, but lower than the pressure used when the HJ occurred.

2. The flow rate is very high, which leads to excessive use of grout. Without HJ the flowrate could be lower, even with slightly higher pressure than the pressure after hydraulic jacking. The increased grout volume resulting from increased flow rate after HJ in most cases is not likely to contribute to the reduction of water ingress.

3. The volume increase in HJ fractures might lead to compression of smaller fractures, resulting in less penetration of grout in smaller fractures close to the tunnel.

The suggested grouting procedure is a result of thorough study and interpretation of grouting logs combined with a literature study, and the procedure is a suggested measure to reduce unnecessary grout consumption.

## 6.3 Final comments regarding grout consumption during pre-grouting

The research in this PhD were performed under what is considered to be typical Norwegian tunnelling conditions. Within the field of tunnelling a great variability of rock mass conditions are encountered, which give rise to a broad spectrum of challenges during grouting. Examples of such conditions are:

1. Tunnels with very high overburden, resulting in high groundwater pressures and high stress in the rock mass.

2. Areas with very fine fractures, which are not groutable but transport water.

3. Rock mass with fractures with a high degree of gauge material.

4. Karst formations, with large cavities.

Large differences in rock mass conditions imply that pre-grouting needs to be adapted to each site and in some cases high consumption of grout and/or HJ is necessary to achieve the required tightness. It is important however to optimise pre-grouting by reducing the grout consumption in situations where excessive grout is used due to HJ.

# 7 Main conclusions of the PhD research

The major findings of the PhD research are summarized in the following bullet points. For further details, reference is made to the subsequent full papers and the thesis.

1. The data log from grouting rigs are valuable for understanding the grouting progress and detecting occurrence of hydraulic jacking of fractures during grouting. To make reliable analyses based on logged data from grouting rigs it is imperative with sufficient logging frequency. The study concludes that logging every $10^{th}$ second or less frequent is inadequate and that a general improvement in logging frequency on grouting rigs would be beneficial for the utilisation of the logged data.

2. It was found that hydraulic jacking is a common occurrence during pre-grouting in Norwegian tunnelling and hydraulic jacking was indicated for 23% of the 3391 analysed grout holes in this study.

3. Both grout and time consumption were found to be considerable higher in grout holes where hydraulic jacking was indicated, especially when grouting with microfine cement, where the increase in grout consumption was 115% and increase in time consumption was 90%.

4. Several distinct differences were found between standard and microfine cement grout. The general grout consumption was doubled when using standard compared to microfine cement. Additionally there were more holes with no grout take in rounds grouted with microfine cement. Hydraulic jacking was found to be appearing more frequently when grouting with standard cement than when grouting with microfine cement. These findings indicate that the understanding of cement behaviour and characteristics is still insufficient and needs to be further investigated.

5. It was found that the grout did not penetrate fractures as small as expected from published laboratory tests, and the spread of grout seemed to be restricted to fractures with relatively large apertures. Despite this, the grout spread in the relatively large fractures ensured that the grouting was successful with regard to sufficient reduction of water ingress into the tunnel, in an area with strict requirements regarding water inflow.

6. In general, coarse grained rock types were found to have greater hydraulic apertures, than fine grained rock types.

7. It was found that high pressure pre-grouting in general is successful for reduction of water ingress into tunnels. In regard to project economy and environmental aspects there is a potential for improvement, particularly regarding unnecessary consumption of grout caused by hydraulic jacking of fractures.

8. Predicting hydraulic jacking in Norwegian rock mass conditions is difficult and during high pressure grouting it is not possible to avoid all occurrence of HJ. The main focus should be on how to handle the grouting when such events occur, to reduce the unnecessary consumption of grout.

The fundamental objective for this PhD was to achieve an increased understanding of rock mass grouting with the use of high grouting pressures and to investigate if the current practice of using high grouting pressure is optimal regarding reduction of inflow into tunnels, in line with good economy and environmental aspects.

The PhD study concludes that high pressure pre-grouting under normal conditions, such as investigated in this PhD research, is a successful method with respect to the reduction of water ingress according to the required demands. On the other hand, a successful grouting project also includes economy and environmental aspects. The findings indicate that the grout consumption in some cases appear to be excessive and needs to be further evaluated. The conclusion of excessive grout consumption is related to the grouting procedure after the occurrence of hydraulic jacking of fractures in the rock mass. Changing the grouting strategy after the occurrence of hydraulic jacking has the potential of significant decrease in cement consumption in tunnelling projects, still maintaining the goal of the grouting efforts.

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# Paper I

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# Development of an algorithm to detect hydraulic jacking in high pressure rock mass grouting and introduction of the PF index

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ABSTRACT

Pre-excavation rock mass grouting is a common procedure for reducing water ingress into tunnels during construction in Norwegian tunnelling. The level of grouting pressure is a disputed subject and the knowledge of how the rock mass responds to the high pressure and how this inflicts on the grouting results is sparse and little investigated. For this reason, it is of interest to use data from high pressure grouting performed in Norwegian projects to investigate these matters. This paper presents the development of a method for identifying hydraulic jacking during rock mass grouting and the making of an algorithm to perform computerized detection of hydraulic jacking in grouting logs. The algorithm is the base for a larger study, where screening for hydraulic jacking is performed in over hundred grouting rounds, distributed on several Norwegian tunnels excavated in rock mass. The relation between grout flow and grouting pressure has shown to be vital for the understanding of the grouting progress and events occurring during the grouting. Interpretation of pressure and flow as two separate variables, which are affected by aliasing, caused by low and irregular sampling frequency is a challenging task, and it was found to be helpful to create a parameter to represent this relationship, named the PF index (Pressure Flow index). This parameter has also shown to be useful in other applications such as monitoring the grouting progress on site.

## 1. Introduction

Pre-excavation rock mass grouting is a common procedure for reducing water ingress into tunnels during construction in Norwegian tunnelling. The methodology used in Norway is developed through many years of practical experience in underground excavations such as road and railroad tunnels in both urban and rural areas, subsea tunnels and hydroelectric power development. As a result of the prior experience and a research project called "Tunnels for the citizens" in the early 2000s, the grouting pressure used is high compared to e.g. US and Swedish practice. Common grouting pressures in Norwegian tunnelling projects are 60–80 bar, largely depending on the overburden and geology. One of the most important motivations for using high grouting pressure, is to ensure penetration of grout into fine fractures. The knowledge of how the rock mass respond to the high pressure and how this inflicts on the grouting results is sparse and little investigated, at the same time the pressure is a disputed subject in Norway and even more in many other countries.

The most controversial effect from grouting with high pressure is hydraulic jacking of fractures existing in the rock mass. Some of the discussed issues are; at which pressure can hydraulic jacking be expected, what are the risks connected to this event, how does it inflict on the sealing effect of the tunnel, how does it affect the usage of time and cost for the tunnelling project? To investigate these matters it is if interest to identify occurrences of hydraulic jacking during rock mass grouting and collect relevant information related to these events. Data from pre-excavation rock mass grouting performed in Norwegian projects are well suited for this type of study as high grouting pressure is common procedure.

Initially in this study grouting pressure and flow during grouting were examined by visual inspection of graphs from grouting logs. The grouting logs were retrieved from contractors, after the grouting work was finished. The sites were Norwegian road and railroad tunnels excavated with drill and blast in hard rock mass, such as granitic gneiss. The method of grouting was high pressure pre-excavation grouting with grouting pressures up to 80 bar, both systematic grouting with overlapping rounds and grouting on demand. The grout was cement based and made from Portland clinker, additive was superplasticiser. Silica

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was added if industrial cement was used.

The purpose of this initial work was to learn about typical grouting behaviour, find evidence of hydraulic jacking and learn how the operators pursued the criteria given in the contract. During this work, it was learned that the grouting progress of each hole differs greatly, the operator's technique has great variability and that hydraulic jacking is a very common occurrence during grouting in Norwegian tunnelling projects. To achieve a correct identification of hydraulic jacking in the available data, it is essential to find a reliable characterisation for hydraulic jacking. By defining boundary conditions for hydraulic jacking in the data, it would be possible to interpret larger datasets and ensure the reproducibility of the results.

This paper presents the development of a method for identification of hydraulic jacking during rock mass grouting and the making of an algorithm to perform computerized detection of hydraulic jacking in grouting logs. The algorithm is the base for a larger study consisting of screening for hydraulic jacking in data obtained from over hundred grouting rounds distributed on several Norwegian tunnels excavated in rock mass. In future publications, the results generated from the algorithm will be compared to geological parameters, theoretical assumptions on when hydraulic jacking will occur, type of grout, grout consumption and time usage.

## 2. Hydraulic jacking

Hydraulic jacking occurs when the pressure inside the fracture is higher than the normal pressure acting on the fracture. This force makes the fracture open, which means that the aperture of the fracture is increasing. Gothäll and Stille (2009) suggests that this process is evolving through three different regimes. The first regime is the low pressure regime where the major part of the load is carried by the contact asperities in the fracture, the second regime is the critical regime where the pressure in the asperities is equal to the pressure of the fluid/grout. The final regime is the post-critical regime, where the grout pressure exceeds the normal pressure and the asperities are no longer in contact. All three regimes may be present at the same time during grouting, but in different parts of the fracture. During unloading of the asperities in the transition between the low pressure regime and the critical regime, the dilation of the fracture will be of the same scale as the decrease in elastic deformation of the asperities. The increase in the fracture aperture during this process is estimated to be five percent or less, if the fracture walls are reasonably matched. This aperture increase is relatively small and of no practical importance for grouting, according to Gothäll and Stille (2009). In the post-critical regime, the increase in aperture is significant and with practical importance for the grouting. A significant increase in aperture would lead to a change in the flow pattern of the cement, which could be visible in the flow and pressure data recorded at the grouting rig.

## 3. Detecting hydraulic jacking from pressure and grouting logs

There are not many publications which discuss detailed and quantified definition of hydraulic jacking during rock mass grouting regarding interpretation of real data, but some authors discuss how hydraulic jacking can be discovered by using the logged data.

Lombardi and Deere (1993) suggest how hydraulic fracturing or jacking can be detected by looking at the ratio between the flow and the pressure, by dividing flow (Q) on pressure (P), as illustrated in Fig. 1. The curve in graph (d), showing the Q/P ratio over time, decrease as the resistance in the hole is increasing. The sharp peak in the graph is referred to as hydraulic fracturing, or jacking. Just before the pronounced peak in the Q/P ratio, the pressure is dropping (a) and the flow is increasing (b).

Rafi (2014) presented Fig. 2, as an example on how one can distinguish hydraulic jacking of a rock fracture in a grouting log. The Figure is based on the Real Time Grouting Control method (RTGC),

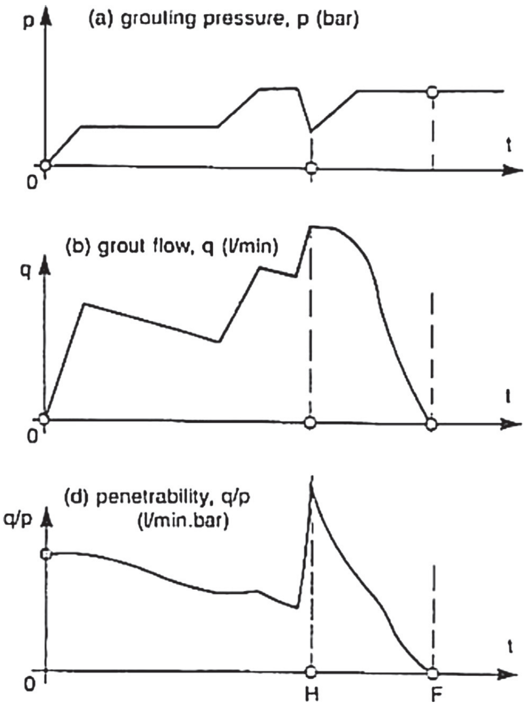


**Fig. 1.** (a) Grouting pressure during hydraulic jacking/fracturing (H), (b) flow during H and (d) Q/P ratio during H (Lombardi and Deere, 1993).

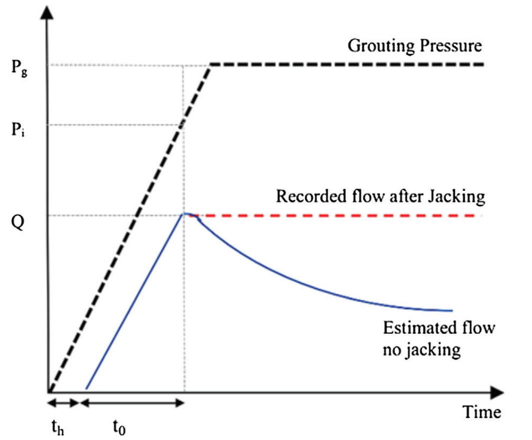


**Fig. 2.** Recorded flow and estimated flow using the RTGC method (Rafi, 2014).

where the grouting work is evaluated with back analysis and it is possible to estimate the expected flow (Stille, 2015). This back calculation requires knowledge of the yield value and the viscosity of the grout. In this case the predicted flow is estimated to decrease while the pressure is stabilizing, as expected when cement based grout is flowing as a 1D (channel) or 2D (disc) flow in a fracture with a constant aperture. The recorded flow is deviating from the estimated flow behaviour and is steady, not decreasing as expected. If the recorded flow behaviour is deviating from the predicted flow it is either due to wrong assumption of the dimensionality, the aperture size or, hydraulic

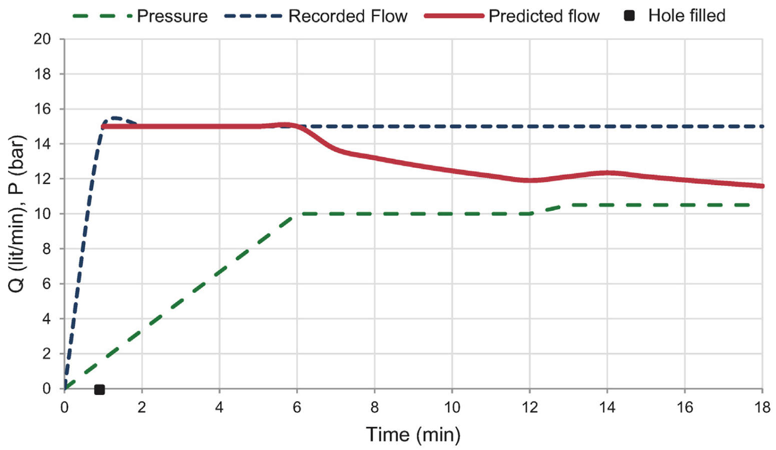


**Fig. 3.** Recorded flow and estimated flow using the RTGC method on real data (Rafi and Stille, 2014).

jacking of the fracture (Rafi, 2014).

Fig. 3 shows a graph of the recorded flow and pressure from a real grouting project in Laos presented by Rafi and Stille (2014). The predicted grout flow is estimated by using the RTGC-method. The onset of the jacking event is when the recorded flow is deviating from the predicted flow, which in this case is when the pressure is stabilizing, at a steady flow rate. There might be other explanations for this type of behaviour of the flow and pressure, as there are several plausible reasons for wrong assumption of the dimensionality and change in aperture size in a fracture, which could represent a pressure and flow behaviour similar to events representing hydraulic jacking or fracturing. This will be discussed further in Section 4.2.

Warner (2004) is putting emphasis on the pressure drop at a constant flowrate as a symptom of hydraulic jacking, but also list numerous events that could cause a pressure drop at a constant flow rate:

1. hydraulic fracturing
2. grout loss into a concealed pipe or other substructure
3. outward displacement of a downslope or retaining wall
4. grout entering much larger fractures or voids
5. grout encountering a softer or more permeable formation
6. thinning the grout or other rheological change that increases mobility
7. leakage of grout and pump malfunction

Warner (2004) also stated that reduction in the pressure increase during the pressure build-up at a constant flow rate, could indicate a significant event in the grouting.

The technique of hydraulic fracturing for estimating the minimum in-situ stress component in rock mass, is also a potential source for learning about the behaviour of flow and pressure in hydraulic jacking during rock mass grouting. The method is performed by isolating a section of a borehole by pressurizing two inflatable packers. The section is placed in intact rock with no prior fractures. Water is pumped into the sealed off section until a fracture is generated, known as hydraulic fracturing. The procedure is repeated at least two times, resulting in a reopening of the generated fracture, known as hydraulic jacking. The procedure is described in detail in ASTM International (2004). The method is well established and a common method for stress measurements in rock mass in Norway.

## 4. Method

### *4.1. Discussion of applicable methods for defining hydraulic jacking in grouting logs*

The first process in this research was visual inspection of graphs from grouting logs and literature study, followed by trial and error approach with data from finished grouting projects with the use of theories and material published by Lombardi and Deere (1993), Stille (2015), Warner (2004) and ASTM International (2004), described above.

The RTGC method was not found to be a suitable method for detecting hydraulic jacking, because the rheology of the cement in the available data was unknown and the method is primarily designed for predicting the spread of grout over time. It was concluded that this method could have potential for detecting hydraulic jacking in the future, with continuous measurements of the rheology of the grout during the grouting process.

The Q/P ratio presented by Lombardi and Deere (1993), was a promising approach, but did not work well with Norwegian data because of the use of high pressure. If the flow is small, and the grouting pressure is high, a significant drop in the pressure would not result in a significant change in the Q/P ratio, which was often the case in the available data.

The behaviour of flow and pressure presented by hydraulic fracturing for in-situ stress testing was also found not to be directly applicable for this study. Table 1 lists some vital differences between hydraulic fracturing for in-situ stress testing and pre-excavation grouting. In practical sense, this means that during the hydraulic fracturing a new fracture is generated and for each time the newly generated fracture is jacked, the fracture is expanding. This means that the water cannot penetrate past the fracture tip, while in rock mass grouting, a fracture of unknown extent is already existing. In most cases the fracture is filled with water before the grouting takes place, also the jacked area might occur in a part of the fracture where the grout front already has passed. For this reason, when jacking occurs the response from the flow and pressure would be of different scale and it is reasonable to believe that the change in pressure and flow would be more significant in grouting. Also, the response during hydraulic jacking would be different with a Newtonian and a Non-Newtonian fluid.

The general understanding derived from the theory presented from Lombardi and Deere (1993), Stille (2015), Warner (2004) and ASTM International (2004) is that hydraulic jacking can be detected by the following pressure and flow behaviour:

**Table 1**
Differences between hydraulic fracturing (HF) for stress measurement and hydraulic jacking (HJ) of fractures in rock mass grouting.

| | Parameter | HF | HJ |
|---|---|---|---|
| 1 | Fluid | Water (Newtonian fluid) | Cement based grout (non-Newtonian fluid) |
| 2 | Hole section | 1 m | 13–27 m |
| 3 | Fractures | No pre-existing | Pre-existing |
| 4 | No. of fractures | 1 generated | Unknown |
| 5 | State of fractures | Fracture propagates during reopening cycles. No fracture infilling | Unknown extent of fractures, and apertures may vary greatly. Might have fracture infilling |
| 6 | Orientation of fractures | Generated perpendicular to direction of least in-situ stress | Random orientation |
| 7 | Sampling interval | < 1 sec | Approximately every 10th sec |

- sudden decrease in pressure while flow is stable
- sudden increase in flow while pressure is stable
- both decrease in pressure and increase in flow
- steady flow and pressure, after a period of pressure increase

### *4.2. Different events with similar responses of pressure and flow behaviour*

Is the suggested pressure and flow behaviour the signature for hydraulic jacking, or could there be other events during the grouting that could exhibit the same behaviour? Both Rafi (2014) and Warner (2004) states that this is the case. Therefore, it was important to distinguish these events from the data and remove them, to ensure a representative selection of hydraulic jacking events for further analysis. To obtain an understanding of the behaviour of grout flowing through sections of changing geometry, a simple numerical model was made to illustrate the behaviour of a Bingham fluid trough changing geometries, Fig. 4.

The simulation was made by designing a 2D numerical model of a Bingham fluid flowing in a 1 m wide channel between two surfaces with changing aperture, the method for modelling is derived from Morris et al. (2015). The yield stress and the viscosity of the Bingham fluid was respectively, 0.94 Pa and 0.017 Pa·s. These values were derived from Eriksson et al. (2004), and is representing ordinary Portland cement, milled down to a $d_{95}$ of 30 µm, mixed in a field mixer with a w/c ratio of 0.8 and 1% superplasticizer. All simulations have a channel length of 20 m, with a constant flow of 10 l/min.

Fig. 4 shows the results from the simulation of the three different fracture geometries. In model (a) the aperture of the channel is 420 µm. In a section of 2 m the aperture is increased to 4 mm, representing a void in the channel. The simulation shows a linear increase in pressure before the grout is entering the void. This pressure behaviour is as expected because of the increase in total friction in the flow path. After the grout front enters the void there is no significant increase in friction, before the void is filled with grout, which means that the pressure is almost constant. As the grout front is forced through the last section with smaller aperture, new friction is added to the system and the pressure is increasing. This means if grout is entering a void, or an open space, and the fracture geometry before the open space remains unchanged, and the rheology of the grout is unchanged, the pressure will not decrease at a constant flow rate, as suggested by Warner (2004), point 2, 4 and 5 in Section 3. Also, this simulation indicates that stabilizing pressure after a pressure increase at a constant flow, does not necessarily mean hydraulic jacking.

In model (b) the channel has an increasing aperture, from 340 µm to 1360 µm. The simulation shows a rapidly increase in pressure, as the total friction in the channel is increasing rapid in the narrow area. As the aperture in the channel increases, less friction per length of channel is added and the increase in pressure is reduced significantly. This illustrates that a grouting log with constant flow and a pressure increase followed by a relative stable pressure could indicate an increase in fracture aperture, increase in width of the fracture, grout entering a void/open space, or intersecting another fracture.

In model (c) the simulation is showing a channel with a constriction. Initially the aperture is decreasing from 400 µm to 180 µm, followed by an increase to 400 µm. The simulation shows a relatively low increase in pressure followed by a rapid increase in pressure as the channel aperture is decreasing. When the grout enters the section with the increasing aperture, the increase in friction per length of channel is significantly reduced, and the increase in pressure is also significantly reduced.

In other words, as the front of the grout moves through the fracture system, it is accumulating friction. As long as the source of the friction in the pathway remains unchanged and the flow is constant or increasing and the rheology of the grout is unchanged or getting thicker, these simulations indicate that it is not possible to see a fall in pressure. An exception would be occurrences at the grouting rig. Grout eroding the fracture or bursting through joint fillings blocking the flow path, might result in a reduction in pressure under a constant, or increasing flow. Hydraulic jacking, where the fracture aperture is increasing, or hydraulic fracturing, where a new fracture is generating a new pathway for the grout, could also exhibit this flow and pressure behaviour.

It was concluded that the following events could exhibit steady flow and pressure, after a period of pressure increase:

- larger aperture further out in the fracture
- the grout is first flowing in a channel and then starts radial flow when the grout reaches a more open area (from 1D flow to 2D flow).
- the grout has reached an intersection with another fracture
- the grout has reached a void, a free surface, or a neighbouring drill hole
- erosion of material in the fracture
- grout bursting through joint fillings blocking the flow path
- changing rheology of the grout
- grout set in to motion after a stoppage or rapid increase of flow
- hydraulic jacking

In the grouting logs, it is not possible to distinguish between most of these numerous events because of the unknown geometry of the fracture systems and no continuous verification of the grout rheology during the grouting. It was therefore decided to exclude steady flow and pressure, after a period of pressure increase as an indicator for hydraulic jacking in this study.

It was concluded that the following common events could exhibit a decreasing or stable pressure behaviour while the flow is increasing, alternatively increasing or stable flow while the pressure is decreasing:

1. hydraulic fracturing
2. grout set in to motion after a stoppage or rapid increase of flow
3. sudden washout of fine material that caused constrictions in the fracture
4. the grout bursting through joint fillings blocking the flow path
5. increase of flow while the grout is filling an open void
6. hydraulic jacking

Hydraulic fracturing of intact crystalline rock would require significantly higher pressure than 80 bar. In Norwegian tunnelling projects, grouting of holes that do not take grout is terminated because of the risk of ejection of the packer and the attached grouting rod. Therefore, it is not likely that hydraulic fracturing is occurring during grouting in the projects presented in this study. To avoid recording events caused by rapid increase in flow by the operator, or when grout was set in motion after stoppage, as described in point 2 above, it was decided that sudden significant increase in flow, or the first 40 s of grouting after stoppage should not be screened for hydraulic jacking. It

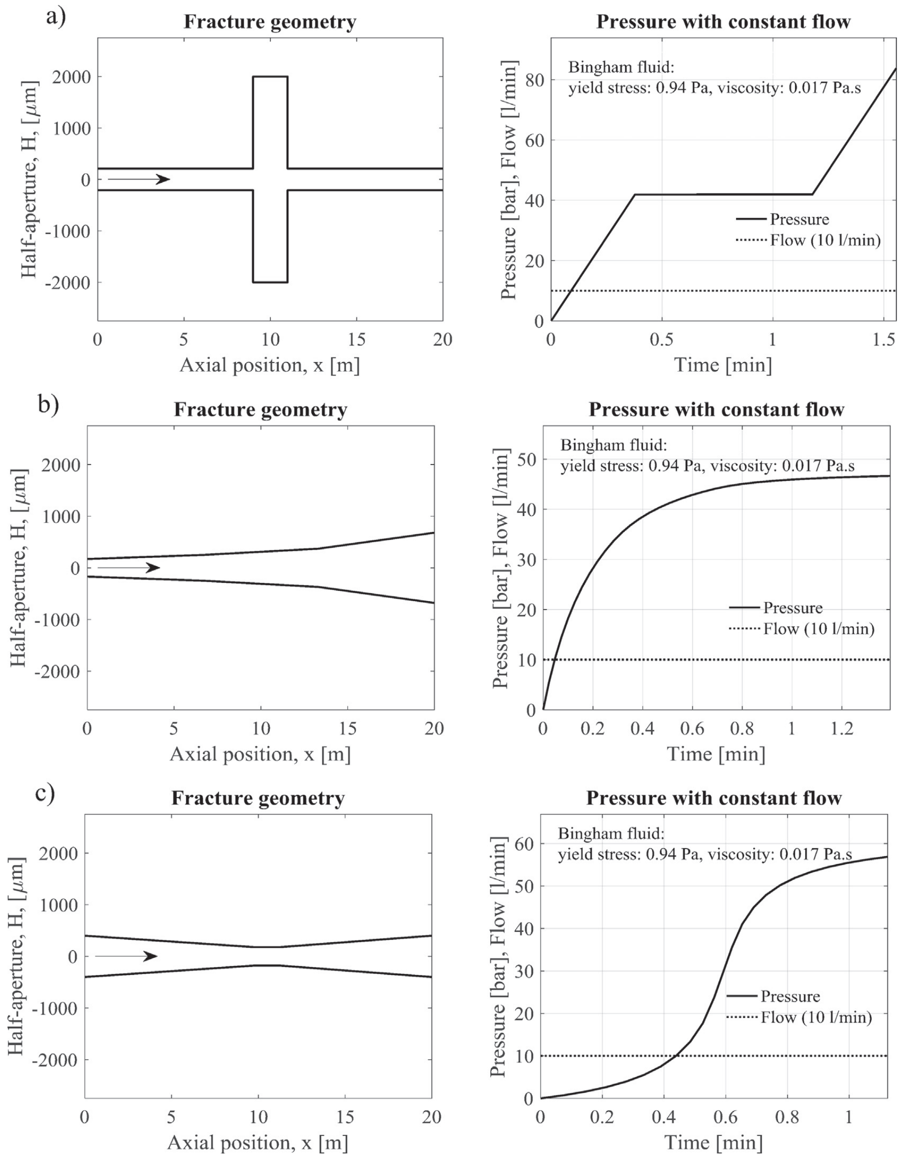


**Fig. 4.** Numerical model (a) grout flowing in a channel, encountering a void and the accompanying flow and pressure behaviour. Model (b) grout flowing in a channel with increasing aperture. Model (c) grout flowing through a channel with a constriction.

was also decided that the first 90 s after the grouting has started should not be screened for hydraulic jacking, because of hole filling and transition for the grout to start filling fractures. The detailed specifications regarding these decisions can be studied in Appendix A. Originally the screening was set to begin when pumped volume was larger than hole volume, but this approach did not fit the real data. The reason for this is probably that many holes are filled with water or grout from linked holes before the grouting starts.

Supplementary data associated with this article can be found, in the online version, at https://doi.org/10.1016/j.tust.2018.06.027.

The specific details regarding flow and the length of the time periods mentioned, were decided after a careful study of grouting behaviour in different projects, grouting rigs and operators. These restrictions will eliminate events with hydraulic jacking occurring during rapid changes on the grouting rig, but because of the uncertainties it would bring to the results it was chosen to apply these restrictions.

The research so far has not revealed a method to differentiate sudden washout of material that have caused constrictions and blockades in the flow path. It is assumed that such events occur at lower pressures than hydraulic jacking. Further work is planned to analyse the correlation between site parameters and events occurring at low pressures, and new knowledge is expected to be obtained.

During the trials with real data it was also found issues regarding the relation between flow and pressure when grouting in very open fracture systems. When the fracture system is filled with fluid (water/grout) the interaction between flow and pressure is very respondent, which mean that if the flow is increased, the pressure is increasing as a response. If the grout is filling an open void/fracture, it seems like the pressure does not respond as pronounced. This hypothesis is partly supported by the numerical model shown in Fig. 4, numerical model (a). This can result in false positive jacking events in open fracture systems with a high grout take. Since it is difficult to achieve high pressure under such conditions, these false recorded hydraulic jacking events will be occurring at low pressures. This is another reason why it is important to examine the results for hydraulic jacking recorded at low pressures more thoroughly. In this study, it is chosen to leave out events occurring when the value of pressure, given in bar, is equal to or lower than the value of the flow, given in l/min, because it indicates high flow with little response in pressure increase. The criterion is valid only when the event is starting, not during the event. This measure does not solve the issue, but reduce the extent of the issue.

### 4.3. Interpretation of real data

A major challenge during the interpretation of data from the grouting rigs was, low sampling rate in combination with oscillations in pressure and flow caused by the piston/plunger pump. The sampling rate is approximately every 10th second on some grouting rigs, whilst on other grouting rigs the sampling rate is more irregular and approximately every 17th second. Both infrequent and irregular sampling causes aliasing, and the behaviour of the flow and pressure between each sampling is unknown. Handling data with aliasing is challenging and can lead to misinterpretation; data from grouting rigs with a sampling frequency lower than every 10th second are therefore eliminated from the study.

Another important issue in this analysis is how representative the pressure measured at the grouting rig is for the pressure in the grouting hole. Tests performed have shown that pressure measured on the grouting rigs is slightly lower, but close to the pressure inside the grouting hole (Tunbridge et al., 2014). The pressure loss from the grouting rig to the hole is small, this also includes the oscillation caused by the piston or plunger on the grouting pump, see Fig. 5. In this case the sampling rate on the pressure logger in the hole is once every second, while the sampling rate on the grouting rig is once every 10th second. The pressure curve indicated by the extrapolation between samplings on the rig every 10th second is very similar in shape to the pressure curve indicated by the extrapolation between samplings in the hole every second. This result indicates little aliasing in the data, as long as the pump cycle is 20 s or longer while the sampling rate is every 10th second (Nyquist criterion). The oscillation in pressure in combination with low sampling frequency, is considered as a negative factor in this study, but it is important to emphasise that the oscillating pressure could give a positive effect on the grouting process with regard to penetration of cement into small fractures (Ghafar et al., 2016).

### 4.4. Development of the PF index

Interpretation of pressure and flow as two separate variables, which are affected by oscillations by the piston/plunger pump and low sampling frequency, turned out to be a challenging task. It was soon discovered that it would be useful to find a parameter that could represent the relation between flow and pressure. Such a parameter could illustrate the grouting process and emphasize changes in the relationship between pressure and flow, that could indicate the occurrence of significant events, e.g. hydraulic jacking and be useful when defining boundary conditions for detecting hydraulic jacking in the data.

Different methods for illustrating the relation between pressure and flow were investigated. The first approach was using the Q/P ratio presented by Lombardi and Deere (1993). Since this was not found to be an adequate method for Norwegian projects, new alternatives were investigated. It was found that by simply subtracting the pressure from the flow the pressure build-up is represented in a distinct way, as well as the general relationship between the volumetric flow and the pressure. To create a dimensionless value with a practical scale, some adjustments were made to the formula. Addition of 90, creates a positive value and multiplication with 0.9 adjusts the range of the scale. This value is hereby called the PF index (Pressure Flow index), Formula (1). The PF index was first introduced as the QP index in Strømsvik and Grøv (2017), but is slightly modified afterwards to make the index dimensionless.

$$PF\ \ index = 0.9\ \ \mathrm{min}/l{*}Q_v - \frac{0.9{*}P}{1\ \ \mathrm{bar}} + 81 \tag{1}$$

where $Q_v$ is flow given in l/min and $P$ is the grouting pressure measured at the grouting rig given in bar.

Fig. 6, examples (a) and (b) show comparisons of the Q/P ratio and the PF index. In example (a) the flow is stable at approximately 15 l/min, 30 min after the grouting has started there is a pressure drop, with a slight increase in flow, which can indicate the occurrence of a hydraulic jacking. This event is accompanied by a significant increase in the PF index, but the Q/P ratio does not give any significant indication of this event. In example (b) the event occurring after 7 min and 30 s of grouting indicate hydraulic jacking. The event is more distinct than in example (a) and both PF index and Q/P ratio show a significant increase during this event, but the PF index is more pronounced compared to the maximum and minimum values of the PF index and Q/P ratio. These examples illustrate why the PF index is the preferred choice for recognising hydraulic jacking.

To be able to use the PF index as a tool for computerized interpretation of hydraulic jacking on data logs from the Norwegian projects it was necessary to perform further adjustments by filtering the data, this process will be described in Sections 4.4.2 and 4.4.3.

#### 4.4.1. Interpretation of flow

On the grouting rigs the flow is measured by a flowmeter, and in

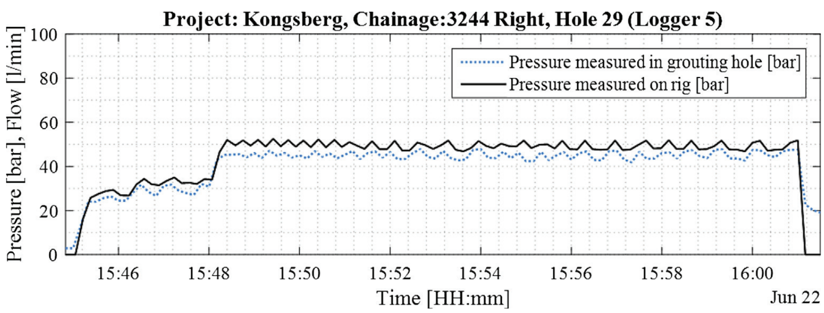


**Fig. 5.** Grouting pressure measured at the rig every 10th second and grouting pressure measured in the hole every second (Strømsvik and Grøv, 2017).

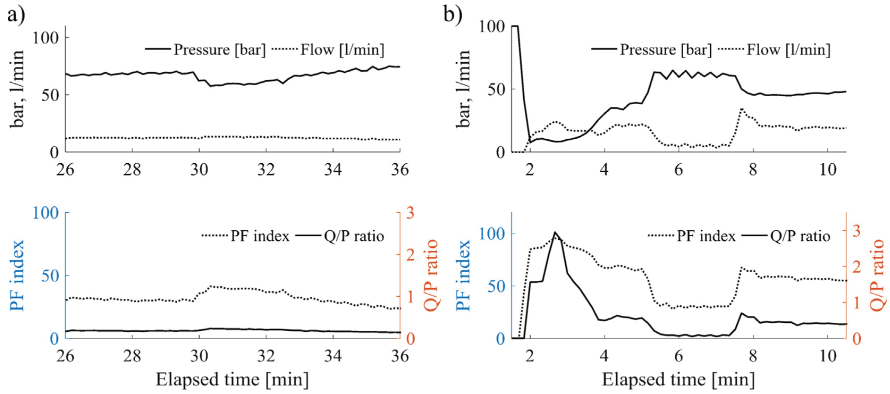


**Fig. 6.** (a) and (b) pressure and flow over time with the accompanying PF index and Q/P ratio shown beneath.

addition the pumped volume is often continuously measured. This means that the volumetric flow is measured in two different ways. The measurement by the flowmeter is greatly dependent on the timing of the sampling in the pumping cycle, consequently if this measurement is used to back calculate the pumped volume, it would give incorrect results. If the pumped volume is used to estimate the volumetric flow over time, the flow is not effected by the pump strokes to the same extent, but directly related to the actual pumped volume. After a trial with both parameters it was chosen to use the measurement from the pumped volume to calculate the value for volumetric flow ($Q_v$), when calculating the PF index. The measurement performed by the flow meter ($Q$) is also an important parameter in the detailed analysis in the algorithm. The specific details can be found in Appendix A, where a detailed flow chart of the algorithm is presented.

#### 4.4.2. Filtering pressure data

In some of the grouting logs, sudden large pressure drops occurred occasionally, followed by a similar pressure increase in the next sampling whilst the pressure and flow were in general steady, see Fig. 7(a). The reason for these occurrences are not known but these events are not related to hydraulic jacking or fracturing, therefore a moving median

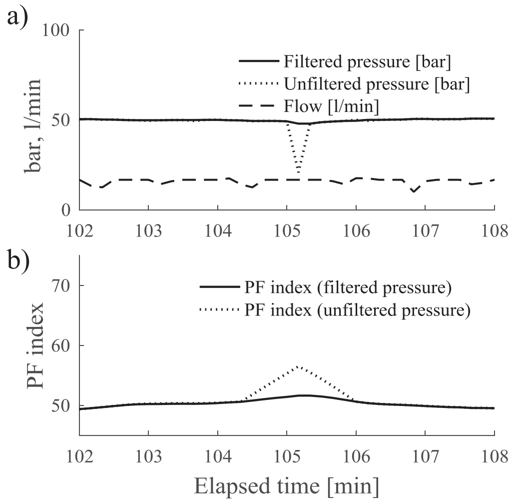


**Fig. 7.** (a) Filtered pressure and unfiltered pressure, (b) PF index estimated with filtered and unfiltered pressure.

with a span of three was applied to the logged pressure to remove these events from the data. The large sudden drops in pressure have significant effect on the PF index, as seen in Fig. 7(b), where the pressure drop has influenced the PF index over a larger time span. This is due to a moving average filter applied on the PF index, described in the next section. In some cases, this pressure drop could wrongfully be interpreted as hydraulic jacking and this is the reason for using the moving median filter on the pressure log.

#### 4.4.3. Filtering of the PF index

The purpose of the PF index is to present an overall trend of the grouting progress. The oscillation in pressure and flow caused by the pump is considered to be noise in the data, and thus the PF index can be smoothed. This does not mean that the oscillations are removed from the full analysis, but only in the PF index. A double moving average filter with a span of five, has shown to be effective in removing the oscillations, but preserving the general trend of the grouting behaviour. The span of five is adapted to data where sampling is approximately every 10th second, and must be reassessed with other sampling intervals. By using such filtering, one must be aware that a sudden event will affect the neighbouring samplings as shown in Fig. 7(b). On the three samplings closest to a sudden decrease of the flow by the operator, the moving average filters should not be applied directly, but be applied separately. Otherwise, abrupt changes in the pumping will be smeared out by the filter, and might lead to erroneous interpretation of hydraulic jacking.

### 4.5. Creating the algorithm

The PF index in combination with logged pressure, flow and time was further used to create an algorithm which can indicate the onset of events plausible to be hydraulic jacking. The algorithm was developed and tuned through analysis of a great number of grouting rounds from varying geology, overburden, w/c ratios, grouting rigs, operators and projects, to ensure that the algorithm would work correctly for the different tunnelling conditions. The boundary conditions for what is considered as a hydraulic jacking or not, is based on the literature study, study of hydraulic fracturing tests, the simulations with the 2D numerical model and study of grouting logs from pre-excavation grouting in rock mass in Norway.

## 5. Results

### 5.1. Algorithm to detect events in grouting rig data in rock mass grouting

The main attribute of the algorithm is that the PF index should

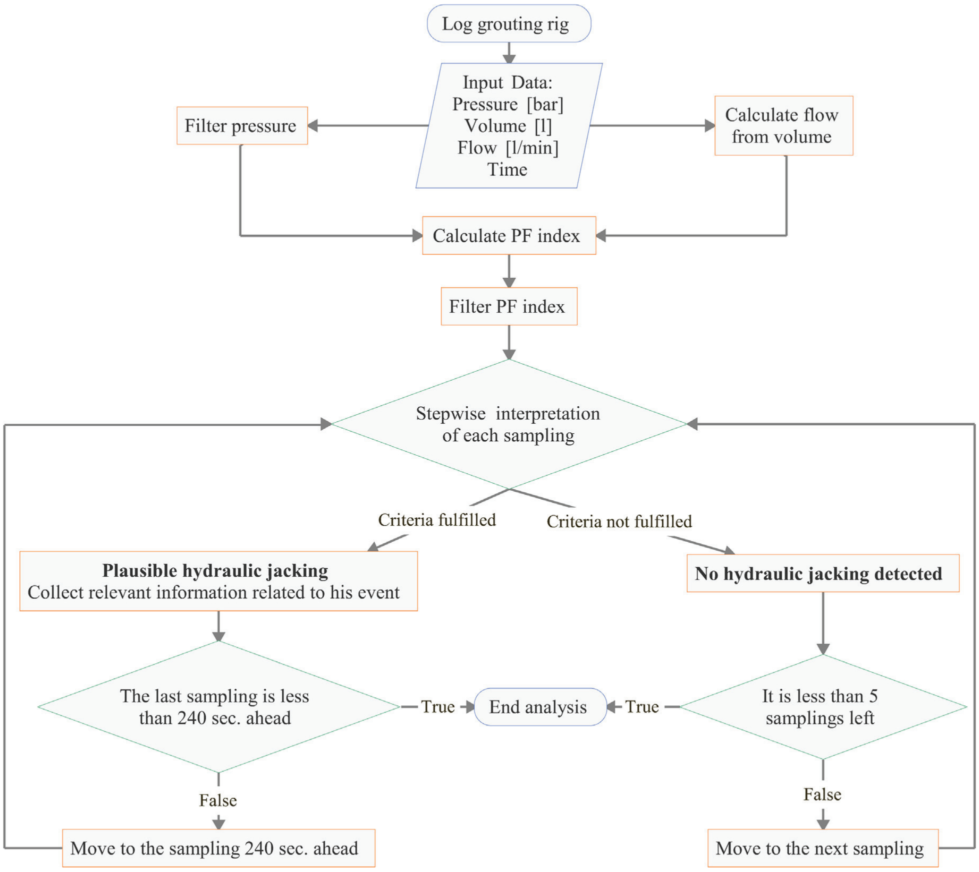


**Fig. 8.** Flow chart showing the structure of the algorithm created to detect hydraulic jacking in data logs from grouting rigs.

increase by a given value within a limited timeframe, at the same time as the pressure is stable or reduced while the flow is stable or increasing. In addition, the algorithm can identify and exclude the following scenarios in the grouting behaviour:

1. start of grouting, or restart after temporary stoppage
2. rapid reduction of flow by operator (also decrease in pressure)
3. rapid increase of flow by the operator (also increase in pressure)
4. repeated rhythmic pressure drops caused at the pump by the pumping cycle

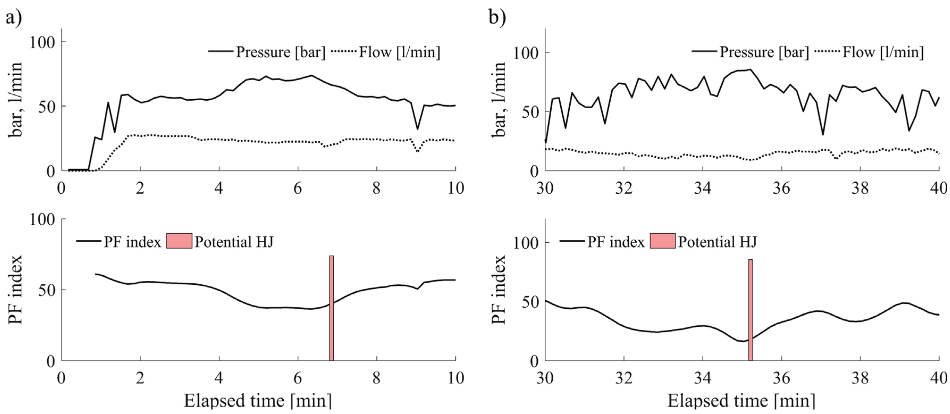


**Fig. 9.** (a) and (b) pressure and flow over time with the accompanying PF index beneath. The bar on the PF index is marking the onset of hydraulic jacking (HJ).

Fig. 8, shows a flow chart illustrating the structure of the algorithm created for detecting events indicating hydraulic jacking. A detailed flow chart of the algorithm is presented in Appendix A.

When the algorithm has located potential jacking, information related to the event is gathered, such as total volume grouted, time, pressure, integral of pressure, etc. This information can be compared to the data from the site, such as grout type/properties, overburden, rock mass quality, etc.

Fig. 9, examples (a) and (b) show a graphic view of two plausible events where the onset of hydraulic jacking is identified by the algorithm. In example (a) there is a pressure drop with a relative constant flow and in example (b) there is both increase in flow and decrease in pressure where the algorithm indicates the plausible jacking. The case shown in (b) is slightly more complicated to interpret because of several events with an increase in the PF index and high fluctuations in the pressure, but the boundary conditions given for hydraulic jacking appear to be set at an appropriate level, as the algorithm identifies the most likely jacking event.

### 5.2. Other uses of the PF index

The PF index was created as a tool in computerized screening for hydraulic jacking, but it was discovered that the index could be useful in other applications as well. It prominently illustrates the pressure built-up, and could be used to monitor the progress of the pressure build-up during the grouting as well as identifying hydraulic jacking during grouting. In Norway, the most important aspect in the stop criteria today is reaching a pre-defined stagnation pressure (the grout flow must be close to zero), or a pre-defined pressure with a flow beneath a certain value. By using the pre-defined pressure and flow, the PF index representative for the stop criterion can be calculated. In Fig. 10, the stop criterion related to pressure and flow is marked in grey. As the PF index reaches the grey level the criterion is reached. In this case the stop criterion is set to be a grouting pressure of 80 bar, with a flow of 5 l/min or less. The pressure built-up is getting close to the stop criterion, but what appears to be a severe hydraulic jacking aborts the pressure build-up phase, and the grouting continues at a significant lower pressure.

## 6. Discussion

The main challenges in this study were handling data generated from low sampling frequencies in combination with piston/plunger pumps and unknown geometry of the fracture system. It was therefore necessary to develop an appropriately adapted method for the available data in this project, which has a sampling frequency of approximately once every 10th second. If the method is to be used for other sampling frequencies, it must be carefully adapted. As discussed in the description of the method development, there are several factors that need to be taken into account, such as adjustments made by the operator, fluctuations caused the pumping cycle on the rig and rheological attributes when the grout is pushed from standstill to flowing. Trials of the algorithm show that in most cases all these factors are eliminated when the algorithm is detecting jacking, but it has not been possible to eliminate all the different varieties of disturbance, that might lead to wrongful interpretation of hydraulic jacking. The absolute most challenging aspect of this study has been interpreting hydraulic jacking at lower pressure, because in cases where the grout is filling an open fracture, the pressure does not respond as pronounced. This can result in false positive jacking events in open fracture systems with a high grout take. In further analyses, parameter analysis will be performed to see if it is possible to discover how the events recorded at lower pressure behave, compared to those recorded at higher pressure.

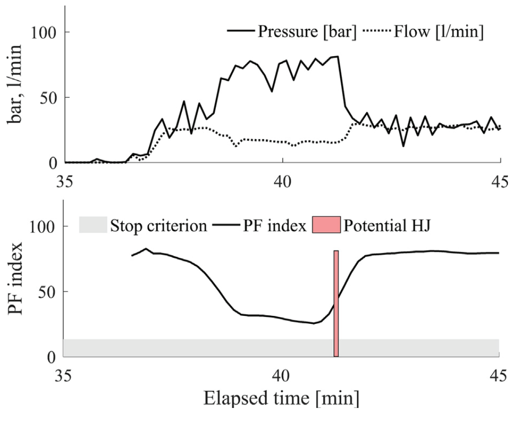


**Fig. 10.** Pressure and flow over time with the accompanying PF index beneath. The bar on the PF index is marking onset of hydraulic jacking (HJ).

Despite all the discussed constraints, the testing of the method to detect the onset of hydraulic jacking has been successful in regard to the presented theoretical assumption of when hydraulic jacking can be assumed, and the method seems well suited for the further plan to perform a large-scale analysis of hydraulic jacking events occurring in Norwegian tunnelling projects. Currently it is not possible to verify the method directly to provable occurrences of hydraulic jacking during rock mass grouting, because there are no available methods to register these events. Therefore, the developed algorithm and the PF index is theoretical, it is developed through findings from the literature study, study of hydraulic fracturing tests, the simulations with the 2D numerical model and the study of grouting logs from pre-excavation grouting in rock mass in Norway.

As the future logging equipment and grouting rigs are developing, the interpretation of grouting behaviour will be less challenging and give more exact interpretations of the grouting behaviour and performance. During this development, it is also reasonable to believe that the operation of the grouting progress in the field could be more computerized and less operator dependent. Also, the rig could be monitored from office and the grouting procedure adapted to the site, dependent on the continuous collected data.

## 7. Conclusion

Recognising hydraulic jacking during rock mass grouting and the making of an algorithm to perform computerized detection of hydraulic jacking in grouting logs proved to be a challenging task, because of low sampling frequencies in combination with piston/plunger pumps and unknown geometry of the fracture system. The study concluded that the best way of detecting events with hydraulic jacking is focusing on increasing or stable flow while the pressure is decreasing, alternatively a decreasing or stable pressure while the flow is increasing. Events that were found to cause risk for false positive interpretation of hydraulic jacking are the following:

1. Grout pushed into motion after standstill
2. Operator decreasing or increasing the flow at a rapid rate
3. Fluctuations caused the pumping cycle on the rig
4. Erosion of fracture infilling or bursting through fracture infilling blocking the flow path
5. Grouting with low pressure, in an open fracture system

The issues presented in point 1–3 were mainly solved in the algorithm, but point 4–5 needs to be further investigated by interpretation of grouting behaviour during different events.

Despite all the discussed constraints, the testing of the method to detect the onset of hydraulic jacking has been successful in regard to the presented theoretical assumption of when hydraulic jacking can be assumed and the method seems well suited for the further plan to perform a large-scale analysis of hydraulic jacking events occurring in Norwegian tunnelling projects. In future publications, the results generated from the algorithm will be compared to geological parameters, theoretical assumptions on when hydraulic jacking will occur, type of grout, grout consumption and time usage.

During the development of the method a new parameter was created, called the PF index. The index is a dimensionless number, representing the relation between flow and pressure during grouting. It was found to be useful when screening for hydraulic jacking, general interpretation of the grouting progress and could work as a target value, if the grouting criterion is based on reaching a pre-defined pressure at a pre-defined flow rate.

Detection of hydraulic jacking and collection of data related to these events will give valuable information that could contribute to the understanding of such events during rock mass grouting. Identification of incidents of hydraulic jacking during grouting would be a valuable tool for adapting the grouting procedure to the actual site conditions. The matter of which hydraulic jacking should be avoided or not is not within the scope of this article.

## Acknowledgements

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# Paper II

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# The significance of hydraulic jacking for grout consumption during high pressure pre-grouting in Norwegian tunnelling

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ABSTRACT

Pre-grouting is a common measure for reducing water ingress into tunnels. This study has investigated parameters which are expected to have an impact on grout consumption during high pressure pre-grouting of rock mass in Norwegian tunnelling, with special attention to the occurrence of hydraulic jacking of fractures. A significant positive correlation between grout consumption and hydraulic jacking was found, especially when grouting with microfine cement, but it was not established to what extent hydraulic jacking contributes to the total grout consumption. Also, there was found to be a general increase in time consumption for grout holes where hydraulic jacking was indicated. It was concluded that hydraulic jacking does not tend to be a good choice for the economy of the projects, because it involves longer grouting time and higher grout consumption, particularly when using microfine cement.

## 1. Introduction

Hydraulic jacking (HJ) during pre-grouting is an incident which is alleged to have a significant impact on the grout consumption during pre-grouting. Despite this fact, no examples of noteworthy research based on field data has been found in literature. This study attempts to fill this gap, by analysing incidents of HJ in Norwegian tunnelling projects.

Some of the presumed consequences of hydraulic jacking during pre-grouting are:

1. Higher consumption of grout
2. Increased use of time
3. Loss of control
4. Instability in the rock mass

This paper will address the two first bullet points above; consumption of grout and usage of time.

Grout consumption during rock mass grouting governs time and costs and is dependent on numerous variables such as rock mass properties, grout characteristics, grouting pressure and HJ. To be able to assess if HJ is contributing to a significant increase in grout consumption it is important to study the dependency between these parameters. Accordingly, this study considers the above mentioned variables that are presumed to have a significant contribution to grout consumption, in addition to HJ.

The main subject of investigation was grouting data from complete rounds of pre-grouting performed during construction of Norwegian road and railroad tunnels. One round of pre-grouting is typically performed as follows: 25–70 holes of 15–30 m length, drilled into the face of the tunnel, as illustrated in Fig. 1. Packers attached to grouting rods are placed approximately 2 m into the drill holes and grout is pumped into each hole. The grouting rigs commonly have 3–4 grout lines which can operate simultaneously. The grouting is first performed in the bottom holes, moving upwards.

## 2. Basics aspects of pre-grouting

### 2.1. Purpose of pre-grouting and environmental aspects

The main goal of pre-grouting in Norway is to reduce the water ingress according to an allowable inflow rate. As described by Grøv and Woldmo (2012), the tunnel should be “tight enough for its purpose”, meaning that the tunnels do not have to be waterproof, but the water inflow should be reduced enough to safeguard the environment above and inside the tunnel. This should be performed in a cost-effective way, with least possible impact on the local and global environment.

In Norway the most common grout used for pre-grouting is cement based, made from Portland clinker. According to Andrew (2018), decomposition of carbonates is one of the three largest contributors to anthropogenic emissions of carbon dioxide to the atmosphere, the other

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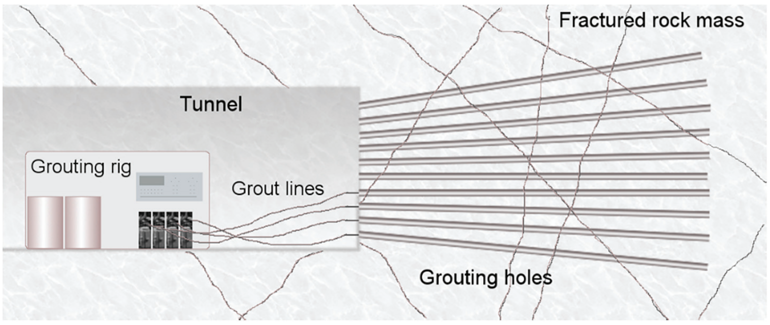


**Fig. 1.** Illustration of one round of pre-grouting.

two are fossil fuels and deforestation. Cement production is the largest source of carbon dioxide emission from the decomposition of carbonates. This reflects that excessive use of cement during pre-grouting is not only undesirable for the project economy, but also for the global carbon footprint.

Pre-grouting is an expensive and time-consuming process in Norwegian tunnelling. Fig. 2 shows estimated costs for grouting per metres of tunnel related to the allowable inflow rate per 100 m of tunnel (LRIR) in 2012. This illustrates the importance of understanding the grouting process to apply the correct grouting procedure in each project.

### 2.2. Hydraulic jacking of fractures in rock mass

Rock mass grouting with the use of high pressure, could lead to hydraulic jacking (HJ) of fractures. HJ occurs when the pressure inside the fracture is higher than the normal pressure acting on the fracture. This force makes the fracture open, which means that the aperture of the fracture is increasing. A more detailed description of this process can be found in Stille (2015) and Strømsvik et al. (2018).

Stille (2015) presents the following negative consequences of HJ during pre-grouting:

- Higher consumption of grout, due to higher flowrate and increased volume of fractures.
- Uplift of the overburden, if the fractures are close to horizonal oriented.
- Increased transmissivity outside the grouted zone, due to increased apertures of fractures.
- Finer factures can be exposed to compression during grouting, making them more difficult to grout.

According to Rafi and Stille (2015) deformation of the fracture could in some cases be beneficial since it might improve the penetrability of grout. In Norwegian rock mass grouting it is believed that in

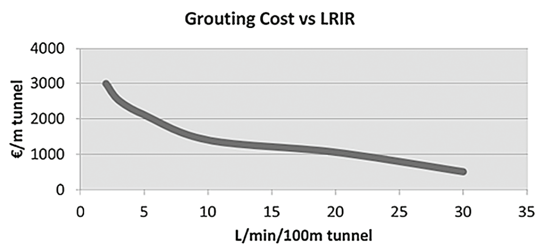


**Fig. 2.** Costs per metres of tunnel relative to allowable inflow rate per 100 m of tunnel (Grøv and Woldmo, 2012).

some circumstances HJ could be beneficial and improve the effectiveness of grouting, particularly in conditions with clay rich fracture fillings (Aarset et al., 2011).

The occurrence of HJ is according to Lombardi (1985) more prone to happen if the grouted volume is high. To reduce the risk of HJ, the limiting envelopes in the GIN-method are designed according to that principle (Lombardi and Deere, 1993). If the grouting pressure is high, the grouted volume should be low, if the pressure is low the grouted volume should be increased.

In Norwegian rock mass grouting practice, it is uncommon to instruct whether or not jacking should be avoided. HJ is an everyday occurrence which normally is not given any special attention during the grouting works. In most cases the grouting continues after HJ has occurred in a grout hole, which makes this study feasible. There is a lot of individual experience-based theories in the industry regarding HJ and the opinions between rig operators, site engineers and cement suppliers vary greatly.

Theory describes that HJ of fractures during grouting has many negative consequences. What are the rationales for using high pressures and risking HJ during rock mass grouting? One argument is that high pressure is optimal to ensure best possible penetration into fine fractures. Draganović and Stille (2011) concluded that high pressure could improve penetrability by eroding partially-built plugs along edges of fractures and hold the fractures open for a long time. Also, a higher penetration rate is achieved in by using high pressure. Pre-excavation grouting in Norway typically operates with a grouting pressure of 60–80 bar, significantly higher than most other places worldwide.

### 2.3. Fracture geometry and grout spread

Fractures in the rock mass are often thought of as infinite planes of two irregular rock surfaces with contact asperities. The flow pattern is dependent of the geometry of the fracture void space, and the flow along some paths is much faster than along others. This difference in flow velocity is called channelling (Hakami, 1995). In a grouted fracture, primarily the central part of the channel will be filled with grout, while narrow zones close to the asperities are hard to grout. Such narrow contact zones might be prone to transport water, depending on the connectivity between the asperities (Pusch et al., 1991). This also illustrates the challenges in predicting the grout spread and grout consumption during rock mass grouting.

In fracture models, the asperities are normally modelled as isolated elements in the fracture, as done for example by Hakami (1995) and Lombardi (2003). This seems to be most likely when pressing two surfaces with irregular surfaces together. This does not support a high connectivity between asperities, thus there is no need to grout these narrow contact zones. This type of distribution of asperities and narrow contact zones might not be the case in fractures with gouge materials, and crystallization. Under such conditions, the connectivity between water conducting narrow contact zones might be of concern and HJ of fractures in these conditions could have a positive impact on the reduction of water ingress.

Because of the considerable length and high number of drill holes in one round of pre-grouting, it is common that several holes intersect the same fracture. The grout consumption and the relationship between pressure and flow in each hole partly depend on the aperture of the fracture in the area close to the grout hole, the general fracture geometry, grout properties and the presence of grout from previously grouted holes. This implies that holes intersecting the same fracture will have different grouting history, not only dependent of the fracture geometry but how the grouting is performed in the full round of pre-grouting. This complicates the analysis of grout consumption in individual holes when interpreting data from pre-grouting. For this reason, it was chosen to evaluate data from full rounds of grouting in this study.

### 2.4. Grout properties and penetrability

As mentioned in Section 2.1, the most common grout used for pre-grouting in Norway is cement based, made from Portland clinker with different fineness. According to the Norwegian Public Road Authorities the fineness of the cement is characterized as following:

- Standard grout cement (OPC): $d_{95} < 40\,\mu m$
- Microfine cement (MFC): $d_{95} < 25\,\mu m$
- Ultrafine cement (UFC): $d_{95} < 13\,\mu m$

The decision basis for which grout to use is individual for each project, largely depending on the preference of the contractor. During the tunnelling project the fineness of the cement is sometimes changed due to diverse challenges during grouting.

According to Stille (2015) the grouts ability to penetrate fine fractures (penetrability) is dependent of the relationship between the size of the grains and fracture apertures. In fine-grained cement this relationship is complex, mainly due to an increase in specific surface area, resulting in greater surface activity. Draganović and Stille (2011) compared a cement with $d_{95}$ of 32 µm with two finer cements with $d_{95}$ of 12 µm and 16 µm, where the finer cements were found to have a considerably poorer penetrability. According to Stille (2012) studies has shown that the penetrability of cements is decreasing with $d_{95}$ smaller than 20 µm. The penetrability is also affected by the water/cement ratio (w/c ratio), cement quality, additives, type of mixer and temperature.

A study presented by Håkansson et al. (1992) found an increase in both yield stress and plastic viscosity with an increase in specific surface of the particles in cement based grouts. Studies presented by Skjølsvold and Justnes (2016) in relation with the TIGHT project, found that in some microfine cements the setting started after approximately 30 min. According to Grøv and Woldmo (2012) the short setting time of microfine cement can give less consumption of grout compared to standard grout cement.

### 2.5. Significance of rock mass conditions

Rock mass properties and ground water conditions are undoubtedly affecting the grouting progress, grout consumption and grouting outcome. The challenge is to define which parameters matter most, and how to predict and adapt grouting to the different conditions, with both satisfactory result and good economy. This large scale study was compiled of available information from project owners and contractors available after construction. The disadvantage of this data set was that there was no assessment or classification of the rock mass before the grouting was performed.

For the classification of the rock mass, the only available data was geological mapping performed during tunnel excavation based on the Q-method (NGI, 2013). For the tunnels included in this study the Q-system was used for the main purpose of designing rock support in tunnels. The weakness of this geological classification data is that the mapping was performed in pre-grouted rock mass, which affects the classification. The data was still considered as useful for the study.

The Q-value is calculated by the following equation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{1}$$

where $RQD$ is Rock Quality Designation, $J_n$ is number of joint sets, $J_r$ is joint roughness number, $J_a$ is joint alteration number, $J_w$ is joint water reduction factor and $SRF$ is stress reduction factor. Further information regarding these parameters can be found in NGI (2013).

There have been several studies on the correlation between grout consumption and Q-value, including its input parameters. Bhasin et al. (2002) evaluated the Q-value and its input parameters in relation to grout consumption in Frøyatunnelen, a subsea tunnel in Norway. In this setting the Q-value was set for the full tunnel profile for a length of 5 m (one round of blasting) in pre-grouted rock mass. In this study there was not established any significant correlation between grout consumption and Q-values, but a tendency towards low Q-values and high grout consumption was found. There was neither found any statistical correlations between grout consumption and the input parameters of the Q-system, but trends indicated higher grout consumption with low RQD, high $J_n$, low $J_r$ and high $J_a$. $J_w$ and SFR were not evaluated due to little variation of the parameter values. Bhasin et al. (2002) also found that if the degree of fracturing ($RQD/J_n$) was below 2, the grout consumption increased considerably.

Rastegar Nia et al. (2017) performed a multiple regression analysis on grout take, Q-value, joint aperture and grout pressure and found a statistically significant correlation between grout take and these parameters. In the study of Rastegar Nia et al. (2017), the Q-system was used for rock mass classification of drill hole sections of five metres, followed by stepwise grouting of these sections. These two studies show a different use of the Q-system in regard to grouting and the study performed by Rastegar Nia et al. (2017) appears to the author to be the most correct method, but this type of detailed data was not available for this study.

### 2.6. General remarks on high grouting pressure and safety

The level of grouting pressure used in Norway might seem extreme and hazardous and some might question how such high grouting pressures inflict on rock mass stability and working safety. It is therefore important to inform that the grouting pressures presented in this paper represent a normal grouting procedure and are not adjusted to fit to this project. No incidents related to personnel injury or rock fall were experienced in any of the tunnels used in this research project.

The reason for the use of high pressure is a combination of geology and experience. The last glacial period eroded and removed most of the weathered rocks covering Norway and Sweden, leaving behind hard and massive bedrock. The majority of the rock mass in Norway is therefore self-bearing, hard crystalline rock. The technique of pre-grouting is developed through many decades of practical experience with underground excavations in this type of geology, resulting in a high grouting pressure, compared to other countries.

**Table 1**
Overview of data from the 6 tunnels used for the study.

| Tunnel | Rounds | Holes | Type cement | Overburden | Geology | Target Pressure |
|---|---|---|---|---|---|---|
| A | 31 | 1012 | OPC | 9–45 m | Gneiss with veins of diabase | 60–80 bar |
| B | 8 | 332 | MFC | 65–100 m | Gneiss with veins of amphibolite and pegmatite | 80 bar |
| C | 12 | 429 | 9 MFC<br>3 OPC | 24–86 m | Gneiss with veins of amphibolite and pegmatite | 60–80 bar |
| D | 16 | 581 | 8 MFC<br>8 OPC | 165–200 m | Banded gneiss | 80 bar |
| E | 6 | 227 | 1 MFC<br>5 OPC | 183–188 m | Amphibolite | 80 bar |
| F | 18 | 810 | OPC | 23–78 m | Monzonite | 80 bar |

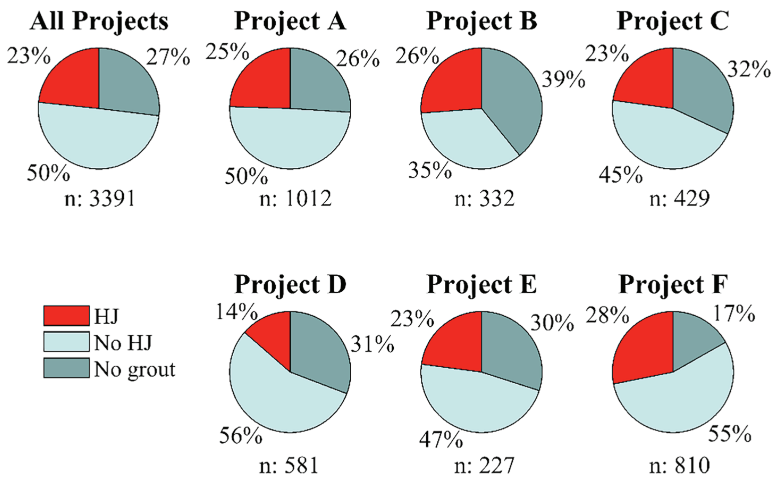


**Fig. 3.** Pie charts showing percentage of grouted holes where HJ was indicated. The dark fields marked "No grout" represents the percentage of holes where the grout consumption was less, or similar to the volume of the grout hole.

## 3. Analysis based on Norwegian cases

### 3.1. Background data

Initially 7 tunnels, all excavated with drill and blast were selected for this research project. Unfortunately, a large part of the collected data had to be discarded, due to a sampling less frequent than every 10th second in the data logs from some of the grouting rigs, or malfunctions in the logging system. However, the data in the study represent 3391 grout holes subdivided on 91 grouting rounds in 6 tunnels. A summary of the tunnels is presented in Table 1.

All tunnels were grouted with Portland cement with different degrees of fineness. In this study, standard cement (OPC) is cement with a $d_{95} = < 40\,\mu m$ and $> 25\,\mu m$ with silica as additive, microfine cement (MFC) is defined as Portland cement with a $d_{95} < 25\,\mu m$ with plasticizer as additive. The target pressures presented in Table 1, is the chief stop criterion for the grouting of each grout hole. None of the grout rounds included in this study was grouted with both OPC and MFC.

The following stop criteria for grouting was used for the grout rounds in this study:

1. Reaching a pre-demined grouting pressure, at the same time as the grout flow should be small, or close to zero.
2. After a pre-determined volume was grouted, with insufficient decrease in the grout flow and/or insufficient increase in pressure, the w/c ratio of the grout was reduced.
3. After a pre-determined maximum volume was grouted, with insufficient decrease in the grout flow and/or insufficient increase in pressure, the grouting was ended.

Collected data from each site consist of:

- Geological report from pre-investigation.
- Final project report (if available).
- Geological mapping and rock mass classification (Q-system). Performed by a site geologist from the project owner, after each round of blasting.
- Grouting procedures.
- Data logs from grouting rigs: timeseries with pressure, flow and grouted volume.

All the data logs from the grouting rigs were screened for HJ, using the algorithm published in Strømsvik et al. (2018). The pie charts presented in Fig. 3 show percentage of grouted holes where HJ was indicated. The dark fields marked "No grout" show the percentage holes where the grout consumption was less, or similar to the volume of the grout hole. These initial results suggest that it is a high degree of HJ during rock mass grouting in Norwegian tunnelling projects. It also shows that a high percentage of the drilled holes are not groutable.

Fig. 4 shows data from 2365 holes grouted with OPC, where the

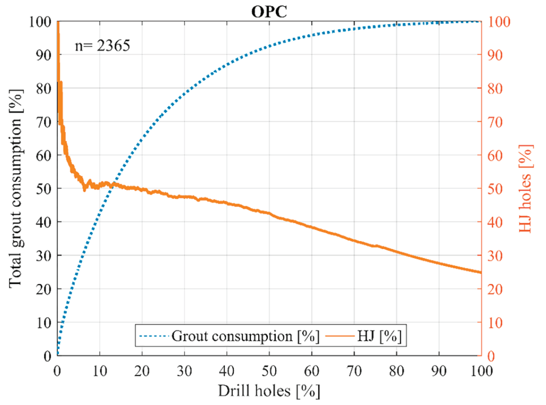


**Fig. 4.** Accumulation of grout consumption and percentage of holes where HJ is indicated, for holes grouted with OPC.

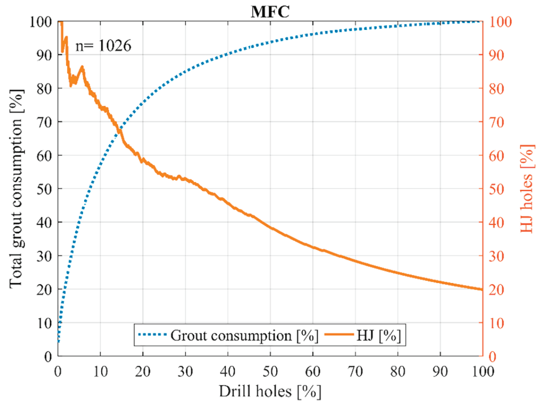


**Fig. 5.** Accumulation of grout consumption and percentage of holes where HJ is indicated, for holes grouted with MFC.

holes are sorted according to grout take. Holes with the highest grout take on the left side, with descending grout take towards right. The blue dotted line represents accumulated grout volume and the orange line represents percentage of holes where HJ is indicated. Approximately 10% of the holes contributes to 43% of the grout consumption and about 51% of these holes have a pressure and flow behaviour that indicate HJ during grouting. The plot indicates a correlation between grout consumption and HJ.

Fig. 5 shows the same data projection as described above, for 1023 holes grouted with MFC. In this selection of data approximately 10% of the holes contributes to 57% of the grout consumption and it is indicated HJ in about 75% of these holes. Less holes account for a higher part of the total grout consumption with MFC, compared with OPC. The correlation between HJ and grout consumption is considerable stronger for MFC than OPC.

### 3.2. Method for correlation analysis and data comparison

To be able to investigate the correlation between HJ and grout consumption it is important to establish the connection between all parameters which affect the grout consumption. It was assumed that the following factors in the available data could have an impact on the grout consumption in one round of grouting:

1. Q-value, including input parameters
2. Percentage of holes with no grout take
3. Percentage of holes with HJ
4. Maximum grout pressure
5. Average grout pressure over time
6. Grouting time
7. Number of holes in a round
8. Length of holes
9. Type of grout
10. The chief stop criterium: reaching a target pressure

A pairwise correlation analysis for bullet points 1 through 8 was performed by using Pearson's linear correlation and the statistical software R Studio (Dalgaard, 2002). The statistical significance level was set to p-value $< 0.05$. Some of the results are presented in correlation matrixes (Figs. 6 and 7), with Pearson's correlation coefficient R. Low R-values are indicated with low contrast, and high R-values are indicated with high contrast. Positive correlations are blue and negative correlations are red. No distinct limits for high, medium and low correlations for R-values are given because this it is an arbitrary value which needs to be evaluated relatively within each correlation analysis. Parameters with no statistical significance are crossed out. The Q-value is treated as a log value in the analysis, since it is presented as a log value by NGI (2013). The Q-value is estimated for 5-meter sections of pre-grouted rock mass after each round of blasting.

The following sample selections were chosen for the correlation analysis:

- All rounds of grouting (n: 91)
- Rounds with OPC (n: 65)
- Rounds with MFC (n: 26)
- Tunnel A (n:31).

Only Tunnel A was chosen to be analysed as an individual project, because the rest of the projects had too low sample size, meaning too few rounds of grouting. For the comparison of grout consumption using OPC and MFC, it was produced histograms showing frequency distributions. This method was chosen because of large difference in sample size. OPC is the most common grout type in Norwegian pre-grouting practice and it was more available data for this type of grout. In addition to grout consumption per round, the following comparisons between OPC and MFC were performed using histograms:

- Percentage of holes with no grout take
- Percentage of holes with HJ
- Grout consumption in non HJ holes
- Grout consumption in HJ holes
- Time consumption per metres hole in non HJ holes (including breaks)
- Time consumption per metres hole in HJ holes (including breaks)
- Percentage of increase in grout consumption in HJ versus non HJ holes
- Percentage of increase in usage of time in HJ versus non HJ holes

The results are presented in Figs. 8–16. Holes with no grout take are only presented in the histogram in Fig. 9, in the other histograms holes with no grout take are excluded from the analysis.

### 3.3. Results from correlation analysis

The total grout consumption per round did not show any significant correlation with length of grout holes or number of grout holes in a round. There was a weak significant negative correlation between number of holes and length of holes ($R = -0.23$). This confirms that it is most appropriate to compare the total grouted volume in a round to the geological conditions at a site, not individual holes.

Fig. 6(a), (b), (c) and (d) show the results of the correlation analysis from the geological mapping performed in the tunnel using the Q-system and grout consumption per round. The top row of the matrixes shows the correlations between grout consumption and the Q-parameters and will be the main focus in the result comments. It was little change within the values for $J_r$, $J_w$ and SRF and they were therefore not included in the correlation matrix. $J_w$, was reported to be 1 in 88% of the rounds. The geological mapping in the tunnel was performed after blasting of the grouted area, which is the probable reason for no or little water in most of the fractures. It also indicates successful grouting, regarding reduction of water ingress.

In the analysis with all 91 grout rounds (a) and OPC (b), it was indicated a significant negative correlation between grout consumption and Q-values, and a significant positive correlation between grout consumption and $J_a$. In the analysis with all rounds both correlations are weak, for OPC the correlations are slightly stronger. In the analysis performed with MFC (c) and Tunnel A (d) no significant correlation was found between grout consumption and Q-values, or any of the input parameters. The results suggest that the correlation with the joint alteration number ($J_a$) is the cause of correlation with the Q-value for OPC, this is confirmed by the relatively high R-value in the correlation between the Q-value and the $J_a$ value. The correlations found within OPC is strongly contributing to the correlations found in the analysis including all rounds. Project A, which is grouted with OPC shows, no significant correlations with the Q-values or the input parameters. Data from project C, D, E, F was then analysed to find the source of the correlations between grouted volume, Q-values and $J_a$. It was found that for project E and F the R-value for $J_a$ was relatively high (0.53 and 0.46), but these correlations were not found statistically significant. In both cases it is a low number of samples and heteroscedasticity in the data, which is likely to be the cause of no statistical significance. Heteroscedasticity means that the variability in $J_a$ is unequal across the range of grout consumption. A correlation analysis conducted without data from tunnel E and F, also resulted in no significant correlation.

The results from the correlation analysis illustrates the danger of statistics, and that no clear correlation was found with grouted volume and Q-value, or any of the input parameters in this study. It is concluded that the only factor in the Q-parameters that show a slight trend with grout consumption is the $J_a$-value, but only for two of the tunnels, tunnel E and F. It was a tendency towards higher grout consumption with high $J_a$ in these two tunnels. The $J_a$ values in all 91 rounds range between 0.75 and 4, where 4 represents more fracture filling. These results will be further discussed in Section 4.

As expected, the correlations between Q-values and the input parameters are generally strong in all selections, but in some cases, there is a weak correlation or no statistical significance. The reason for this is little change in the values for some of the input parameters.

The correlation matrixes presented in Fig. 7(a), (b), (c), (d) show the following parameters from one round of grouting:

- Vol: grouted volume
- HJ: percentage of HJ holes

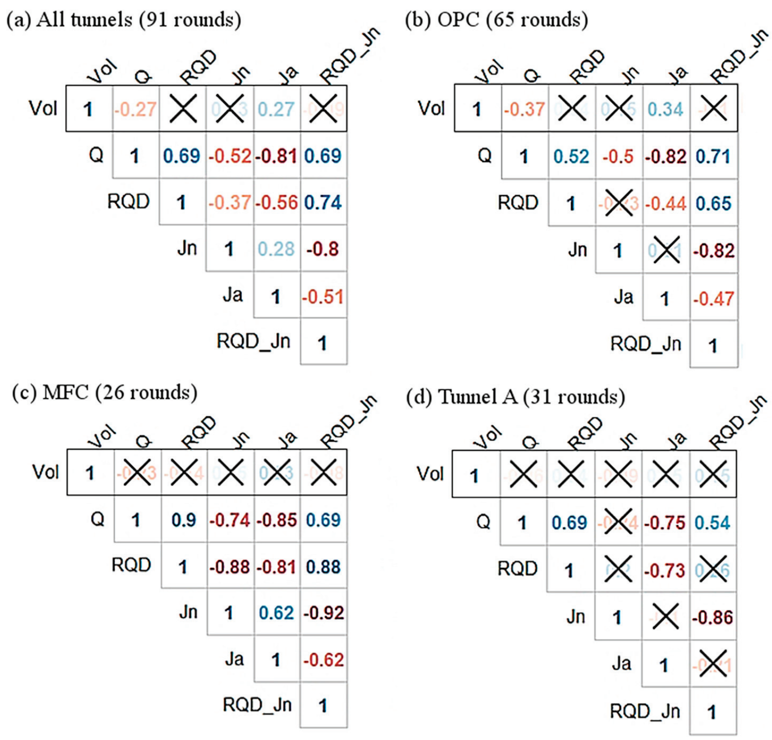


**Fig. 6.** Results from Pearson's linear correlation analysis performed between grout consumption, the Q-value and input parameters, with four different sample selections: (a) all rounds (b) OPC rounds (c) MFC rounds and (d) Tunnel A.

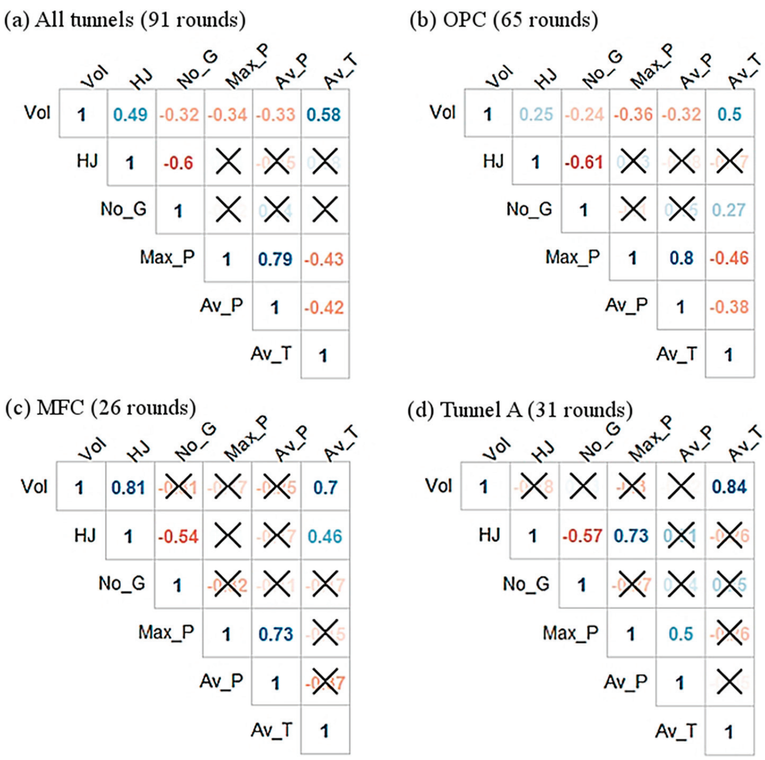


**Fig. 7.** Results from Pearson's linear correlation analysis performed between grout consumption, percentage of HJ holes, percentage of holes with no grout take, average maximum pressure, average of the average grouting pressure and average usage of time with four different sample selections: (a) all rounds (b) OPC rounds (c) MFC rounds and (d) Tunnel A.

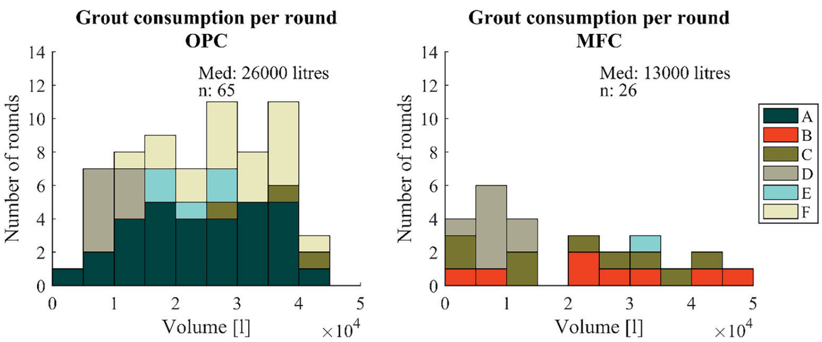


**Fig. 8.** Histograms showing frequency distribution of the total grout consumption per round; rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

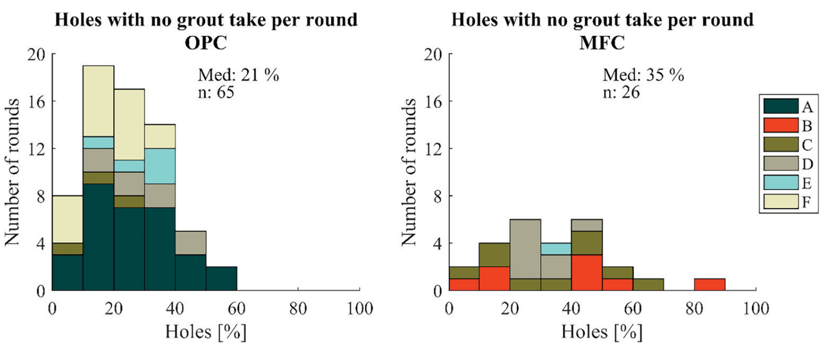


**Fig. 9.** Histograms showing frequency distribution of the percentage holes with no grout take per round. Rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

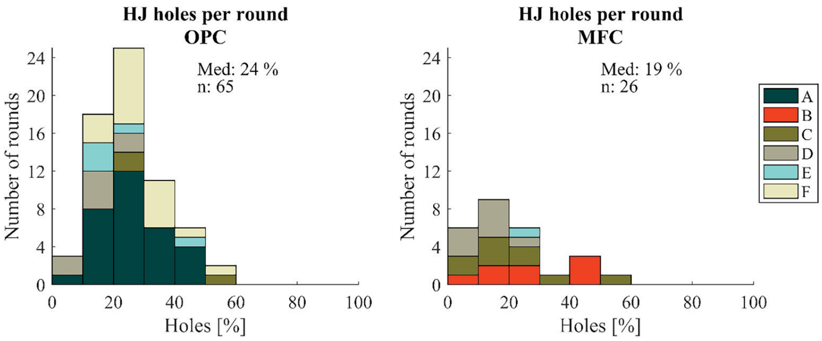


**Fig. 10.** Histograms showing frequency distribution of the percentage of HJ holes per round. Rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

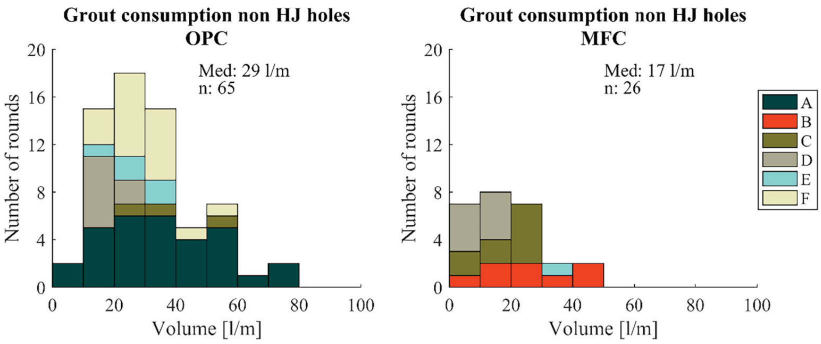


**Fig. 11.** Histograms showing frequency distribution of the grout consumption per metre of drill hole per round, in holes where no HJ is detected; rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

- No_G: percentage of holes with no grout take
- Max_P: average of the max pressure logged in the holes
- Av_P: average of the average pressure logged in the holes
- Av_T: average of the grouting time in the holes

As above, the top row of the matrixes shows the correlations between grout consumption and the listed parameters and will be the main focus in the result comments. The correlation matrix with all rounds of grouting (a) shows that there were found significant correlations between grouted volume and all the tested parameters. The best correlations were between grouted volume and average grouting time of the holes and percentage of HJ holes, which also represent the only positive correlations. The correlation with average maximum pressure and average pressure is negative, which indicate less grout consumption, if the pressure is higher. This is not compliant with the theory about high pressure and high grout consumption, but it reflects the chief stop criterion. The operator strives to reach the target pressure and terminates the grouting when the target pressure is met. If the target pressure is hard to reach, or cannot be reached, the grouting will continue for a longer period, which results in increased grout consumption.

The correlation between grouted volume and percentage of holes with no grout take is negative, as expected. Looking at the selection with only OPC rounds (b), the same trends are seen, but some of the R-values have decreased, particularly in the correlation between grouted volume and percentage of HJ. Looking at the correlation matrix for MFC rounds (c), it is a clear change in the correlations and significance. It is a strong and significant correlation between grouted volume and the percentage of HJ holes and average time use, while the other three parameters compared to grout take are not significant. At the same time the correlation between percentage of HJ holes and percentage of holes with no grout take, is relatively similar with the first two data selections (a) and (b). This indicates that when grouting with MFC, the grout take is high when there is many HJ holes, which coincides with the findings

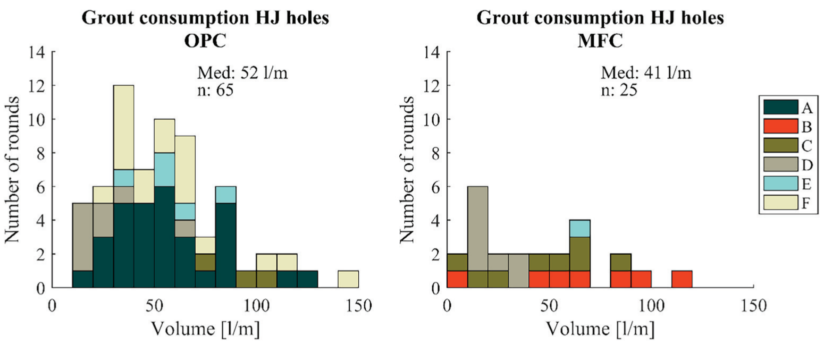


**Fig. 12.** Histograms showing frequency distribution of the grout consumption per metre of drill hole per round, in holes where HJ is detected; rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

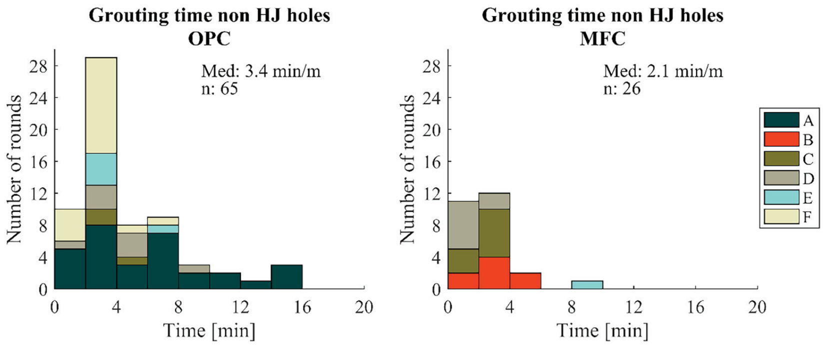


**Fig. 13.** Histograms showing frequency distribution of the time usage per metre of drill hole per round, in holes where no HJ is detected; rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

in Fig. 5. In the correlation matrix with data from Tunnel A (d), where only OPC is used, the only variable that have significant correlation with grout consumption is average time spent on each hole in a grouting round.

### 3.4. Results for histogram comparisons

Fig. 8 shows two frequency histograms comparing grout consumption using OPC and MFC. The median for OPC is 26,000 L per round of grouting, while the median for MFC is 13,000 L. For OPC the histogram shows approximate normal distribution, while for MFC it shows two clusters. In the cluster with least grout consumption, using MFC, the average percentage of HJ holes is 11%, while in the cluster with highest grout consumption the average percentage of HJ holes is 30%. If a similar comparison is done with the OPC data, the average percentage of HJ holes in rounds with a grout consumption less than the median value is 24% and above it is 26%. This shows a different trend for grout consumption using OPC compared to using MFC and confirms the results from the correlation analysis presented in Fig. 7, where there is a strong significant correlation between grout consumption and percentage of HJ holes in rounds grouted with MFC. The results indicate that in general the grout consumption using OPC is higher than the grout consumption using MFC. This result might seem surprising and will be discussed in Section 4.

Fig. 9 shows a comparison between percentage of holes with no grout take per round of grouting using OPC versus MFC. The median for OPC is 21%, while the median for MFC is 35%. For OPC the distribution is slightly right skewed and for MFC it is close to a normal distribution. This result indicates a higher occurrence of holes with no grout take using MFC.

Fig. 10 shows a comparison between percentage of HJ holes in each round of grouting using OPC versus MFC. The median for OPC is 24%, while the median for MFC is 19%. For OPC it is an approximate normal distribution, while for OPC it is a tendency towards two clusters, as in the results in Fig. 8. In general, the results indicate a higher occurrence of HJ using OPC.

Fig. 11 shows a comparison between grout consumption per meter hole in holes where HJ is not indicated, using OPC versus MFC. The median for OPC is 29 l/m, while the median for MFC is 17 l/m. For both OPC and MFC the distribution is right skewed. The results indicate that in general the grout consumption using OPC is higher than the grout consumption using MFC, in non HJ holes.

Fig. 12 shows a comparison between grout consumption per meter hole in holes where HJ is indicated, using OPC versus MFC. The median for OPC is 52 l/m, while the median for MFC is 41 l/m. For OPC the distribution is right skewed and MFC show a plateau distribution. In general, the grout consumption in HJ holes using OPC is slightly higher than the grout consumption in HJ holes using MFC. Comparing Figs. 11 and 12, there is a considerable increase in grout consumption in holes where HJ is indicated, compared to holes where no HJ is indicated. The increase is 79% for OPC and 141% for MFC.

Fig. 13 shows a comparison between average usage of time per meter hole in holes where HJ is not indicated, using OPC versus MFC. The median for OPC is 3.4 min, while the median for MFC is 2.1 min. For both OPC and MFC the distribution is right skewed. In general, the usage of time with OPC is higher than the usage of time with MFC in holes where no HJ is detected.

Fig. 14 shows a comparison between usage of time per meter hole in holes where HJ is indicated, using OPC versus MFC. The median for OPC is 6.4 min, while the median for MFC is 4.7 min. For both OPC and MFC the distribution is right skewed. In general, the usage of time with OPC is higher than the usage of time with MFC. Comparing Figs. 13 and

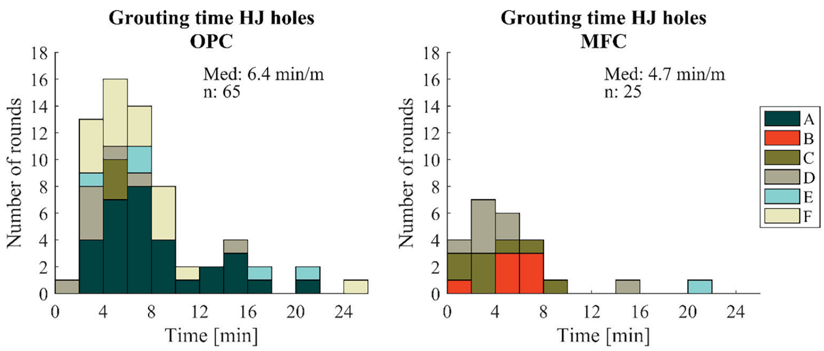


**Fig. 14.** Histograms showing frequency distribution of the time usage per metre of drill hole per round, in holes where HJ is detected; rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

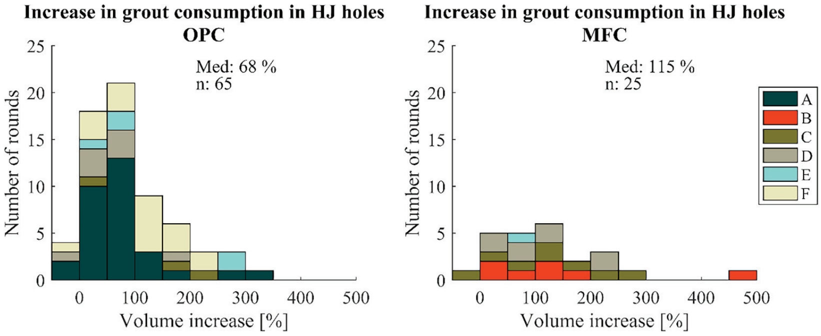


**Fig. 15.** Histograms showing frequency distribution of the increase in grout consumption per round, in holes where HJ is detected. Rounds grouted with OPC to the left and rounds grouted with MFC to the right. The colour grading identifies which tunnel the rounds represent. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

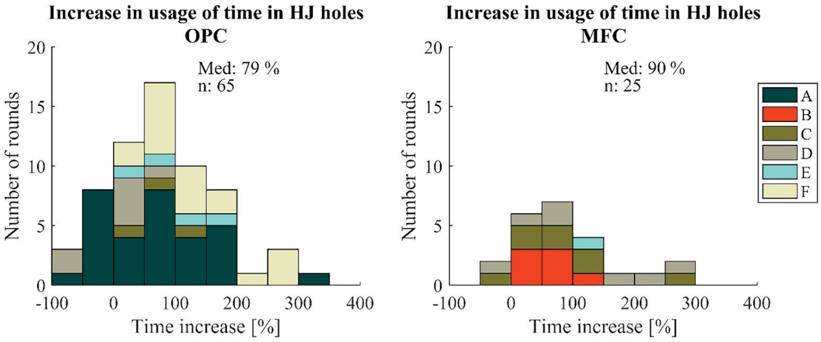


**Fig. 16.** Histograms showing frequency distribution of the increase in usage of time per round, in holes where HJ is detected. Rounds grouted with OPC on the left side and rounds grouted with MFC on the right side. The colour grading identifies which tunnel the rounds represents. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

14, there is a considerable increase in usage of time in holes where HJ is indicated, compared to holes where no HJ is indicated. For OPC the increase is 88%, while for MFC it is 123%.

Fig. 15 shows a comparison of increase in grout consumption in holes where HJ is indicated versus holes where no HJ is indicated in one round of grouting, using OPC versus MFC. The median for OPC is 68%, while the median for MFC is 115%. These results illustrate that the grout consumption is severely higher in holes where HJ is indicated. For both OPC and MFC the distribution is right skewed.

Fig. 16 shows a comparison of increase in usage of time in holes where HJ is indicated versus holes where no HJ is indicated in one round of grouting, using OPC versus MFC. The median for OPC is 79%, while the median for MFC is 90%. These results illustrate that the grout time is severely higher in holes where HJ is indicated. For both OPC and MFC the distribution is right skewed. Comparing Figs. 15 and 16, it can be observed that for OPC the increase in grout consumption and usage of time is relatively similar, but higher for time usage. For MFC the increase in time usage is less than for the increase of grout consumption, which indicate a higher flow rate in HJ holes grouted with MFC.

### 3.5. Summary of results

The main trends found in the results of this study are as follows:

- In rounds with MFC there is a strong positive correlation between grout consumption and percentage of HJ holes in a round, while for OPC no significant correlation has been found.
- Grout consumption and usage of time is higher in holes grouted with OPC than in holes grouted with MFC in holes where HJ is not indicated.
- There is a large increase in grout consumption for both OPC and MFC in holes where HJ is indicated, but the relative increase in holes grouted with MFC is considerable higher.
- In holes where HJ is indicated, there is a large increase in grouting time for both OPC and MFC, but the increase in holes grouted with MFC is relatively larger.
- For OPC, the increase in grout consumption is lower than the increase in time usage. For MFC the increase in grout consumption is higher than the increase in time usage, indicating a higher flow rate in HJ holes grouted with MFC.
- There are generally more HJ holes in rounds grouted with OPC, than in rounds grouted with MFC.
- There are generally more holes with no grout take in rounds grouted with MFC, than in rounds grouted with OPC.

## 4. Discussion

The study shows that a small share of the grout holes in a round of pre-grouting account for a large part of the grout consumption and a high prevalence of HJ has been found in holes with high grout consumption. The grout consumption has shown to be dependent on the stop criterion based on reaching a target pressure. If the resistance against grout take is high, the pressure builds up fast and the grouting is terminated after a short time of grouting. This is the cause of the positive correlation between grout consumption and usage of time and the negative correlation with high grouting pressure (Fig. 7).

The results show that there is generally less grout take using MFC, compared to OPC. The resistance to MFC grout seems to be higher, since the target pressure is reached in a shorter time, resulting in less grout consumption. There are also more grout holes with no grout take using MFC, than with OPC. On the face of it one should think that cement with high degree of fineness would be able to penetrate finer fractures and fill a larger portion of the void space in the fractures, resulting in higher consumption of grout and a slower pressure build-up. This does not appear to be the case in this study. According to the theory presented in Section 2.4 there could be several reasons for this result;

1. Poorer penetrability of fine grained grout. The MFC grouts used in study had a $d_{95}$ close to 20 µm, for this reason it is not expected that this is the cause of the reduced grout consumption for MFC in this study, but it could be the explanation for more holes with no grout take for MFC.
2. Higher yield stress and plastic viscosity in fine grained cements: this could be a contributing factor, since higher frictional resistance within the grout would require higher pressure to move the grout. This leads to a more rapid pressure build-up and the target pressure is reached faster, resulting in less grout consumption.
3. Shorter setting time for MFC. This would change the yield stress and plastic viscosity of the grout and furthermore affect the penetrability and the pressure build-up.
4. If the setting of MFC starts as early as 30 min after mixing it is also plausible that the grout in some cases is stored too long in the agitators on the grouting rig prior to pumping, resulting in decreased penetrability, higher yield stress and higher plastic viscosity.

One of the most important findings is the large difference between OPC and MFC regarding grout and time consumption in non HJ holes versus HJ holes. When using MFC, there is a considerable larger relative increase in both grout consumption and usage of time. Also, the increase in grout consumption for MFC is considerable higher than the increase in grouting time, whereas for OPC the increase in grout consumption for HJ holes is lower than the increase in grouting time. This indicates that the flow rate is increased in HJ holes grouted with MFC. The reason for this is not known. The reason for this is not known but could be related to increased pressure build up in MFC.

Comparing grout consumption and geological conditions, no clear and significant correlation was found. One would assume that the geology would have significant impact on the grout consumption, but there could be several reasons for the lack of correlation in this study. One such reason could be little variation in the geology in the analysed rounds of grouting. Another reason could be that the rock mass classification is performed for large sections in the tunnel with the main purpose of designing rock support and the data are thus not suited for grouting purpose. Further, the geological mapping was performed in grouted areas and the grouting may have improved/changed some of the input parameters. It was concluded that to be able to correlate grout consumption with geology the assessment and classification of the rock mass should be performed before grouting and the data available in this study was not suitable for such analysis. The grout consumption in regard to rock mass properties was therefore not uncovered by this study.

Despite the finding of a strong connexion between HJ and grout consumption, it has not been possible to establish to what extent HJ contributes to the increase in grout consumption. High prevalence of HJ in holes with high grout consumption does not evidence that HJ is the only cause of high grout consumption. As discussed earlier, HJ is assumed to be more prone to happen if the grouted volume is large. At the same time, it is little doubt that HJ increases the fracture volume, which also leads to an increase in grout consumption. The significance of HJ due to high grout consumption needs to be further investigated to fully understand the connection between HJ and grout consumption. To reveal to what degree HJ contributes to an increase in grout consumption, it would be beneficial to study grouting with HJ of fractures in single drill holes in ungrouted rock mass. This type of study would require a test site, not a study of data from pre-grouting during tunnel construction.

The question regarding whether HJ should be avoided or not is not clearly answered from the results in this study, but some assumptions can be deducted. HJ of fractures during grouting increases the volume of the fractures, which can result in excessive grout consumption. Also, HJ of fractures during pre-grouting is irreversible due to the particles in the grout, which could lead to an increase in conductivity along the perimetry of the grout spread. The aperture increase in the HJ fracture could lead to confinement of smaller fractures, resulting in less grout spread in smaller fractures. These arguments suggest that HJ has a negative impact on project economy, environment and tightness.

In some cases, it is also argued that HJ is necessary to achieve the required tightness. As discussed in Sections 2.3 and 2.4, rock fractures have a wide variety in characteristics, which result in diverse flow patterns and different grades of cement infilling. In fractures where asperities are isolated elements, there is no need use HJ as a tool to grout the narrow zones close to the asperities, because these zones will not transport water due to the lack of connectivity. In fractures with high connectivity between asperities with narrow water bearing zones, HJ could be beneficial to reduce the potential pathways for water. Such conditions could be found in fractures with gouge materials, and crystallization. By this line of reasoning it can be concluded that in most cases HJ does not appear to be necessary to achieve the designated inflow requirements.

The main goal of pre-grouting in Norway is to reduce the water ingress, according to an allowable inflow rate, “tight enough for its purpose”. The purpose of pre-grouting is not to make the tunnels waterproof but to safeguard the environment above and inside the tunnel. This should be performed in a cost- and time-effective way, with least possible impact on the local and global environment. The most solid arguments for avoiding HJ of fractures are related to its strong correlation to high consumption of grout and increased grouting time, which both affect project costs. In addition, high grout consumption has a significant negative effect on global carbon footprint.

## 5. Conclusion

This study has investigated parameters that have an impact on grout consumption during pre-grouting in Norwegian tunnelling, with special attention to incidents of hydraulic jacking of fractures (HJ). The occurrence of HJ was determined by using the algorithm described in Strømsvik et al. (2018). The factors correlating with grout consumption in this study were found to be hydraulic jacking, grouting time, type of grout and the chief stop criterion.

The main findings of this study are as follows:

- There was found a significant positive correlation between grout consumption and percentage of holes with hydraulic jacking per round of pre-grouting, especially when grouting with MFC.
- There was a general increase in grout and time consumption in holes where hydraulic jacking was indicated, especially when grouting with MFC.
- The general grout consumption was higher when using OPC, than when using MFC. This might be explained by higher yield stress and plastic viscosity and/or shorter setting time for MFC.
- Overall, there are more holes with no grout take in rounds grouted with MFC, than in rounds grouted with OPC. This might be due to poorer penetrability of fine grained grout.
- Hydraulic jacking appears more frequently when grouting with OPC than when grouting with MFC.

Hydraulic jacking of fractures during pre-grouting does not tend to be a good choice for the project economy, since it is correlated to longer grouting time and higher grout consumption. This particularly applies when using MFC. A successful grouting project is a fine balance between economy and final results. Hydraulic jacking or not should be evaluated for each individual project and the project owners should be aware that hydraulic jacking could lead to unnecessary usage of cement and time, especially when using MFC. Hydraulic jacking during pre-grouting in Norwegian projects ought to be closer monitored in the future and more research related to this issue should be performed.

## Acknowledgements

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# Paper III

# Investigation and assessment of pre-grouted rock mass

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**Abstract**

Pre-grouting is at technique for reducing water ingress into tunnels and caverns by grouting fractures and joints prior to excavation. It is a costly and time-consuming operation and for this reason it is of interest to estimate water inflow to predict when pre-grouting is needed and to optimize the grouting procedure for each site. This study investigates pre-grouted rock mass to evaluate grout spread and transmissivity of water in the rock mass surrounding the built tunnel. The study was performed in three tunnel localities, in tunnels excavated in connection with The Follo Line project in Norway, where pre-grouting was performed using cement based grouts. It was found less cement than expected in the fractures, compared to type of grout, grout consumption and use of grouting pressure, and that fractures in coarse grained rock types had higher hydraulic apertures and rougher fracture surfaces, than fractures in fine grained rock types. It was also found that fractures with smoother surfaces had smaller hydraulic apertures in general.

Keywords: Pre-grouting, hydraulic transmissivity, hydraulic aperture, grout spread

## 1 Introduction

Tunnels have different requirements regarding allowable water inflow, depending on the use and location. Often, the most important factor when determining the allowable water inflow is safeguarding of the environment above the tunnel. In Scandinavian tunnelling, most of the tunnels are excavated beneath the groundwater level. If the area above has infrastructure, residential buildings, agriculture, vegetation or lakes, it is important to ensure that the groundwater level is not lowered to a level that could have negative impact at the surface.

Pre-grouting, by some termed pre-excavation grouting, is a technique for reducing water ingress into tunnels and caverns, by grouting fractures and joints before excavation. The typical setup is to drill 25-70 holes of 15-30 metres length at the face of the tunnel, depending on the tunnel face area and geology. Packers attached to grouting rods are placed approximately 2 metres into the drill holes. The grouting rigs commonly have 3-4 grout lines which can operate simultaneously as illustrated in **Fig. 1**. The grouting is first performed in the bottom holes, moving upwards. In Norwegian tunnelling the most common grout types are cement-based grouts of different grades of fineness and with additives.

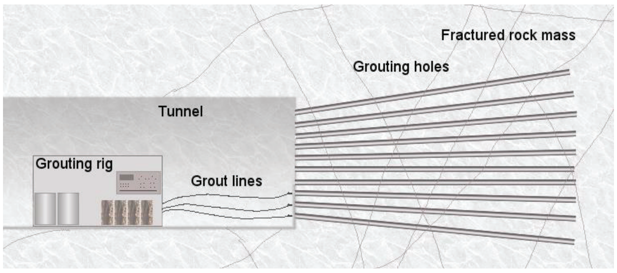



**Fig. 1** Illustration of pre-grouting performed in a tunnel during excavation.

Pre-grouting is a costly and time-consuming procedure and for this reason it is of interest to predict water inflow to plan when pre-grouting is needed. Also, it is important to find the most optimal grouting procedure regarding grout materials, grouting pressure, grout consumption and design of the pre-grouting, e.g. number and length of grouting holes and overlap between rounds of grouting.

In this study pre-grouted rock mass has been investigated to evaluate grout penetration in fractures and transmissivity in the rock mass surrounding the built tunnel. The study includes investigation and assessment of three locations in tunnels excavated in connection with The Follo Line project in Norway. The project is a twin tube railway tunnel connection between Oslo and Ski. The tunnels are excavated by using four TBM's, all starting from an adit at Åsland, which is located in the middle of the two main tunnels. An overview of parts of the construction site at Åsland, consisting of two TBM assembly halls(AH) and a network of tunnels; two access tunnels, two transport tunnels (TT) and a permanent access tunnel (PAT), is presented in **Fig. 2**.

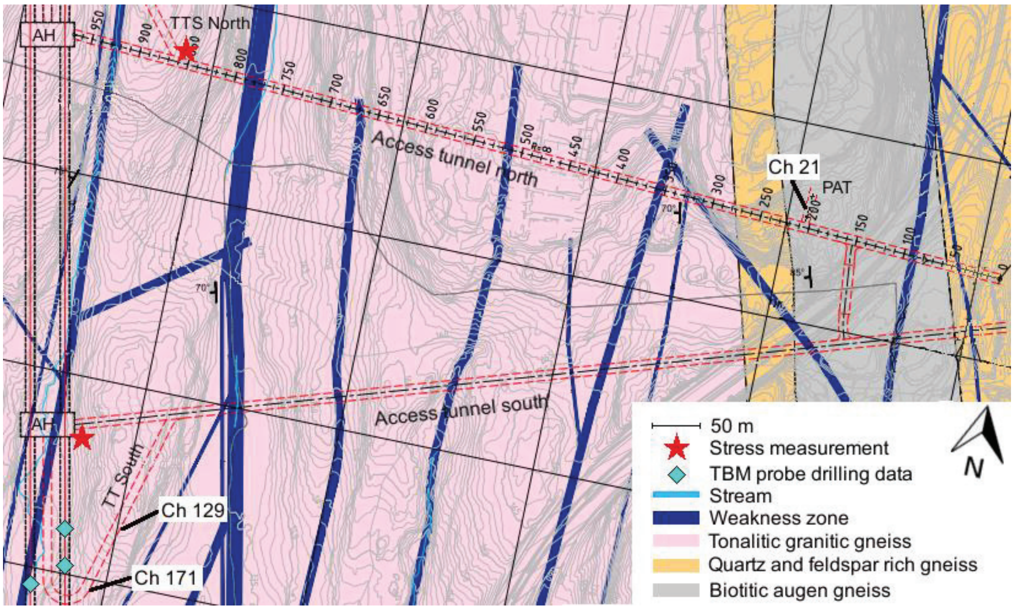



**Fig. 2** Overview of geology and tunnels at Åsland, modified from FPS AS (2014).

Three locations were chosen for this study; TT South, chainage 171 and 129 and PAT, chainage 21. In the following, they will be referred to as Ch. 171, Ch. 129 and Ch. 21. The locations are shown in **Fig. 2**. For each location three holes of 10 metres length with a nominal diameter of 76 mm were drilled, one in the tunnel roof, one in the springline and one in the wall, as illustrated in **Fig. 3**.

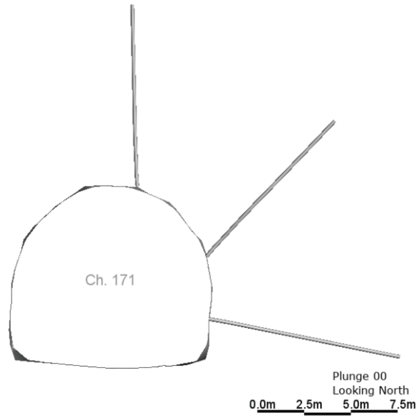



**Fig. 3** Positioning of test holes.

The tunnels investigated in this study are excavated with drilling and blasting. The area above the tunnels is urban, with roads, residential buildings and vegetation. The water inflow restrictions in these tunnels therefore were strict during as well as after construction and continuous overlapping rounds of pre-grouting were performed in all of the tunnels at Åsland. The groundwater level at all three sites are close to the ground surface but fluctuate throughout the year, depending on the season. The testing was done during spring, with high groundwater levels. The overburden at Ch. 171, Ch. 129 and Ch. 21 are 77, 84 and 65 metres, respectively.

Optical Televiewing (OTV), high precision water pumping and core logging was performed in all test holes. The main goal of the study was to identify which fractures the grout had penetrated, how much of the fractures in the surrounding rock mass was grouted and to evaluate the transmissivity of water in the grouted rock mass.

## 2 Background theory

### 2.1 Permeability in rock mass and groundwater flow

Groundwater flows through the rock mass in fractures and this flow can be described in various ways. In principle, the water follows the path of least resistance, i.e. where the aperture is greatest, resulting in different flow distribution within each fracture. Transmissivity is a measure of how much water that can be transmitted over time. Transmissivity of water in a slot is proportional to the cube of the slot aperture and follows the cubic law. This imply that a small change in the fracture aperture will have great impact on the transmissivity of a fracture. One method for estimation of transmissivity is presented in section 5.2.

The varying fracture aperture and flow distribution makes measurement of flow difficult, therefore hydraulic aperture is used in many cases. The hydraulic aperture is the aperture of a fracture that would give the same mean flow as the actual aperture.

According to Gustafson (2012) the hydraulic aperture can be calculated as follows:

$$b = \sqrt[3]{\frac{12\mu T_f}{\rho g}} \qquad (1)$$

Where $b$ is hydraulic aperture, $\mu$ is viscosity of water, $T_f$ is transmissivity, $\rho$ is density of water and $g$ is the gravitational constant.

When a drill hole intersects a fracture, the visible contour of the fracture at the intersection will give brief information about a fracture. As illustrated in **Fig. 4**, drill holes a) and b) intersect the same fracture, but the measured aperture is very different. When pumping water into these two drill holes, the transmissivity still is expected to be roughly similar. Drill holes c) and d) both intersect a fracture with a smaller aperture, but the measured apertures at the intersections are similar or slightly larger than in drill hole b). In these two drill holes the transmissivity is expected to be smaller than in drill hole a) and b). This example illustrates that the measured aperture at the intersection between a drill hole and a fracture is not likely to represent the hydraulic aperture or the mechanical aperture of the fracture. The mechanical aperture is the actual fracture aperture.

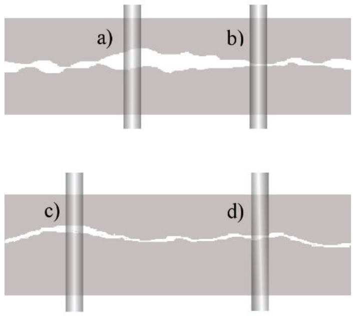



**Fig. 4** Drill holes intersecting fractures, illustrating that the visible part of a fracture in a drill hole does not realistically reveal the fracture geometry.

According to Barton and de Quadros (1997) experiments have shown that the roughness and aperture of a rock joint are the most important factors governing fluid flow through fractures. Li et al. (2008) found that rough fractures exposed for shear displacement of 4-16 mm, experienced a significantly increase in both hydraulic and mechanic apertures compared to the same shear displacement in smoother fractures. The reason for this is that in rough fractures the asperities tend to climb over each other during shear, which decrease the contact ratio between asperities and create a larger void space, compared to smother fractures. This implies that the stress distribution in combination with fracture roughness have great influence on the transmissivity of the fractures.

In Fennoscandia the in-situ stress conditions in the rock mass are complex with horizontal stress normally exceeding the vertical stress. The origin of the high horizontal stress is presumed to be a combination of ridge push from the Mid-Atlantic Ridge and rapid unloading of the surface du to erosion and deglaciation (the Holocene glacial retreat). Both these events are ongoing processes (Stephansson et al., 1991). The ridge push will cause a regional stress field in which the maximum horizontal stress will act NW-SE in central Fennoscandia and affects the stress distribution in the uppermost 1000 metres of rock mass (Stephansson et al., 1991). The change in stress distribution caused by rapid removal of the overburden, have in many areas resulted in the development of tensional fractures approximately parallel to the surface (exfoliation fractures). These rapidly decrease in frequency with depth. These types of fractures have an important role in the interconnection between fractures, which could increase the hydraulic conductivity in the rock mass (Gudmundsson et al., 2002).

In addition to the two discussed factors affecting todays stress situation in Norway, there are local historic events affecting both the stress distribution and the distribution and orientation of fractures and faults in the rock mass. In the Oslo region, which is the setting for this study, the Oslo Graben is such an event. The Oslo Graben is a N-S-trending Carboniferous-Permian rift system, characterized by N-S-trending faults, reactivation of pre-existing Precambrian faults and the formation of half grabens (Heeremans et al., 1996). This event could in combination with today's stress field, also affect the hydrogeology in the area.

Origin, classification and characteristics of rock mass fractures can vary greatly, as described by Palmstrøm (2015) and Gustafson (2012). Besides stress and fracture roughness, fracture filling and rock types has an important role in regard to transmissivity of water. Most fractures are partly filled with rock fragments, secondary minerals, or minerals that have been precipitated from the flowing groundwater. The fracture infilling depends on the rock types, in combination with the tectonic history and groundwater composition. Gustafson (2012) describes a general trend suggesting that fine grained granite with a high $SiO_2$ content has higher hydraulic conductivity, while basic rock types e.g. greenstone has lower hydraulic conductivity. The explanation given for this is that dark basic rock types tend to have higher tensile strength, but lower modulus of elasticity, than acidic rock types. Therefore, acidic rock types tend to fracture more easily. Also, dark rock types decompose more readily, resulting in more fracture infill.

In summary, understanding of the hydrogeology in a rock mass it is important to get an overview over the in-situ stress and the geologic history in the area, in combination with the orientation of the fracture sets and the type of rock mass.

### 2.2 Grout spread in rock mass

Grout spread through fractures in a rock mass is governed by many of the same main principles as described in section 2.1 regarding water flow through rock mass. The grout flows along the lines of least resistance, i.e. where the aperture is greatest, resulting in different flow distribution within each fracture. The main difference is that cement-based grout is not a Newtonian fluid, but a particle-based fluid that can be described as a Bingham material, as described by i.e. Stille (2015). The most significant difference is that when pumping grout into fractures, the frictional forces in the fluid are significantly higher, resulting in pressure increase. Also, the grout is not able to penetrate the same fracture volume as water.

In Norwegian projects the main goal during pre-grouting is to fill fractures 5-6 metres beyond the profile of the tunnel (Aarset et al., 2011). During and after the grouting it is not possible to evaluate if this criterion is met or not. It is neither possible to determine how the grout is distributed, in regard to aperture of the fractures. The result of pre-grouting is determined by the degree of tightness after construction. This limits the knowledge and learning of where the grout is spreading in the rock mass and how the pre-routing can be optimized with regard to tightness, grout consumption and usage of time for each site.

According to Stille (2015) the ability of grout to penetrate fine fractures (penetrability) depends on relationship between the size of the grains and fracture apertures. In fine-grained cement this relationship is complex, mainly due to an increase in specific surface area, resulting in greater surface activity. The penetrability is also affected by the water/cement ratio (w/c ratio), cement quality, type of mixer and temperature. Stille (2015) describes laboratory tests indicating that INJ30 cement has a critical aperture of 90-157 μm, dependent on water/cement ratio (w/c ratio), type of mixer and temperatures. The critical aperture is defined as the aperture sufficiently large for free grout flow, with no filtering.

When grouting rock mass fractures with the use of high pressure, hydraulic jacking (HJ) could occur. HJ occurs when the pressure inside the fracture is higher than the normal pressure acting on the fracture. This force makes the fracture open, which means that the aperture of the fracture is increasing. A detailed discussion of this process can be found in Stille (2015) and Strømsvik et al. (2018).

Stille (2015) draws attention to the following negative consequences of HJ during pre-grouting:

1. Higher consumption of grout, due to higher flowrate and increased volume of fractures.
2. Uplift of the overburden, if the fractures are close to horizonal oriented.
3. Increased transmissivity outside the grouted zone, due to increased apertures of fractures.
4. Finer factures can be exposed to compression during grouting, making them more difficult to grout.

In areas with low connectivity between fractures, HJ could give better penetration of grout.

## 4 Site description

### 4.1 Stress measurements

Rock stress measurements were conducted by SINTEF at two locations in the tunnels at Åsland, by using 3D-overcoring. The method of stress measurements is as described in Dahle and Larsen (2005). The results are presented in **Table 1** and **Fig. 5**, the locations are marked in **Fig. 2**.

**Table 1** Test results from 3D stress measurements at Åsland.

| | Test location north | | Test location south | |
|---|---|---|---|---|
| | Stress [MPa] | Orientation | Stress [MPa] | Orientation |
| $\sigma_1$ | 24,3±2,3 | N169º Dip: 3º | 24,5±2,4 | N90º Dip: 7º |
| $\sigma_2$ | 14,6±2,3 | N78º Dip: 3º | 15,7±3,1 | N182º Dip: 18º |
| $\sigma_3$ | 11,8±2,3 | N304º Dip: 86º | 10,3±1,1 | N339º Dip: 70º |

The stress measurements showed stress vectors significantly higher than theoretical estimations based on the overburden in the area. The high horizontal stress is likely to originate from å combination of tectonic stress from both the Mid-Atlantic Ridge push and rapid unloading of the surface du to erosion and deglaciation. It can be noted that $\sigma_1$ rotates 79º in only 400 metres, the assumed reason for this is the close presence of major weakness zones, as illustrated in **Fig. 2**. Despite the large change in the direction of the major principal stress, the dip is still close to horizontal. Ch. 171 and Ch. 129 are approximately 120 and 160 metres from test location south, while Ch. 21 is approximately 650 metres from test location north.

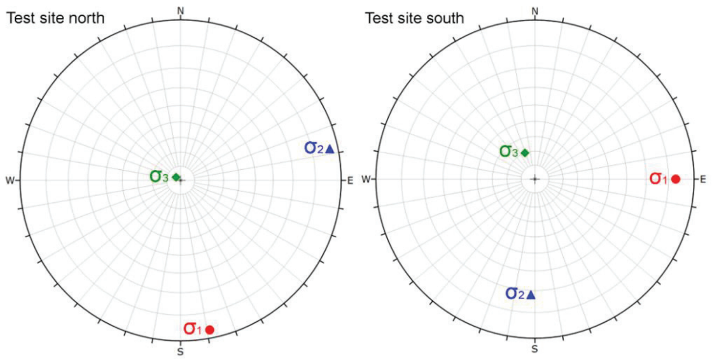



**Fig. 5** Dip vectors of stress measurements, in lower hemisphere equal angle stereonets. Test location north on the left and test location south on the right.

### 4.2 General fracture distribution and zones of weakness

To get an overview of the general fracture distribution at Åsland, data from probe drill holes in front of two TBM's passing close to the test locations were evaluated. **Fig. 6** a) shows fractures logged by OTV of probe drill holes from TBM at three locations, marked in **Fig. 2**. The probe holes are drilled roughly parallel to The TBM tunnel alignment, which has an approximate direction of 170º SSE. It can be noted that these drill holes are oriented about 80º different in strike direction than the test holes placed in the wall and springline in this study. The stereonet shows that the major fracture set is vertical, with strikes towards E and W and dips of 70º-90º. Since the TBM probe drillings are oriented horizontal they would not reveal potential presence of subhorizontal fractures. For this reason, it was chosen to look at well holes drilled in a close to vertical direction.

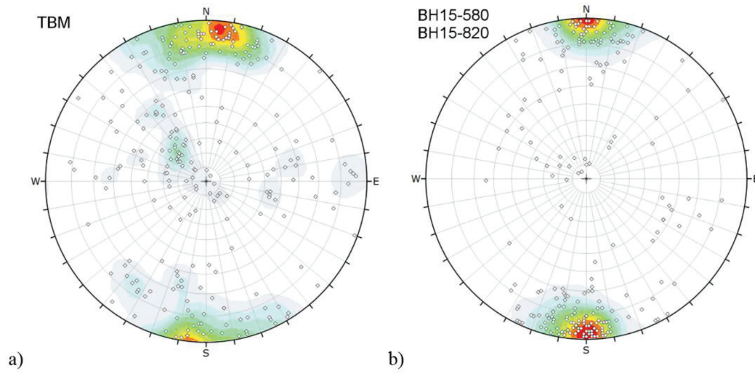



**Fig. 6** Fractures plotted as poles in lower hemisphere equal angle stereonets. a) probe drill holes in front of two TBM's, b) two well holes drilled close to the tunnel alignment.

The closest place for such data was two well drillings performed 3 km SSE for the TBM probe drillings, placed close to the tunnel alignment. The locations for the well drillings are relatively far away from the test locations, but is still within the same region as described regarding regional stress distribution and fracture distribution in section 2.1. **Fig. 6** b) shows fractures logged by OTV of two well holes drilled close to the tunnel trace (BH15+820 and BH15+580). BH15+820 is a close to vertical hole of 70 metre length, with 10.5 metres deviation from vertical direction. BH15+580 is 150 metres long, with a 50º dip towards North. These drill holes are oriented in a direction that would intersect subhorizontal fractures. It can be observed that the major fracture set is similar with the finding from the OTV probe drilling, with strikes towards E and W and dips of 70º-90º. There are relatively few horizontal fractures.

As shown by **Fig. 2**, the Åsland area has many N-S trending weakness zones/faults belonging to the regional, N-S-trending Carboniferous-Permian rift system. Four subhorizontal weakness zones were also found in this area, with crushed, weathered and clay rich material. The origin of these weakness zones is not known to authors of this study, but they are possibly related to the Caledonian Orogeny.

### 4.3 Grouting works

The test holes at Ch. 171, Ch. 129 and Ch. 21 are drilled in pre-grouted rock mass. The grout rounds performed at these locations are presented in **Table 2**, with hole length, number of holes, chief stop criterium and which test location the rounds belong to. **Table 3** shows type of cement and grout consumption.

All rounds were grouted with Portland cement of different degrees of fineness. In this study, Micro Fine Cement (MFC) is defined as Portland cement with a $d_{95}$<25 µm, Ordinary Portland Cement (OPC) is cement with a $d_{95}$=<40 µm and >25 µm.

**Table 2** Design of grout rounds and target pressure in the stop criterium.

| Chainage | Holes | Length | Target pressure | Test location |
|---|---|---|---|---|
| 155→181 | 29 | 26 | 60 bar | Ch. 171 |
| 164→182 | 28 | 18 | 60 bar | Ch. 171 |
| 105→131 | 30 | 26 | 60 bar | Ch. 129 |
| 10→37 | 30 | 27 | 80 bar | Ch. 21 |

**Table 3** Grout type and grout consumption per round.

| Chainage | Grout | w/c ratio | Additives | Consumption |
|---|---|---|---|---|
| 155→181 | MFC | 1.0/0.8/0.6 | Superplasticiser | 102 996 kg |
| 164→182 | MFC | 1.0 | Superplasticiser | 7 625 kg |
| 105→131 | MFC | 1.0/0.8/0.6 | Superplasticiser | 43 218 kg |
| 10→37 | OPC/ Zugol | 0.9/0.5 | Silica slurry and superplasticiser | 43 152 kg |

At Ch. 171, two rounds of grouting overlapped at the test location. The round starting at chainage 155 was grouted before the round starting at chainage 164. For Ch. 129 and Ch.21 there was only one round of grouting at each of the test locations.

When using MFC the grouting at each hole was always started using w/c ratio 1.0, after grouting 900-2000 litres without reaching the target pressure of 60 bar, the w/c ratio was reduced to 0.8. If the target pressure still was not reached after grouting additionally 900 litres, the w/c ratio was reduced to 0.6.

OPC was used in the grout round at chainage 10. Each hole was always started using w/c ratio 0.9, after grouting approximately 1200 litres without reaching the target pressure of 80 bar, the w/c ratio was reduced to 0.5. In 7 holes it was added Zugol to the grout, Zugol is granulated natural fir tree bark. The Zugol was added because of large water bearing fractures in the surrounding rock mass and high connectivity between the drill holes. In 8 holes accelerator was added after a long period of grouting, because of high consumption of grout.

The pressure and flow data from the grouting rigs were screened for occurrences of HJ. For the grout rounds at chainage 155 and 164 the algorithm described in Strømsvik et al. (2018) was used. It was not possible to perform such analysis on the grouting rounds at chainage 105 and 10. At chainage 105 the log interval was over 10 seconds and at chainage 10 the log from the grouting was not possible to locate. The data from the grout round at chainage 105 was analysed by visual inspection of the pressure flow charts, using the general principals suggested in Strømsvik et al. (2018).

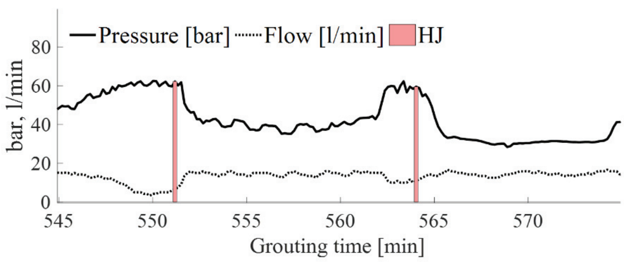



**Fig. 7** Example of HJ in hole 19 at chainage 155.

At chainage 155 indication of HJ was found in 11 of the grout holes. The jacking occurred at grouting pressures from 35 to 70 bar, most commonly 40-50 bar. **Fig. 7** shows an example of HJ at a grouting pressure of 60 bar in hole 19.

At chainage 164 there was indication of HJ in one grout hole, at 50 bar. In the grouting round at chainage 105 it was interpreted to be HJ in 6 holes. The jacking occurred at grouting pressures from 40 to 60 bar, most commonly 45 bar.

In many of the grout holes the grout consumption was approximately similar or less than the volume of the drill hole. This indicate that no groutable fractures were intersected or that the holes were filled with hardened grout from other holes grouted prior. This was the case for 31% of the holes at chainage 155, 64% of the holes at chainage 164, 65% of the holes at chainage 105 and 11% of the holes at chainage 10. Chainage 155 and 164 overlapped and the high number of non-groutable holes and the relatively lower grout consumption at chainage 164 is because the rock mass at this location was already grouted.

At Ch. 171 and 129 no specific inflow limit was set, but the requirement was that the groundwater level should not be permanently impacted by the tunnel. The groundwater level was closely monitored with wells at the ground surface. At Ch. 21 the water inflow limit was < 10 l/min per 100 metres of tunnel. At all three test locations the requirements regarding water ingress into the tunnel were met, and the grouting works was considered successful.

## 5 Investigation, testing and analysis of drill holes

At each location, three holes were drilled as illustrated in **Fig. 3**, and the following investigations and tests were performed:

- Optical Televiewing (OTV), performed by Geologin AS.
- High precision water injection tests in 0.5 m sections, performed by Geosigma AB.
- Core logging, performed by the main author.

Based on this, a 3D model implemented with data from OTV, water injection tests and core logging was made.

The investigation and test methods are presented in detail in the following.

### 5.1 Optical Televiewer

The purpose of OTV was to get exact orientation of the drill holes and orientation and apertures of all fractures intersected by the drill holes. The approximate apertures for all fractures were measured from the OTV pictures, as illustrated in **Fig. 8**. It was not possible to evaluate a precise apertures of fractures less than 1 mm.

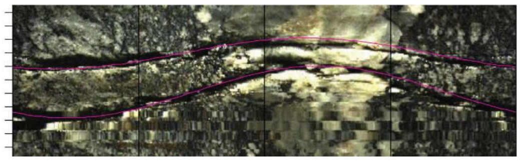



**Fig. 8** Two large fractures with an approximate aperture of 6 mm, located in OTV pictures. The spacing between each notch to the left represents 1 cm.

**Fig. 9** a) shows the OTV picture of a subhorizontal fracture with an aperture of approximately 8 mm filled with cement, **Fig. 9** b) shows the same fracture from the drill core. The aperture estimated by measuring on the OTV picture, matches the thickness of the cement found in the core. It can be noticed that there is three layers of cement with different colour. The reason for this layering is unknown, but it could be due to several reasons; such as HJ of the fracture, pauses during grouting of a hole, or that the fracture is intersected by more than one grout hole.

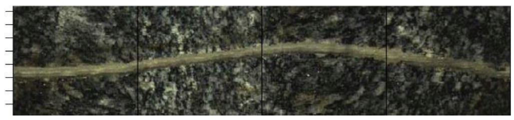
a)  b) 

**Fig. 9** a) large cement filled fracture seen in the OTV. The spacing between each notch to the left is 1 cm. b) the same fracture located in the core. Estimated aperture of the fracture is 8 mm.

### 5.2 Method for high precision water injection tests.

The high precision water injection tests were conducted by Geosigma AB with their Water Injection Controller (WIC). The water pump has PLC based automatic control system, equipped with data acquisition system, flow metres and pressure transducer. The minimum and maximum measurement limits are 5 ml/min and 65 l/min. Test sections of 0.5 metres were isolated by two individually operated hydraulic packers of 0.45 metres length, which were pressurised with water.

Single packer tests were performed when it was not impossible to expand both packers due to large fractures or cavities and in the end of each drill hole. The packers could not be expanded over a fracture with an aperture of more than 2 cm. The water pressure during the water injection tests were 5 bar over the natural formation pressure, the duration of the water injection was approximately 10 minutes of steady pressure. If the flow rate was below the measurement limit of 5 ml/min, the test was aborted. In total, 149 high precision water injection tests were performed in this study.

Prior to the investigations, mechanical packers and pressure gauges were installed in the drill holes to measure the formation pressure. The formation pressure used in the calculations was a combination of measured pressure and estimated pressure. It was assumed that the initial pressure did not vary significantly within the relatively short drill holes, which was verified by measuring in two different sections in some of the holes.

Leakage from the drill holes from both sides of the packer sections was measured before and during injection tests, to assess the possible impact from interconnection of fractures in the test section and outside the test section.

The hydraulic transmissivity was estimated in accordance with Moye's formula (Moye, 1967), shown in Equation 2.

$$T_M = \frac{Q_P * \rho_W * g}{dP_P} * C_M \qquad (2)$$

$$C_M = \frac{1 + \ln\left(\frac{L_W}{2r_W}\right)}{2\pi} \qquad (3)$$

$T_M$ = hydraulic transmissivity ($m^2/s$)
$Q_p$ = flow rate at the end of the flow period ($m^3/s$)
$\rho_w$ = density of water ($kg/m^3$)
$g$ = acceleration of gravity ($m/s^2$)
$C_M$= geometrical shape factor (-)
$dP_p$= injection pressure Pp– Pi(Pa)
$r_w$= borehole radius (m)
$L_w$= section length (m)

Some of the planned test sections had to be adjusted, see **Table 4**.

**Table 4** Overview over test sections diverging from the test plan.

| Hole ID | Depth [m] | Comment |
|---|---|---|
| Roof 129 | 5.40-6.90 | Offset in packer placement due to large fracture. Large flow not measurable. |
| Wall 129 | 0.6-9.7 | No measurable transmissivity in the entire section. |
| Abut 129 | 0.00-2.55 | Double-drilled start of the hole.<br>Measurements could not be performed in the affected section. |
| Wall 21 | 4.55-5.70 | Offset in packer placement due to large fracture. |

**5.3 Core Logging**
In the core logging the following was emphasized:

- Verification of structures interpreted from OTV.
- Evaluating fracture fillings and presence of cement from pre-grouting.
- Measurement of Joint Roughness Coefficient (JRC).
- Rock type classification.

When assessing the drill cores, it was revealed that some of the fractures interpreted in the OTV analysis were structures and some of the interpreted structures were fractures. Also, non-interpreted fractures and fractures filled with cement were found. Corrections according to these findings was made. In test hole positioned in the wall at Ch. 171, a large dyke of amphibolite at 6 metres depth appeared to be massive without fractures in the OTV images, while the core was heavily fractured with fracture infill, indicating that some of the fractures were present before the drilling. Measurable transmissivity in this section confirms this. Picture of the core section is shown in **Fig. 10**.

**Fig. 10** Picture of a core section with heavily fractured amphibolite, each row of core represents 1 metre.

All the fractures found in the cores from each of the nine test holes were evaluated by visual inspection, defining the texture of the infill with wet fingertips, scraping with a hard object and dripping of hydrochloric acid. If there was no reaction with hydrochloric acid on the fracture surface, the presence of cement could be excluded, since the grout used was made from Portland clinker. The fracture fillings were categorized as following:

*Cemented fracture:* grey filling, non-slippery, relatively soft, strong reaction with hydrochloric acid.
*Trace of cement:* trace of grey/white material, non-slippery, soft, strong reaction with hydrochloric acid.
*No filling:* clean fractures, no reaction with hydrochloric acid.
*Fracture fill 1:* yellow/white, slippery, very soft, no reaction with hydrochloric acid (talc).
*Fracture fill 2:* yellow/white, non-slippery, hard crystallization, reacts with hydrochloric acid (calcite).
*Fracture fill 3:* rusty, non-slippery, grainy, no reaction with hydrochloric acid.
*Fracture fill 4:* green, slippery, soft, no reaction with hydrochloric acid, only in amphibolite (chlorite).

The joint roughness coefficient (JRC) is an empirical index used for surface roughness characterisation. According to Grøneng and Nilsen (2009) JRC can be estimated by using several different methods. For this study it was chosen to use a table for typical roughness profiles for JRC, presented by Barton and Choubey (1977). This method was evaluated to be the most appropriate due to the short fracture surface available from the core. The fracture surfaces were measured by using a contour gauge. It was challenging to conduct a good and reliable evaluation based on a surface of 50 mm at the smallest, but the method worked well to differentiate the roughens of the fractures evaluated within this study.

The evaluation of fracture filling and measurement of JRC, was in some cases problematic because of disturbance of the fractures during core drilling.

### 5.4 Methods for data interpretation and 3D model

By creating a 3D model, the results from OTV, water injection tests and core logging could be combined. The 3D software used for this purpose was Leapfrog Works 2.2. The following elements were implemented into the 3D model:

- Profile of the tunnel lining at each test location.
- Exact placement and direction of each test hole.
- All fractures with depth, strike, dip, filling, measured aperture and JRC.
- All sections of high precision water injection tests.
- Rock types.

The 3D model was essential for comparing different types of data, because of the large data set. The following data was extracted and are presented in this paper:

1. Number and orientation of all fractures.

2. Number and orientation of grouted fractures.
3. Hydraulic apertures compared to JRC.
4. JRC of fractures in different types of rock.
5. Distribution of rock types, fractures in rock types and permeability in rock types.
6. Permeability in all test holes.

To investigate the data in bullet point 3, above, it was necessary to estimate hydraulic apertures for specific fractures, since the measured aperture at the drill hole intersection is not likely to represent the actual fracture aperture, as described in section 2. **Fig. 11** illustrates a single fracture within a water injection test. The transmissivity, calculated by using Formula 2, represents the transmissivity for the entire test section, not the fracture. The transmissivity is dependent of the shape factor of the drill hole, estimated by using formula 3. When the hydraulic aperture is estimated, it is important to keep in mind that it is not estimated for the fracture, but for the test section. In this study it is chosen to assume that the hydraulic aperture for the test section is a good enough representation of the hydraulic aperture of the single fracture within the test section. In the combined 3D model, all single fractures placed within one section of water injection with measurable transmissivity was identified, and the approximate hydraulic apertures were calculated. If more than one fracture was present within a water injection section, it was not possible to estimate the hydraulic apertures for the individual fractures.

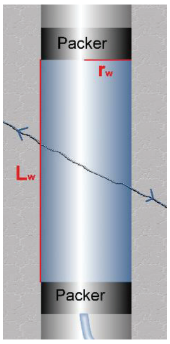



**Fig. 11** Water pumping section with a fracture.

For the fractures with measurable transmissivity a pairwise correlation analysis with Pearson's linear correlation and the statistical software R Studio (Dalgaard, 2002) has been performed. The parameters included in this analysis was calculated hydraulic aperture and JRC. The statistical significance level was set to a p-value $< 0.05$.

All stereonets presented in this paper are produced by using the software Dips from Rocsience.

**6 Results**

A total of 103 fractures were identified; 40 fractures at Ch.171, 24 at Ch. 129 and 39 at Ch. 21. The fractures for each location are plotted in stereonets, shown in **Fig. 12**. It can be noted that in all three test locations, the majority of the observed fractures are subhorizontal. In this regard it is important to bear in mind that the test holes are all oriented with an E-W, or vertical direction, which would largely underestimate steep fractures with similar strike direction. Both the TBM probe holes and the well holes indicate that the major fracture set is oriented E-W and with a close to vertical dip, which means that the major fracture set is not represented in this study. The consequence of this will be discussed in section 7.

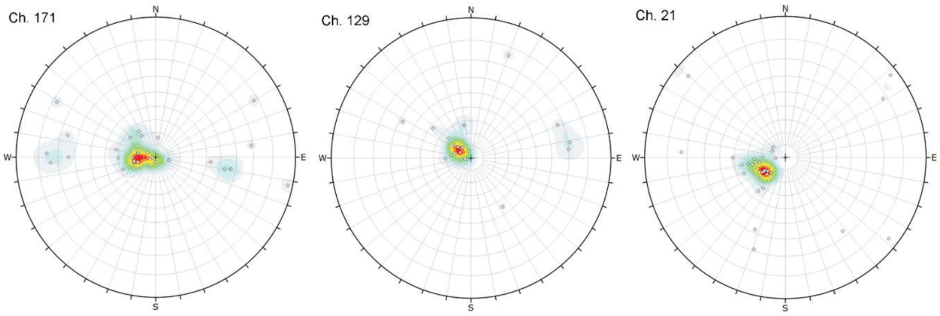



**Fig. 12** Pole vector projection of all fractures at each location, in lower hemisphere equal angle stereonets.

A total of 20 fractures were cemented (19%). Four of the cemented fractures had an approximate aperture of 1 mm, no smaller cemented fractures were found. None of the cemented fractures were evaluated to have measurable transmissivity. **Fig. 13** presents a stereonet of all the cemented fractures, where it can be observed that 17 of the 20 fractures are subhorizontal.

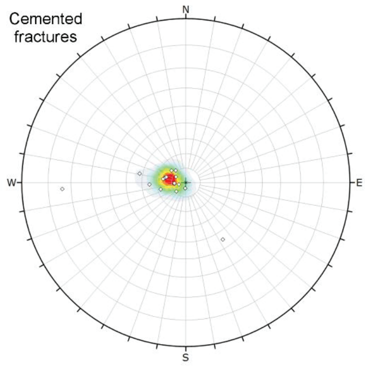



**Fig. 13** Pole vector projection of all cemented fractures in a in lower hemisphere equal angle stereonet.

**Table 5** shows percentage of fractures filled with cement. As expected a higher percentage of the fractures close to the tunnel profile are cemented, but at all three test locations, most fractures are not filled with cement.

Trace of cement was found in 6 fractures (6%). Four of the fractures with trace of cement was smaller than 1 mm, one fracture had an approximate aperture of 1 mm and one fracture had an approximate aperture of 2 mm. There was found measurable transmissivity in three of the fractures with trace of cement. Two of the fractures had no measurable transmissivity and one fracture was not possible to evaluate because of the presence of other open fractures in the same water injection test section.

**Table 5** Percentage of cemented fractures.

| Test location | Cemented fractures | |
|---|---|---|
| | 0-5 metres | 5-10 metres |
| Ch. 171 | 36% | 22% |
| Ch. 129 | 21% | 20% |
| Ch. 21 | 12% | 0% |

Percentage of grouted fractures is not as expected. It was found close to no grout in small fractures, which is surprising compared to expected penetrability for this type of grout.

13 structures which were interpreted as pre-existing fractures did not have measurable transmissivity. All these fractures had an aperture of less than 1 mm and most of these fractures had fracture fill type 2; calcite.

30 of all 103 fractures were single fractures placed within one section of water injection with measurable transmissivity. For this selection of 30 fractures, it was performed a pairwise correlation analysis between calculated hydraulic aperture and JRC, as described in section 5.4.

The results of the analysis are summarized in **Fig. 14**. The correlation coefficient is low, and there is no statistic significant correlation between hydraulic aperture and JRC. In this regard it is important to keep in mind that the surrounding rock mass is pre-grouted, and the transmissivity is most likely affected by this. By looking at the corresponding scatter plot, it can be observed that there is heteroscedasticity in the data. This means that the scatter has more spread in one end of the scale. When the JRC is low, which represents smoother fractures, the hydraulic apertures are generally in the smaller end of the scale. With increasing JRC the hydraulic apertures are in both ends of the scale, including both small and large hydraulic apertures. This effect is the cause of lacking statistical correlation, but it can still be concluded that it is a tendency towards smaller hydraulic apertures with low JRC.

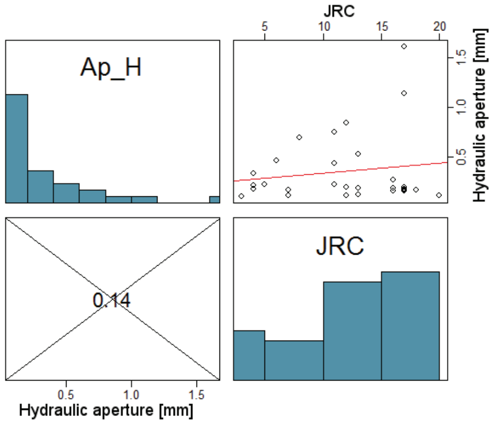



**Fig. 14** Results from the pairwise correlation analysis between hydraulic aperture (Ap_H) and JRC. The upper right shows scatterplot of the values and the lower left shows the correlation coefficient. The bar plots show the distribution of the two parameters.

The following rock types were encountered in the test holes: tonalitic gneiss (TTG), granitic gneiss (GG), supracrustal gneiss (SCG), amphibolite (A), garnet amphibolite (GA), pegmatite (PG) and poor pegmatite (PP). The classification of poor pegmatite was added because the pegmatite found in the upper part of the roof hole at Ch. 21 was of poor quality. The JRC value of the fractures were evaluated with regard to rock type. The median JRC for all the represented rock types is shown in **Fig. 15**. Some of the fractures were in the transition (Tr) between two rock types. In general, higher JRC values were found in tonalitic gneiss, granitic gneiss, pegmatite and garnet amphibolite and lower JRC values in amphibolite, supracrustal gneiss and fractures in the transition zones between two rock types. Most of the fractures found in the transition zones were between amphibolite and other rock types.

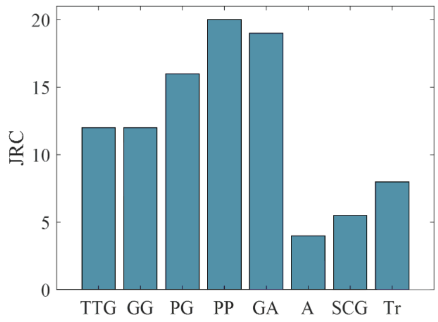



**Fig. 15** Median of JRC for rock types.

The average grain size of the rock types was measured to be as following: granitic gneiss 4 mm, tonalitic gneiss: 3.5 mm, pegmatite: 10 mm, poor pegmatite: 15 mm, garnet amphibolite 10 mm (garnet)/0.3 mm (amphibolite), supracrustal gneiss: 0.5 mm and amphibolite: 0.3 mm.

These results indicate that fracture surfaces in rock types with coarse mineral grains are rougher than fracture surfaces on rock types with fine mineral grains.

**Fig. 16** shows the distribution of rock types encountered in the drill holes at each test location, the distribution of fractures in each of these rock types and the distribution of calculated hydraulic apertures in each of the rock types. At all three locations tonalitic gneiss is the dominant type of rock.

At Ch. 171 the distribution of fractures between the rock types are roughly even, but there are relatively less fractures in the granitic gneiss, although the present fractures have a larger hydraulic aperture than the fractures in the other present rock types. Supracrustal gneiss and garnet amphibolite are more fractured than average, but the present fractures have smaller hydraulic apertures. At this locality the amphibolite has approximately average number of fractures, with average hydraulic apertures.

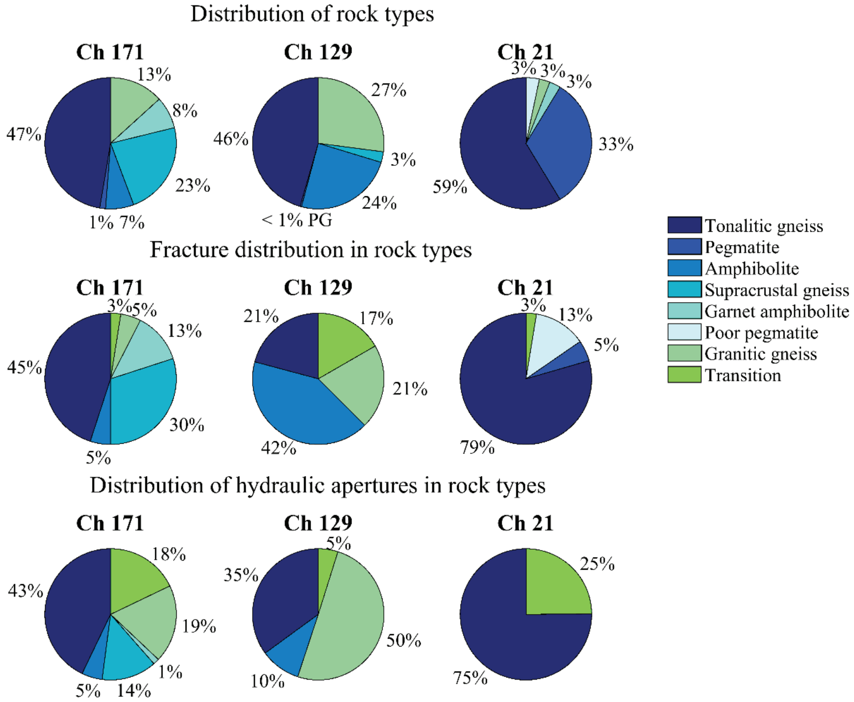



**Fig. 16** Pie charts showing distribution of rock types at each side, distribution of fractures in each rock type at each test location and distribution of hydraulic apertures in each rock type at each test location.

At Ch. 129 the amphibolite is considerably more fractured than the other rock types present, but the fractures have considerable smaller hydraulic apertures than average. The granitic gneiss is slightly less fractured than average, but the present fractures have considerable larger hydraulic apertures than average. The tonalitic gneiss has considerably less fractures than average, but the fractures has larger hydraulic apertures. 17% of the fractures are in the transition between two rock types, these fractures have smaller hydraulic apertures than average. All the fractures in a transition is between amphibolite and other rock types.

At Ch. 21 it can be observed that tonalitic gneiss and poor pegmatite are more fractured than the other rock types present. The hydraulic apertures in tonalitic gneiss are approximately average for this location. During the high precision water pumping test the transition zone and the poor pegmatite was in the same test section, which means that the fractures present in the transition zone and the poor pegmatite have considerable larger hydraulic apertures than average. The pegmatite at this location has considerably less fractures than average and the fractures present did not have measurable transmissivity. Therefore, it was not possible to calculate hydraulic apertures.

These results indicate that fractures in coarse grained rock types, generally have large or medium hydraulic apertures, and low or medium degree of fracturing, while fine grained rock types have smaller hydraulic apertures, but higher degree of fracturing. Garnet amphibolite consists of amphibolite (fine grains) and garnet crystals (coarse grains). The JRC is generally measured to be high, but in this type of rock the hydraulic apertures are smaller than average. These results do not agree with the theory from Gustafson (2012); acidic $SiO_2$ rich rock will tend to be more fractured and have higher transmissivity than dark mafic rock. Indeed, there is a tendency towards lower transmissivity in mafic rock types, but also supracrustal gneiss which is a $SiO_2$ rich rock type was found to have lower transmissivity. Furthermore, the $SiO_2$ rich rock was generally not more fractured than the mafic rock types.

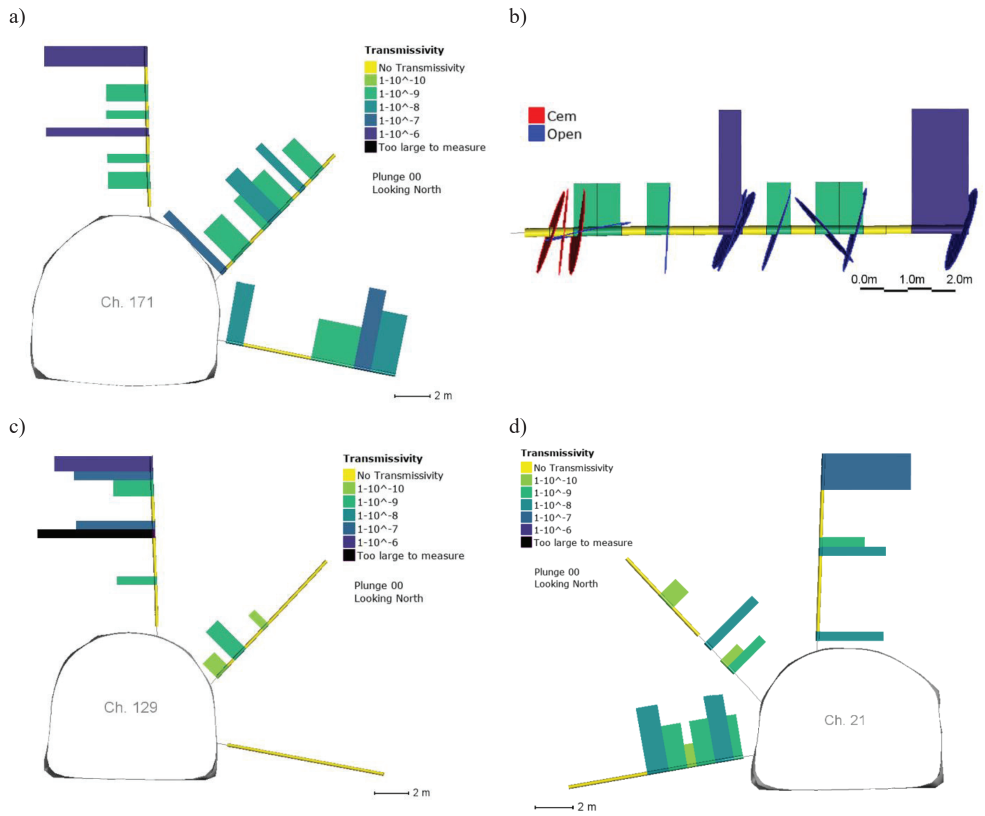



**Fig. 17** a) Transmissivity at Ch. 171, b) roof hole at Ch. 171 with fractures, c) Transmissivity at Ch. 129, d) Transmissivity at Ch. 21.

**Fig. 17** shows excerpts from the 3D model, showing the transmissivities along the test holes. The bar plots along the test holes are in log scale. **Fig. 17** a) represents test location Ch. 171 and **Fig. 17** b) the test hole in the roof at test location Ch. 171 tilted sideways, with fractures. It can be observed that all the sections with measurable transmissivity are directly linked to fractures, and that cemented fractures have no measurable transmissivity. One section with a measurable transmissivity, have both a cemented fracture and a fracture with trace of cement. It can be concluded that the cemented fracture has no measurable transmissivity, because another water injection test section is intersecting the fracture with trace of cement, and the transmissivity is identical in this section. The same trends are observed for the other eight test holes. **Fig. 17** c) present the transmissivity at Ch. 129 and **Fig. 17** d) the transmissivity at Ch. 21. The test hole in the wall at Ch. 129 only intersects two fractures, which are cemented and have trace of cement. It can be observed that at in all the locations the fractures with highest transmissivity are found in the mid to deeper parts of the test holes placed in the roof. The reason for this is that the drill holes in the roof have intersected large ungrouted subhorizontal fractures, that were not within the reach of the pre-grouting. At all three

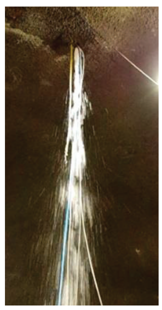


**Fig. 18** Water pouring out of the test hole in the roof at Ch. 129.

locations it is approximately 5 metres between each subhorizontal fracture with high transmissivity. The roof hole at Ch. 21 is terminating in one of the subhorizontal weakness zones in the area, explaining the difficulties during the grouting work at this location.

As illustrated in **Fig. 18**, these subhorizontal fractures were very water conductive and the water was pouring into the tunnel after the drilling of the test holes for this study. This demonstrates that grouting of the large water conductive fractures close to the tunnel profile, was sufficient enough to meet the strict requirements for inflow at these three test locations.

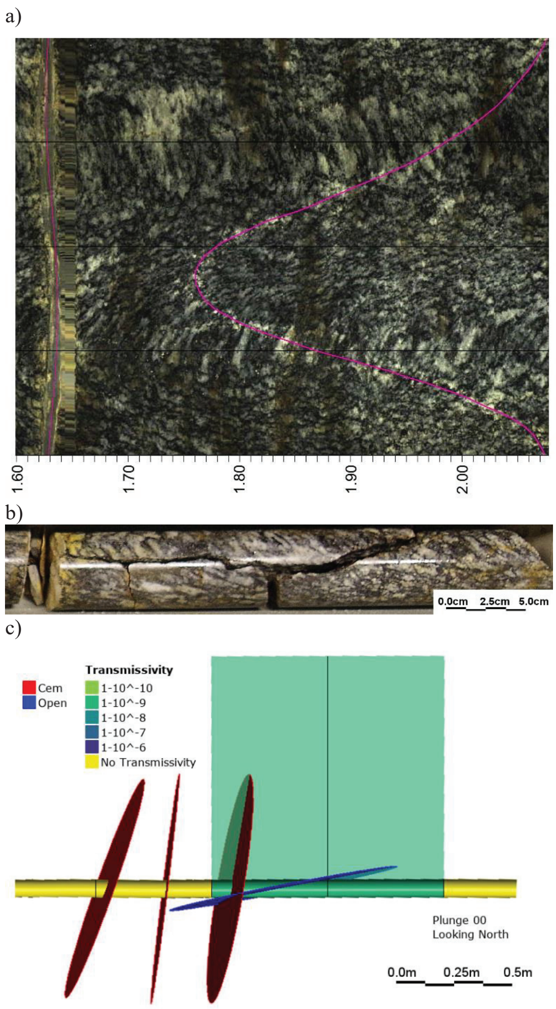



**Fig. 19** The same hole section presented in three different ways; a) OTV, b) drill core, c) 3D model. In c) the section is including the rightmost red disc and the blue disc.

**Fig. 19** represents a detailed case example from the roof hole at Ch. 171. **Fig. 19** a) is the view from the OTV, showing the rock in the drill hole wall. In this view it is not possible to see the part of the thin fracture that intersects with the cemented fracture, however it is possible to evaluate the approximate apertures of the visible parts of the fractures. The aperture of the cemented fracture is 8 mm, while the aperture of the fracture with trace of cement is estimated to be smaller than 1 mm. **Fig. 19** b) is a photo of the drill cores representing the same section. In the fracture with the smallest aperture it was only found trace of cement in the 10 cm closest to the cemented fracture. **Fig. 19** c) shows the 3D model of the section with the two intersecting fractures with the sections of water injection tests, illustrating that the large fracture is successfully sealed with cement, while the fracture with the small measured aperture has measurable transmissivity. The estimated hydraulic aperture for this fracture is 0.19 mm, which is small, but should be within the range of MFC grout, according to Stille (2015).

## 7 Discussion and summary

The number of cemented fractures in the zone 5 metres outside the tunnel profile was surprisingly low, representing 36% of the fractures at Ch. 171, 21% at Ch. 129 and 12% at Ch. 21. According to Stille (2015) one would expect that smaller fractures than 1 mm, but larger than 0.157 mm, would easily be filled with cement, with the types of cement used at the test locations in this study. However, no fractures under 1 mm filled with cement were found. The grout consumption in the grout rounds at the test locations is very large, and the grouting pressure used is relatively high. It is reasonable to assume that the large subhorizontal fractures in this area have consumed the major share of the grout. The grout spread in these fractures ensured that the grouting was successful in regard to sufficient reduction of the water ingress into the tunnel.

In this study it is found that most of the fractures intersected by drilling are subhorizontal (0º±30º). 85% of the grouted fractures were also subhorizontal. In this regard it is important to keep in mind that the direction of the drilled test holes in this study has largely impacted which type of fracture sets the test holes have intersected. Due to a N-S orientation of the tunnel, it was not practicable to drill test holes in the direction that would easily intersect the major fracture set in the area, but in retrospective it is realized that the test holes in the abutment and wall should have been drilled in opposite angles from the tunnel profile, to better represent fractures with different orientations. The total share of cemented fractures for all the fractures close to the tunnel profile is therefore not revealed by this study.

It can be noted that the prevalence of subhorizontal fractures is higher in the test locations compared to the well holes (**Fig. 6**). The well holes are drilled close to vertical and should give a good representation of subhorizontal fractures in the area. The reason for this is unknown, but the close vicinity of the subhorizontal weakness zones to the test locations might be an explanation.

No significant correlation between JRC of fractures and hydraulic apertures was found, but a tendency towards smaller hydraulic apertures with low JRC. With increasing JRC the hydraulic apertures were in both ends of the scale, including both small and large hydraulic apertures. This could be related to the finding in the study by Li et al. (2008), that shearing of rough fractures gives increased hydraulic apertures compared to smooth fractures, but with no shearing of rough fractures, this will not be the case. This could explain the heteroscedasticity of the data.

It was found generally higher JRC values in coarse grained rock types, such as tonalitic gneiss, granitic gneiss and pegmatite, and lower JRC values for fine grained rock types, such as amphibolite and supracrustal gneiss.

From the pie chart presented in **Fig. 16**, it was found that in fine grained rock types, such as amphibolite and supracrustal gneiss, the hydraulic apertures were smaller, even though these rock types were more fractured than average. Granitic gneiss was the rock type that was found to have the largest hydraulic apertures, although granitic gneiss was less fractured than average. Tonalitic gneiss had relative average degree of both fracturing and hydraulic apertures.

Gustafson (2012) suggested that acidic, $SiO_2$ rich rock types tend to fracture more easily than basic, dark rock types and that dark basic rock types in general has lower transmissivity. The theory regarding more fractures in the $SiO_2$ rich acidic rock types was not fitting for this study, but the theory stating that acidic $SiO_2$ rich rock will tend to have higher transmissivity than dark mafic rock, were partly fitting for this study, but not for supracrustal gneiss, which is a fine grained $SiO_2$ rich rock type.

Indication of HJ was found several times during grouting. The subhorizontal fractures extend the risk of HJ, since the minor principal stress is close to parallel with the direction of the overburden pressure. The minor principal stress in the areas with HJ was approximately 10 MPa. The HJ started at 3.5 MPa and was mainly occurring at 4.0 to 4.5 MPa. This indicates that the HJ occurred at a grouting pressure approximately one third of the pressure in the direction of the overburden pressure. HJ at 1/3 of the pressure of the minor principal stress is a surprising result, since HJ of a fracture theoretically only can occur at a pressure similar or higher than the pressure acting perpendicular to the fracture surface. The reason for HJ at a significantly lower grouting pressure than the minor principal stress is not known, it can only be speculated if the high anisotropy in the stress might inflict, or the in-situ stress at the test locations are significantly different from the locations where the stress measurements are performed.

In this case the consequences of HJ can be critically discussed. The orientation of the most groutable fractures is favourable for uplift of the overburden, also, the grout consumption is highly likely to be increased due to increased volume of the large fractures. Additionally, it is plausible that HJ of large fractures could have resulted in decrease in the aperture of smaller fractures, reducing the penetrability in these fractures during grouting.

In the grout round at chainage 155 a total of 103 tonnes of cement was used, which in this case represented 116 097 litres of grout. The grouting works lasted for 64 hours. Assuming that the grout spread in large subhorizontal fractures, it can be speculated if this large amount of grout was necessary to achieve the required tightness around the tunnel profile. Let us presume that three different subhorizontal fractures were intersected by the grouting holes and the fractures were smooth with a large average groutable aperture of 3 mm. This would give a disk shape distribution of the grout, with a spread radius of 55 metres in each of the 3 fractures. This equivalent to an area of 1.8 soccer fields in each fracture. In real life it is possible that the grout spread would be even larger du to channelling and anisotropic spread of the grout. This suggests that the grouting performed in this area might have been excessive and it is likely that the tunnel would be tight enough with less grout.

In many cases it seems like the philosophy during rock mass grouting when tunnelling in sensitive areas is “better to be safe than sorry”. The difficulties and costs with performing post-grouting, or risking damage of surfaces structures due to drawdown of the groundwater table in many cases results in excessive use of grout and time. With the available information during rock mass grouting in todays practice, this absolutely safe mentality is understandable. For the future, it would be beneficial to have a better understanding of hydrogeology, fracture distribution and stress condition in the rock mass before taking qualified decisions on-site regarding when the grouting should stop.

## 8 Main conclusions

In this study pre-grouted rock mass have been investigated in regard to spread of grout and transmissivity. The following main conclusions were drawn;

- The grout penetration into small fractures was less than expected. Only fractures that had a measured aperture of 1 mm or larger, at the drill hole intersection, were found to be fully grouted. From laboratory studies the grout used at the test locations at this study should be able to penetrate fractures down to 0.16 mm. Over-all, 20% of the fractures were filled with grout.
- It was found a tendency towards smaller hydraulic apertures with low JRC values. With increasing JRC the hydraulic apertures were in both ends of the scale, including both small and large hydraulic apertures.
- It was found generally higher JRC values in coarse grained rock types, such as granitic gneiss, tonalitic gneiss and pegmatite, and lower JRC values for fine grained rock types, such as amphibolite and supracrustal gneiss.
- In fine grained rock types, such as amphibolite and supracrustal gneiss, the hydraulic apertures were smaller, even though these rock types were more fractured than average. Granitic gneiss was the rock type that was found to have the largest hydraulic apertures, although granitic gneiss was less fractured than average. Tonalitic gneiss had relative average degree of both fracturing and hydraulic apertures.

This study also demonstrates that measurement of in-situ rock mass stress is an important tool to understand the hydrogeology and the grout spread in the rock mass.

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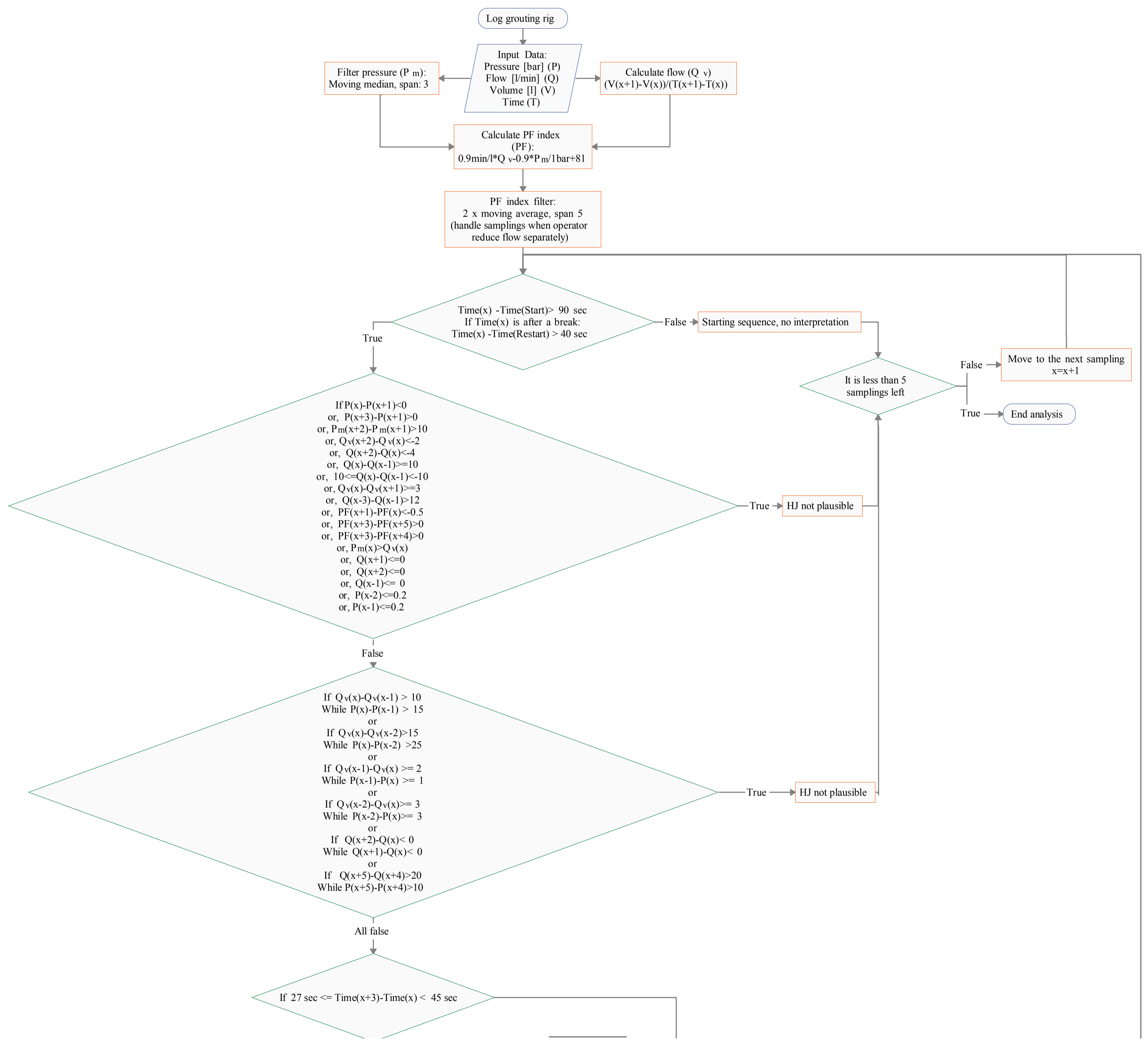
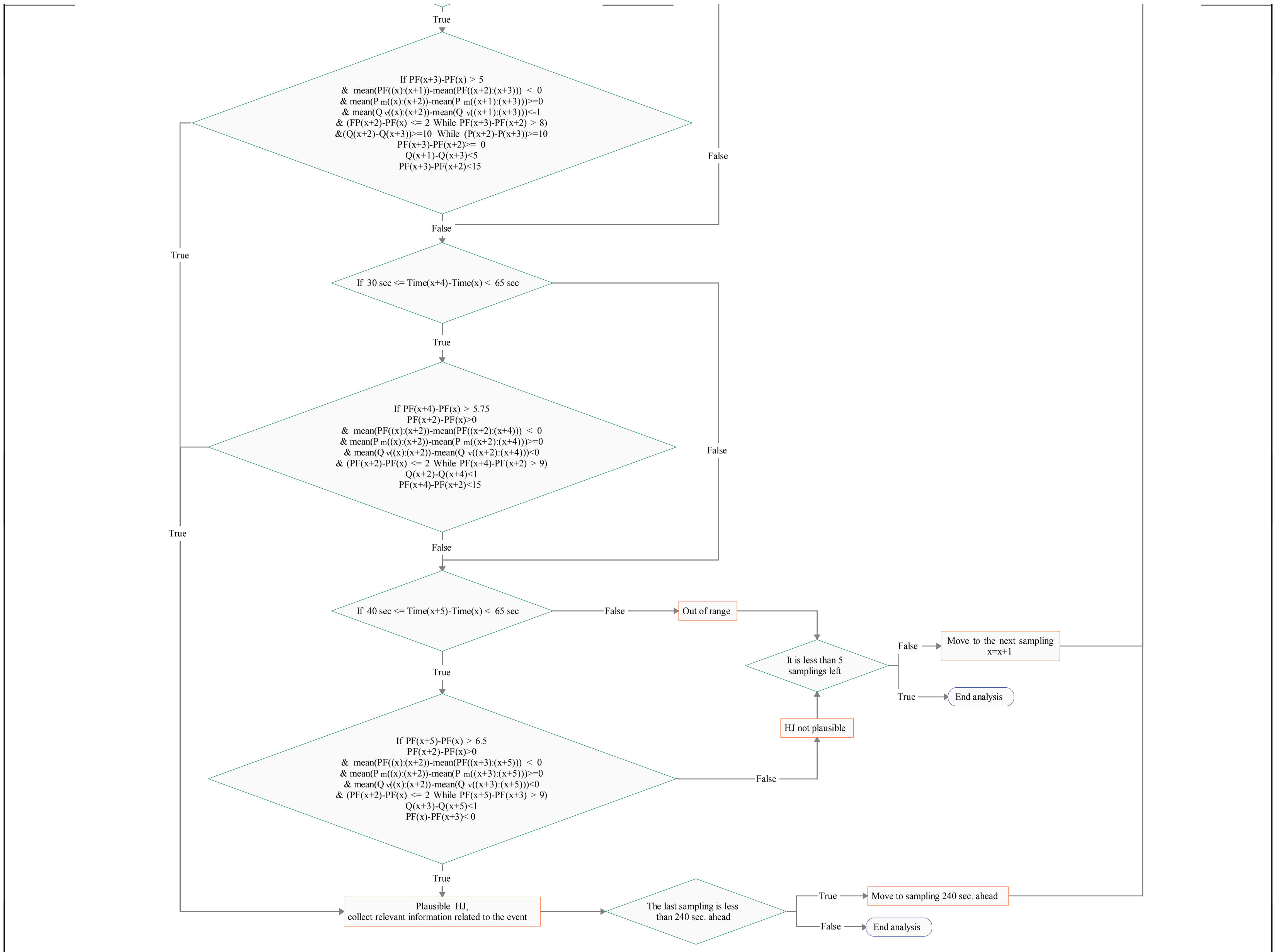
# Appendix A



Log grouting rig
Input Data:
Pressure [bar] (P)
Flow [l/min] (Q)
Volume [l] (V)
Time (T)
Filter pressure (P m):
Moving median, span: 3
Calculate flow (Q v)
(V(x+1)-V(x))/(T(x+1)-T(x))
Calculate PF index
(PF):
0.9min/l*Q v-0.9*P m/1bar+81
PF index filter:
2 x moving average, span 5
(handle samplings when operator
reduce flow separately)
Time(x) -Time(Start)> 90 sec
If Time(x) is after a break:
Time(x) -Time(Restart) > 40 sec
False
Starting sequence, no interpretation
True
It is less than 5
samplings left
False
Move to the next sampling
x=x+1
True
End analysis
If P(x)-P(x+1)<0
or, P(x+3)-P(x+1)>0
or, Pm(x+2)-P m(x+1)>10
or, Qv(x+2)-Q v(x)<-2
or, Q(x+2)-Q(x)<-4
or, Q(x)-Q(x-1)>=10
or, 10<=Q(x)-Q(x-1)<-10
or, Qv(x)-Qv(x+1)>=3
or, Q(x-3)-Q(x-1)>12
or, PF(x+1)-PF(x)<-0.5
or, PF(x+3)-PF(x+5)>0
or, PF(x+3)-PF(x+4)>0
or, Pm(x)>Qv(x)
or, Q(x+1)<=0
or, Q(x+2)<=0
or, Q(x-1)<= 0
or, P(x-2)<=0.2
or, P(x-1)<=0.2
True
HJ not plausible
False
If Qv(x)-Qv(x-1) > 10
While P(x)-P(x-1) > 15
or
If Qv(x)-Qv(x-2)>15
While P(x)-P(x-2) >25
or
If Qv(x-1)-Qv(x) >= 2
While P(x-1)-P(x) >= 1
or
If Qv(x-2)-Qv(x)>= 3
While P(x-2)-P(x)>= 3
or
If Q(x+2)-Q(x)< 0
While Q(x+1)-Q(x)< 0
or
If Q(x+5)-Q(x+4)>20
While P(x+5)-P(x+4)>10
True
HJ not plausible
All false
If 27 sec <= Time(x+3)-Time(x) < 45 sec




Flow chart 4.mmap - 19.10.2017 - Mindjet