Investigation and assessment of pre-grouted rock mass

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Abstract

Pre-grouting is a technique for reducing water ingress into tunnels and caverns by grouting fractures and joints prior to excavation. This study investigates pre-grouted rock mass to evaluate grout penetration in fractures and transmissivity of water in the rock mass surrounding the built tunnel, with the use for core drilling, OTV, high-precision water injection tests and core logging. 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 fractures with small apertures, compared with results of grout penetrability in laboratory studies of similar grouts. Further, it was found that fractures in coarse-grained rock types had rougher fracture surfaces and higher hydraulic apertures, than fractures in fine-grained rock types. It was also found that fractures with smoother surfaces had smaller hydraulic apertures in general. Hydraulic jacking was evidenced during the pre-grouting in this area, which is likely to have contributed to unnecessary high grout consumption.

Keywords Pre-grouting · Hydraulic transmissivity · Hydraulic aperture · Grout penetrability

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 m length at the face of the tunnel, depending on the tunnel face area and geology. Packers attached to grouting rods are placed approximately 2 m 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 with additives.

In this study pre-grouted rock mass has been investigated to evaluate grout penetration in fractures and transmissivity of water in the rock mass surrounding the built tunnel. The study includes investigation and assessment of the rock mass at three locations in tunnels excavated with drilling and blasting in connection with the Follo Line project in Norway. Core drilling, Optical Televiewing (OTV), high-precision water injection tests and core logging were performed at all test locations. 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. The study also evaluated hydraulic transmissivity regarding rock types and joint roughness, with the use of joint roughness coefficient (JRC).

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

Background theory

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 implies 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 “Method for high-precision water injection tests”.

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:

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

where b is hydraulic aperture, μ is viscosity of water, T_f is transmissivity, ρ 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. 2, 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.

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 with 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 with smoother fractures. This implies that the stress distribution in combination with fracture roughness has 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

Fig. 2 Drill holes intersecting fractures, illustrating that the visible part of a fracture in a drill hole does not realistically reveal the fracture geometry

rapid unloading of the surface due 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 m of rock mass (Stephansson et al. 1991). The change in stress distribution, caused by rapid removal of the overburden, has 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 today's 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, characterised 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 have 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 SiO₂ content has higher hydraulic conductivity, whilst basic rock types, e.g. greenstone, have 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 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 joint sets and the type of rock mass.

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 "Permeability in rock mass and groundwater flow" 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. Furthermore, the penetration mechanism of the grout during injection involves displacement of the water which saturates the joints in the rock mass.

In Norwegian projects the main goal during pre-grouting is to fill fractures 5–6 m 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-grouting can be optimised 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 flow rate and increased volume of fractures.
2. Uplift of the overburden, if the fractures are close to horizontal oriented.
3. Increased transmissivity outside the grouted zone, due to increased apertures of fractures.
4. Finer fractures 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.

Site description

The Follo Line project is a twin tube railway tunnel connection between Oslo and Ski. The main tunnels are excavated by using four TBMs, all starting from an adit at Åsland, which is located in the middle of the two tunnels. At Åsland there is a construction site, 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). The tunnels at the construction site are all excavated using drilling and blasting. An overview of parts of the construction site is presented in a map in Fig. 3.

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. 3. For each location three holes of 10 m 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. 4.

The tunnels investigated in this study are excavated with the use of 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 levels 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 m, respectively.

Fig. 3 Overview of geology and tunnels at Åsland, modified from FPS (2014)

Fig. 4 Positioning of test holes

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. 3.

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 a combination of tectonic stress from both the Mid-Atlantic Ridge push and rapid unloading of the surface due to erosion and deglaciation. It can be noted that σ_1 rotates 79°

Table 1 Test results from 3D stress measurements at Åsland

	Test location north		Test location south	
	Stress (MPa)	Orientation	Stress (MPa)	Orientation
σ_1	24.3 ± 2.3	N169° Dip: 3°	24.5 ± 2.4	N90° Dip: 7°
σ_2	14.6 ± 2.3	N78° Dip: 3°	15.7 ± 3.1	N182° Dip: 18°
σ_3	11.8 ± 2.3	N304° Dip: 86°	10.3 ± 1.1	N339° Dip: 70°

in only 400 m; the assumed reason for this is the close presence of major weakness zones, as illustrated in Fig. 3. 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 m from test location south, whilst Ch. 21 is approximately 650 m from test location north.

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 TBMs passing close to the test locations were evaluated. Figure 6(a) shows fractures logged by OTV of probe drill holes from TBM at three locations, marked in Fig. 3. The probe holes are drilled roughly parallel to the TBM tunnel alignment, which has an approximate direction of 170° SSE. It can be noted that these drill holes are oriented about 80° different in strike direction than the test holes placed in the wall and springline in this study. The stereonet shows that the major joint set is vertical, with strikes towards E and W and dips of 70°–90°. Since the TBM probe drillings are oriented roughly 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.

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 are still within the same region as described regarding regional stress distribution and fracture distribution in “Permeability in rock mass and groundwater flow”. Figure 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 m length, with 10.5 m deviation from vertical direction. BH15+580 is 150 m long, with a 50° 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 70°–90°. There are relatively few horizontal fractures.

As shown by Fig. 3, 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 the authors of this study, but they are possibly related to the Caledonian Orogeny.

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.

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

Fig. 6 Fractures plotted as poles in lower hemisphere equal angle stereonets. (a) Probe drill holes in front of two TBMs, (b) two well holes drilled close to the tunnel alignment

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 $d_{95} < 25 \mu\text{m}$; Ordinary Portland Cement (OPC) is cement with a $d_{95} = < 40 \mu\text{m}$ and $> 25 \mu\text{m}$.

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 l 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 l, 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 l 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.

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

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

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 s, 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).

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. Figure 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 indicates 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

Table 3 Grout type and grout consumption per round

Chainage (m)	Grout	w/c ratio	Additives	Consumption (kg)
155 → 181	MFC	1.0/0.8/0.6	Superplasticiser	102,996
164 → 182	MFC	1.0	Superplasticiser	7625
105 → 131	MFC	1.0/0.8/0.6	Superplasticiser	43,218
10 → 37	OPC/ Zugol	0.9/0.5	Silica slurry and superplasticiser	43,152

Ch. 21 the water inflow limit was < 10 l/min per 100 m of tunnel. At all three test locations the requirements regarding water ingress into the tunnel were met, and the grouting works were considered successful.

Investigation, testing and analysis of drill holes

At each location, three holes were drilled as illustrated in Fig. 4, 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.

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 precise apertures of fractures less than 1 mm.

Figure 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 colours. 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.

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 m were isolated by two individually operated hydraulic packers of 0.45 m 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 was 5 bar over the natural formation pressure; the duration of the water injection was approximately 10 min 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

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

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

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 Eq. 2.

$$T_M = \frac{Q_p \times \rho_w \times g}{dP_p} \times C_M \quad (2)$$

$$C_M = \frac{1 + \ln\left(\frac{L_w}{2r_w}\right)}{2\pi} \quad (3)$$

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

Some of the planned test sections had to be adjusted (see Table 4).

Core logging

In the core logging the following was emphasised:

- 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 were made. In test hole positioned in the wall at Ch. 171, a large dyke of amphibolite at 6 m depth appeared to be massive without fractures in the OTV images, whilst 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.

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 categorised as follows:

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 (talca)

Fracture fill 2: yellow/white, non-slippery, hard crystallisation, reacts with hydrochloric acid (calcite)

Fracture fill 3: rusty, non-slippery, grainy, no reaction with hydrochloric acid

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

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.
Spr. 129	0.00–2.55	Double-drilled start of the hole. Measurements could not be performed in the affected section.
Wall 21	4.55–5.70	Offset in packer placement due to large fracture.

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 roughness 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.

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

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

Fig. 11 Water pumping section with a fracture

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

1. Number and orientation of all fractures
2. Number and orientation of grouted fractures
3. Hydraulic transmissivity in all test holes
4. JRC of fractures in different types of rock
5. Distribution of rock types, fractures in rock types and hydraulic apertures in rock types
6. Hydraulic apertures compared with JRC

To investigate the data in bullet point 6, 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 “Background theory”. Figure 11 illustrates a single fracture within a water injection test. The transmissivity, calculated by

using Eq. 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 Eq. 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 were 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.

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 were calculated hydraulic aperture and JRC. The statistical significance level was set to a p value < 0.05 .

All stereonet presented in this paper are produced by using the software Dips from Rocscience.

Results

Fracture distribution and grout penetration

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 stereonet, 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 "Fracture distribution and grout penetration".

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. Figure 13 presents a stereonet of all the cemented fractures, where it can be observed that 17 of the 20 fractures are subhorizontal.

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 six fractures (6%). 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.

Thirteen 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.

Case example of test hole section

Figure 14 represents a detailed case example from the roof hole at Ch. 171. Figure 14 (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, whilst the aperture of the fracture with trace of cement is estimated to be smaller than 1 mm. Figure 14(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. Figure 14(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, whilst the fracture with the

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

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

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).

Transmissivity in the rock mass surrounding the tunnel

Figure 15 shows 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 has 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, and none of the cemented fractures were evaluated to have measurable transmissivity. In the six fractures with trace of cement there was found measurable transmissivity in three of the fractures.

Table 5 Percentage of cemented fractures

Test location	Cemented fractures	
	0–5 m	5–10 m
Ch. 171	36%	22%
Ch. 129	21%	20%
Ch. 21	12%	0%

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.

Figures 16, 17 and 18 show excerpts from the 3D model, showing the transmissivities along the test holes at all three test locations. The fractures are excluded from this view due to readability. The test hole in the wall at Ch. 129 had no measurable transmissivity, as it only intersects two fractures, one cemented and one with 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 locations it is approximately 5 m 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. 19, 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.

Estimated JRC for rock types

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 values of the fractures were evaluated with regard to rock type. The median JRC and the average grain size for all the represented rock types are shown in Fig. 20. Some of the fractures were in the transition (Tr) between two rock types, where there is no representative grain size. The grain size of garnet amphibolite was a combination of 10 mm for garnet and 0.3 mm for amphibolite.

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.

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. 14 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

Fracture distribution and hydraulic apertures in different rock types

Figure 21 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 distributions 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

Fig. 15 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

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.

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 have larger hydraulic apertures. Seventeen percent 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 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. 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, whilst 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.

The results presented in Fig. 21 do not reflect that the rock mass is grouted, but the grouted fractures are presented as

Fig. 16 Model showing tunnel profile, hole positioning and hydraulic transmissivity in sections, at Ch. 171. The bar plots along the test holes are in log scale

Fig. 17 Model showing tunnel profile, hole positioning and hydraulic transmissivity in sections, at Ch. 129. The bar plots along the test holes are in log scale

fractures in the rock mass. The grouted fractures found in this study had no measurable permeability, which would inflict on the results, compared with the same study performed in an ungrouted rock mass. For this reason the estimated hydraulic apertures are conservative in sections where the grouted fractures are present.

The pie charts shown in Fig. 22 present 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

Fig. 18 Model showing tunnel profile, hole positioning and hydraulic transmissivity in sections, at Ch. 21. The bar plots along the test holes are in log scale

Fig. 19 Water pouring out of the test hole in the roof at Ch. 129

two types of rock. The two cemented fractures in amphibolite do not have significant impact on the earlier findings, indicating that 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.

Fig. 20 Median of JRC and average grain size for rock types

Hydraulic apertures compared with JRC

Thirty of all 103 fractures were single fractures placed within one section of water injection with measurable transmissivity. For this selection of 30 fractures, a pairwise correlation analysis was performed between calculated hydraulic aperture and JRC, as described in “Methods for data interpretation and 3D model”.

The results of the analysis are summarised in Fig. 23. 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.

Discussion and summary

Fracture distribution and grout penetration

The number of cemented fractures in the zone 5 m outside the tunnel profile was surprisingly low, representing 36% of the fractures at Ch. 171, 21% at Ch. 129 and 12% at Ch. 21. Percentage of grouted fractures is not as expected, compared with results of grout penetrability in laboratory studies of similar grouts. 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 ($0^\circ \pm 30^\circ$). Eighty-five percent 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 realised that

Fig. 21 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

the test holes in the springline 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 completely revealed by this study.

It can be noted that the prevalence of subhorizontal fractures is higher in the test locations compared with 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.

Fracture distribution in rock types, hydraulic apertures and JRC

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. 21, 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, SiO₂-rich rock types tend to fracture more easily than basic, dark rock types and that dark basic rock types in general have lower transmissivity. The theory regarding more fractures in the SiO₂-rich acidic rock types was not fitting for this study, but the theory stating that acidic SiO₂-rich rock will tend to have higher transmissivity than dark mafic rock was partly fitting for this study, but not for supracrustal gneiss, which is a fine-grained

Fig. 22 Pie charts showing distribution and number of cemented fractures in different rock types for each test site

Fig. 23 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

SiO_2 -rich rock type. The different findings in this study could be due to different stress conditions in the rock mass and geological history.

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 with smooth fractures, but with no shearing of rough fractures, this will not be the case. This could explain the heteroscedasticity of the data.

Hydraulic jacking

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 measured 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 presence of a subhorizontal weakness zone above the test area could be the explanation for HJ at a significantly lower grouting pressure than the minor principal stress.

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. This could also explain why there were no grouted fractures with apertures under 1 mm found.

Grout consumption

In the grout round at chainage 155 a total of 103 t of cement was used, which in this case represented 116,097 l of grout. The grouting works lasted for 64 h. 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 disc shape distribution of the grout, with a spread radius of 55 m 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. This was not possible to know during the grouting procedure, since there was not performed any estimation of expected grout consumption based on the types of fractures present in the area.

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 results in excessive use of grout and time. With the available information during rock mass grouting in today’s 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.

Main conclusions

In this study pre-grouted rock mass has been investigated in regard to grout penetration and transmissivity. The following main conclusions were drawn:

- The grout penetration into small fractures was less than expected, compared with measured penetrability of similar grouts in laboratory tests. 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. Overall, 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.
- It was concluded that HJ during pre-grouting in this area might have contributed to unnecessary high grout consumption and decrease in the aperture of small fractures, which could explain why there were no grouted fractures with apertures under 1 mm found.

In general, it was concluded that the grout consumption from pre-grouting in one of the test locations was excessive. The conclusion was made based on which type of fractures the grout had spread in. This was not possible to know during the grouting procedure, since there was not performed any estimation of expected grout consumption based on the types of fractures present in the area. 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.

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References

- Aarset A, Hognestad HO, Fagermo JI, Kveen A, Backer L, Grøv E, Frogner E (2011) Rock mass grouting. Norsk Forening for Fjellsprengningsteknikk (NFF)
- Barton N, Choubey V (1977) The shear strength of rock joints in theory and practice. *Rock Mech* 10:1–54. <https://doi.org/10.1007/BF01261801>
- Barton N, de Quadros EF (1997) Joint aperture and roughness in the prediction of flow and groutability of rock masses. *Int J Rock Mech Min Sci* 34(252):e251–e252. e214. [https://doi.org/10.1016/S1365-1609\(97\)00081-6](https://doi.org/10.1016/S1365-1609(97)00081-6)
- Dahle H, Larsen T, 2005. In-situ rock stress measurements; Brief description of methods applied by SINTEF SINTEF
- Dalgaard P (2002) Introductory statistics with R. Springer, New York
- FPS AS, 2014. The Follo Line, Åsland Rig Area Substructures, Doc.no. UFB-31-A-30035. BaneNOR
- Grøneng, G., Nilsen, B., 2009. Procedure for determining input parameters for Barton-Bandis joint shear strength formulation. Department of Geology and Mineral Resources Engineering
- Gudmundsson A, Fjeldskaar I, Gjesdal O (2002) Fracture-generated permeability and groundwater yield in Norway. *NGU Bull* 439:61–69
- Gustafson G, 2012. Hydrogeology for rock engineers. BeFo
- Heeremans M, Larsen BT, Stel H (1996) Paleostress reconstruction from kinematic indicators in the Oslo Graben, southern Norway: new constraints on the mode of rifting. *Tectonophysics* 266:55–79. [https://doi.org/10.1016/S0040-1951\(96\)00183-7](https://doi.org/10.1016/S0040-1951(96)00183-7)
- Li B, Jiang Y, Koyama T, Jing L, Tanabashi Y (2008) Experimental study of the hydro-mechanical behavior of rock joints using a parallel-plate model containing contact areas and artificial fractures. *Int J Rock Mech Min Sci* 45:362–375. <https://doi.org/10.1016/j.ijrmm.2007.06.004>
- Moye DG (1967) Diamond drilling for foundation exploration. *Civil Engineering Transactions / the Institution of Engineers, Australia*
- Palmstrøm A (2015) Joint characteristics. RockMass AS, Oslo
- Stephansson O, Ljunggren C, Jing L (1991) Stress measurements and tectonic implications for Fennoscandia. *Tectonophysics* 189:317–322. [https://doi.org/10.1016/0040-1951\(91\)90504-L](https://doi.org/10.1016/0040-1951(91)90504-L)
- Stille H (2015) Rock grouting -theories and applications. BeFo, Stockholm
- Strømsvik H, Morud JC, Grøv E (2018) Development of an algorithm to detect hydraulic jacking in high pressure rock mass grouting and introduction of the PF index. *Tunn Undergr Space Technol* 81:16–25. <https://doi.org/10.1016/j.tust.2018.06.027>