Rock Mass Response during High Pressure Grouting

RIKARD GOTHÄLL

Licentiate Thesis

Division of Soil and Rock Mechanics
Department of Civil and Architectural Engineering
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NOMENCLATURE

\( \mu_W \) Viscosity of water ........................................... [Pa s]
\( \rho_W \) Density of water ........................................... [kg/m³]
\( \sigma \) Stress ......................................................... [Pa]
\( \bar{\sigma} \) Effective stress ........................................... [Pa]
\( \tau_0 \) Shear strength of Bingham fluid ......................... [Pa]
\( \varphi \) Hydraulic potential ....................................... [m]
\( \gamma \) Poisson ratio ................................................ [-]
\( \bar{\sigma}_c \) Effective stress in contact areas ....................... [Pa]
\( a \) Radius of loaded zone .......................................... [m]
\( A_c \) Effective contact area fraction .............................. [-]
\( b \) Aperture ......................................................... [m]
\( g \) Gravitational acceleration ................................... [m/s²]
\( h \) Depth from groundwater surface ................................ [m]
\( K \) Hydraulic conductivity ......................................... [m/s]
\( K_s \) Conductivity of sealed zone ................................. [m/s]
\( \Delta P \) Grouting pressure exceeding critical pressure ............ [Pa]
\( P_{E} \) Effective grouting pressure ................................. [Pa]
\( Q \) Inflow per unit time .............................................. [m³/s]
\( Q_l \) Inflow per unit length and time ................................ [m²/s]
\( R \) Achievable radius ................................................ [m]
\( r_g \) Extent of sealed zone ........................................... [m]
\( r_w \) Radius of well or tunnel ....................................... [m]
\( u \) Pore pressure ....................................................... [Pa]
\( W \) Width of channel ................................................ [m]
Part I

THE THESIS
1. INTRODUCTION

1.1 History and Background

Inflow to underground constructions has been a large problem in underground engineering since man first ventured below the surface of the ground. In the mid 1600s, over half the staff at the Falun copper mine were occupied with the drainage of the mine. The advents of mechanised pumps was a big step forward for underground engineering, but it was not until much later that efforts where made to actually stop the inflow of water into underground facilities.

Since then a lot has happened, but the problem with inflow of water remains a large concern in tunnel building projects. Though the focus has shifted from the problem of draining underground excavations to sealing them, the cost of doing so still remains a large part of the total construction cost. The environmental impact of sealing or failing to seal underground constructions has also moved into focus in recent time. The inflow of water can lower the water table which will result in damage to both the ecosystem and the local economy. Unsuccessful efforts to seal the inflow may also result in uncontrolled spread of sealant. These two problems are not mutually exclusive.

1.1.1 Grouting Tradition in Sweden

In Sweden sealing underground constructions has mainly been done by cementitious grouting. This is partly due to high availability of affordable cement grout. The geology in Sweden consists mainly of hard jointed granitic rock with frequent water bearing fractures. This geology often eliminates the need for a concrete lining for support purposes but requires substantial efforts to be sealed.

Grouting is usually integrated in the excavation cycle. A fan of holes are drilled around the tunnel face and grout is injected in these holes one by one or a few simultaneously at moderate to high pressure.

This process may be governed by a controlling process where inflow or permeability is measured before or after the grouting is performed and the result will indicate if another grouting round is to be performed.

The history of early grouting research in Sweden is summarised in.\footnote{54}, since
then there have been ongoing efforts concerning the characterisation of rock from a grouting perspective, concerning the characterisation of grout and concerning the prediction of grouting result. The research regarding engineering methods for sealing underground constructions is closely related to the research of fluid flow in fractures that is also being performed in Sweden.

1.1.2 Why and When to Grout

The most rudimentary reason for sealing an underground cavity is to avoid it from flooding. If the inflow is lower than pumping capacity this is not a concern. Then it may be the working environment or installations that require dry conditions that necessitate sealing of the rock mass. Fulfilling such requirements are not a major concern in Swedish practise and it may be accomplished with routine procedures.

The water that is pumped away from the underground construction would, in the absence of the construction, have been a part of another hydrological process. That process may be held at a value great enough to motivate more costly sealing procedures in order to leave it undisturbed. The disturbance of other hydrological processes in the vicinity of the construction site are often the primary and limiting concern in excavations close to urban or otherwise sensitive areas.

As previously stated, grouting is often included in the drill, blast, muck-cycle. Then it is often called pregrouting. It can also be performed after the excavation cycle at greater expense and often with greater uncertainty. It is therefore often stressed that the pregrouting should yield satisfactory results before the cycle continues, preferably without delaying the propagation of the front.

Sometimes, often when tunnelling at shallow depth, sealing can be performed by drilling from the surface into the volume that is to be excavated and inject sealant from there. This form of grouting is considered a last resort for troublesome rock masses or when an unshielded Tunnel Boring Machine, TBM, is used.

1.1.3 Environmental concerns

In any stable ecosystem the hydrological cycle is an important part of the ecology. Any changes in hydrogeological conditions will have an impact on the ecosystem. Depending on the sensitivity of the ecosystem, the largest acceptable perturbation of the hydrogeological cycle may be very small. All water that drains from the ecosystem would have played a part somewhere else. Since the water pumped out of an underground site is not usually reinfiltreated directly above the drainage site, a local drainage of water will occur in the vicinity, either directly above or downstream. This may lead to a lowered ground-water table and a reduction or change in the vegetation in the affected
1.2 Objectives

A lowered ground-water table can have other severe consequences in an urban area. Foundations on clay or wooden piles are dependant on a minimum ground-water level. If the drawdown due to drainage brings the ground-water table to a level below this level settlements in clay and rottening wooden pillars would ensue causing permanent damage to the supported structures.

The lack of sealing may cause a drawdown of the drown water level but the act of sealing itself may cause environmental damage. This is not surprising for chemical grouts but cementitious grout has very high pH and may cause local damage if it spreads outside the rock mass. Additives and byproducts from the grout may also leak into the ground-water and render nearby wells unusable.

1.1.4 Current Problems in Grouting Research

The goal of the grouting procedure is to quickly seal the rock around the tunnel by letting the grout fill all voids surrounding the excavation perimeter. This will form a tight seal around the rock mass that is to be removed. The grout is injected through bore holes and it should spread sufficiently to fill all accessible voids between two adjacent boreholes. It should not, however, spread in a manner that would cause the grout to move away from the excavation, filling voids that are unnecessary to seal from a construction standpoint. This is not only a waste of grout but may also lead to public relation problems for the operators if the grout were to leak through fractures to the surface as have happened on some occasions. Controlling the spread of grout inside the rock mass is of high importance if a predictable grouting result is to be achieved.

When trying to achieve optimal grout spread, the parameters available to influence are usually the choice of grout mix, bore hole spacing, pressure applied and time or volume spent pumping in each bore hole.

In what is often called "the Norwegian method", the pressure is chosen to be as high as possible in order to achieve the desired grouting result. Other strategies are often more focused on volume of grout injected or the time spent grouting and at lower pressure. In this thesis the mechanics of grouting are modelled to try to tell these different strategies apart in order to determine when and why a certain strategy could or should be used. One of the most current problems is how the grout deforms the surrounding rock mass and how this will affect the sealing result.

1.2 Objectives

These are the general objectives of the thesis.

- To summarise some relevant problems concerning sealing of modern un-
derground constructions, especially pertaining to grouting of such constructions.

- To propose how to model grouting induced stress and the consequences of such stress changes.

The key issue is how to model jacking in a way to make the model consistent with the measurements presented in chapter 6 and to determine under what circumstances jacking is to be considered a failure or a tool. This thesis does not contain the definite answer to these questions but proposes one possible way to ask the right questions.

\subsection*{1.3 Extent and Limitations}

This thesis does not cover or intend to cover the problems surrounding grouting in soils, clays or poor quality rock.

The focus of the research in this thesis is on the long term goal of the Swedish Nuclear Fuel Waste Management Corporation, SKB. It therefore mainly concerns the mechanics and problems of grouting in high quality hard jointed rock with high probability of water bearing fractures. The final repository will be located at great depth but the construction of the repository will start at ground level, so this thesis tries to model not only situations with high confining stress and high water pressure.

This thesis does not in any way cover the problems related to grout properties, such as stability, filtration or additives. For all aspects of this thesis, cementitious grout is considered to be a highly viscous Newton or Bingham fluid, depending on the situation. This makes most models and assumptions valid for other sealants as well.

It is not possible to cover all previous research on fractures and the coupling between hydrological properties and mechanical properties. The literature review in that field has been extensive, but very little of the previous research is applicable on grouting problems the way they are described in this thesis.

There are also many other ways to tackle the problems involved and many other aspects of them that are not covered or mentioned.

\subsection*{1.4 Definitions}

The \textbf{Critical pressure} is the pressure applied when a large change occurs. For most instances the change is a sudden dilation of a fracture, but it could also be a hydrofracturing event or failure of overburden. A rock mass may have several critical pressures and they may have any order.
Achievable radius or achievable distance, usually designated $R$ or $I_{\text{max}}$, this is the maximum distance a Bingham fluid can travel in a conduit. This is usually not the same thing as the penetration distance $r_g$ that is the actual distance from the injection point to the rim of the grout spread pattern.

Hydraulic fracturing is the formation of new fractures, or propagation of existing fractures, due to an applied hydraulic pressure. Hydraulic fracturing may take place during jacking and vice versa but there is a strict difference between the two concepts.

Jacking or hydraulic jacking, is a collective name for sudden pressure induced movements in a rock mass. It is used in the sense of sudden dilation of a single fracture due to an applied internal overpressure. The jacking can either be reversible, elastic jacking, or irreversible, plastic jacking.

The load-affected volume is used as a name for the volume of rock that experiences strain or changes in strain due to the grouting operations.

Sealing efficiency is a measure of the success of a grouting operation. It is usually measured as the percentage of inflow removed from an underground excavation.

Uncontrolled grout flow is a condition where the flow of grout will deviate from the desired spread. Most often the desired spread pattern is a cylindrical 2D pattern with the borehole in the center. An example of uncontrolled grout flow is when the grout finds an unusually large conduit leading a long distance away from the zone intended to be sealed.

1.5 Reading Instructions

The target audience for this thesis are individuals with some background in rock mechanics and cursory knowledge in recent research in the field, especially the research performed here in Sweden.

Chapter 2 tries to establish the fracture as the key entity. The mechanical properties of a rock mass is mainly the function of the fractures in the rock mass. The inflow that is to be sealed takes place inside the fractures, not the rock matrix. Single features of fractures are modelled with easy models but for a coupled problem, these models will be too limited.

The chapters 3 and 4 are meant as a basis for the concepts needed to understand how grouting is modelled and the how the grouting result is predicted.

Chapter 5 returns the topic to the single fracture and describes which different mechanical effects that are necessary to accommodate in order to predict the mechanical result of an internal overpressure inside a fracture.

Chapter 6 describes the conceptual model for grouting induced rock strain. This is the key chapter of the thesis.
1. Introduction
2. SINGLE FRACTURE REVIEW

2.1 Introduction

The properties of a rock mass are often more dependant on the fractures in it than the rock matrix around them. A fracture is an entity that is not characterised or described very easily. It does not consist of anything other than the void between its walls, but it can still have an abundance of properties, some more elusive than others. Most of a fracture’s properties are dependant on the geometry of the void and that is, in its turn, dependant on the history of the fracture and the rock matrix.

In order to derive and describe a conceptual model for high pressure grouting it is first important to understand the basic models of fractures and fluid flow in fractures. These models are the basis for the mechanical models and the prediction models that will be presented later on in this thesis.

The most basic model for inflow calculations is the large well formula

\[ Q_l = \frac{2\pi K h}{\ln \left( \frac{2h}{r_w} \right)}. \]  (2.1)

This representation uses the assumptions that there is no drawdown above the tunnel and that the hydraulic conductivity of the rock mass, \( K \) is constant and isotropic. This assumes that the rock mass is a porous, homogenous medium. This is not a very accurate description of a rock mass that contains a normal amount of fractures. Intact rock can for all intents and purposes of this thesis be considered to have zero hydraulic conductivity. Fluid flow can only occur in fractures in the rock mass. For the large well formula to be a reasonable approximation, the rock mass must either be completely intact, or very fractured to be considered homogenous.

One way to describe the rock mass as a homogenous medium is the concept of REV, representative volume. REV is the volume of rock that contains enough fractures to be considered a homogenous medium. The number of fractures needed is on the order of thousands and REV gives a scale which if it is much smaller than the depth \( h \) indicates that the large well formula may be a reasonable approximation for inflow calculations.

For high quality granite rock REV may become very large, often much larger...
than any reasonable scales for which the rock mass can be considered to have constant hydraulic conductivity. In this case the large well formula becomes more and more inaccurate. For a thorough description of REV see Min (2004)\textsuperscript{31}.

A different approach to continuum models is to look at every single fracture as an individual flow path into the tunnel. If the flow through each fracture is known and the distribution of fractures intersecting the tunnel is known, the inflow to the tunnel can be calculated with the same assumptions as for the large well formula. Neither the flow inside a single fracture nor the distribution of fractures intersecting a tunnel are easily calculated, though the subject remains an active research topic\textsuperscript{27}. With the fracture approach, the exact flow path of the fluids is not important, only the throughput. The throughput is calculated with the hydraulic head and the flow resistance in each fracture.

### 2.2 Fracture Models

#### 2.2.1 The Parallel Plate Model

The most simple and effective model of a fracture is the parallel plate model in which a fracture is modelled as the void between two flat, smooth and parallel surfaces without contact.

The advantages of the parallel plate model is that it is easy to grasp and has analytical solutions to the flow equations for both water and grout\textsuperscript{29}. The downside of the model is that it does not faithfully represent any mechanical properties of the fracture, such as fracture stiffness or aperture variations. The first efforts made to append additional properties to this model, from a grouting perspective, was Hässlers\textsuperscript{31} \textit{Piece of cake}-model. There the part of the fracture walls that are in contact with each other are accounted for. The model models grout take more accurately than the Parallel plate model but does not add any other hydromechanical properties to be predicted more accurately.

The Parallel Plate model always yields rotationally symmetrical flow patterns for grout. Fluids in real fractures with small and varying apertures do not flow in that fashion. The variations in aperture guides the flow of fluids in the fracture into "channels"\textsuperscript{46}. This channelling effect can be clearly seen when a water-conducting fracture intersects an underground excavation, The majority of the water seeping through the fracture comes at a few discrete points along the fracture trace.

### 2.3 Measurements on Fracture Geometry

There are many properties of fractures that can be of interest. The location, size and orientation are the most common properties that can be charted. There
are also several hydraulic tests that can be performed on a fracture or a set of fractures. A fracture’s most important property is, in many aspects, its aperture. The aperture of the fracture governs the water-bearing capacity and the groutability of the fracture. In general there is also a high correlation between the aperture of a fracture and its spatial extension in the other directions. A fracture has an aperture in every point. These apertures form distributions and can be correlated. If the aperture in a single point is measured, it does not really describe the fracture in any other way than that the maximum fracture aperture is at least that large. Such a measurement is often called the mechanical aperture of the fracture even though such a measure cannot be uniquely defined.

Most measurements on the geometry of a fracture surface involves a measurement on the topology of the surface, either via a profilometer or more advanced methods. A different and often more common method to determine the aperture of a fracture is the hydraulic aperture. By assuming that the parallel plate model holds and that fluid flow in the fracture can be described by Darcy’s law the aperture can be calculated from the following formula

\[
Q = -\frac{\rho W g W b^3}{\mu W} \nabla \varphi.
\]  

With this definition of fracture aperture, the aperture is well defined as long as the width, \( W \), that the flow is measured over is chosen properly. This definition of aperture does not correspond well with other definitions of aperture and is also only valid for calculations of water flow, or other Newton fluids. Measurement of the hydraulic aperture also requires that the hydraulic gradient \( \nabla \varphi \) is known. If a fracture is open into an ongoing excavation, the hydraulic gradient is time dependent and may be somewhat elusive. Nevertheless the hydraulic aperture has the advantage of being easy to measure in a more repeatable way. If only one scalar measure is to be used to describe a fracture the hydraulic aperture is the one that makes the most sense.

2.3.1 Fractal and Self-affine Models

Since the advent of chaos theory, fractal measures have become increasingly popular when characterising irregular and complex geometries. The surface of a rock fracture consists of clusters of crystals, themselves clustered together in larger and larger formations. It is apparent from looking at pictures of rock outcrops or rock surfaces that there are scaling phenomena at work. If no reference objects are placed in the picture to show scale, the dimensions of the images can be very challenging to determine. The very nature of the rock seems to indicate that fractals could be an effective tool in trying to describe the geometry of fresh rock surfaces.
Fractal mathematics is however not as simple as it may seem at a first glance. Measurements with fractal methods require the utmost care, both in the actual measurements and the interpretation of the data. The number of works that have tried to describe different fracture properties with fractal methods are very large. Despite all these research efforts there has been very little progress in this field. The early results of Brown and Scholtz (1985) have not been improved despite several efforts (see figure 2.2). This is in the author’s opinion largely due to the inherent but unapparent difficulties in the required analysis.

2.3.2 Fractal Dimension

The most distinguishing new concept of fractal theory is that of fractal dimension or Hausdorff dimension. It is a scaling property of fractals that is distantly related to spatial dimensionality but should not be confused with such dimensionality in any manner. The Hausdorff dimension and the fractal dimension are purely mathematical concepts unlike the topological dimension.

For non-fractal entities the fractal dimension is always the same as the spatial dimension, but for an object with fractal properties the fractal dimension is not an integer. A rock surface with fractal properties has a fractal dimension between 2 and 3. The value of the dimension is closely related to the correlation behaviour of the surface. It can also never be less than 2 or more than 3.

The fractal dimension of the fracture surface is connected to the correlation behaviour of the fracture surface, which in turn is connected to the stiffness behaviour of the surface (see figure 2.1).

It is possible that the fractal dimension can explain some behavioural phenomena of fractures under normal load, but that would be the subject for an
entirely different thesis.

An accurate model of the geometry of a fracture with a small number of measurable parameters would be of great value for the advancement of fracture mechanics. No model that could be used for all relevant mechanical problems has been found in the literature.

If all inherent difficulties with the model can be overcome once and for all, a fractal and self-affine model could be a very interesting model for describing several of the mechanical properties of fractures.

\subsection*{2.3.3 Fracture Statistics}

The topographical data from a fracture measurement is usually analyzed with either Fourier methods or autocorrelation methods. The result for a Fourier analysis is usually a plot looking like figure 2.2.

A power-spectrum plot of topological data will usually have a power law resemblance in some part of the spectrum and a gradual transformation into white noise at higher frequencies. In order for it to really be a power-law spectrum the plot must be linear over several orders of magnitude. The problem with determining the range and proportionality for a power law relationship from a power-law plot is that, since it is a log/log-plot, a vast majority of the samples will be in the upper end of the spectrum, which is dominated by noise and sampling artifacts. The proportionality constant calculated from such a plot is therefore extremely sensitive to the choice of high frequency cut of or any noise filtering methods used.

The autocorrelation function of a surface is closely related to both the power spectrum and the Kriging function of the surface. The way the autocorrelation function tends to zero can also be related to the fractal dimension. The autocorrelation function also has the advantage that it averages over a large number of points, reducing its sensitivity to noise. If done in 2 dimensions, both the autocorrelation function and the Fourier Power function will reveal any anisotropy in the data (see figure 2.3).

If the fracture is anisotropic, it is likely to have undergone shear and there is a higher probability for a high, stress-independent, residual transmissivity. When the fracture surfaces move relative to each other, they become unmated, creating larger open orifices that will remain open even during high normal loads\textsuperscript{20}. This will result in a high residual flow that will be unaffected by increases in normal load.
The concept of effective stress is a key concept in soil mechanics. In soil mechanics the effective stress is defined as the difference between the total stress and the pore pressure

\[ \bar{\sigma} = \sigma - u \]  

(2.3)

The main difference between soil mechanics and the mechanics of fractures, in this case, is that in soil mechanics the contact surface between the soil particles is assumed to be a negligible part of the cross section surface. In a fracture that assumption is not valid since the part of a fracture’s surface area that is in contact can be a significant part of the total area. In a paper from 2000 Pyrak-Nolte and Morris shows that the contact area between two fracture surfaces can be in the range of 0 to 35 percent of the total area and also...
that it is highly dependent on the normal stress. A more suitable representation for the effective stress of a fracture might be

$$\bar{\sigma}_c = \frac{\sigma}{A_c(\sigma)} - u\left(\frac{1}{A_c(\sigma)} - 1\right).$$

(2.4)

Here, $A_c$ is the relative contact area of the fracture and $\bar{\sigma}_c$ the average stress in the asperities in contact. This relation remains to be proven effective. It is also important to distinguish this equation from the equivalent expression for granular soils:

$$\bar{\sigma} = \sigma \cdot A_c - u(1 - A_c)$$

(2.5)

If the fluid pressure is exerted by a grout pumped into the fracture the grout does not form a load carrying structure through the matrix, as water would in a granular material. Thus the grout only carries load across the fracture but does not unload the rock or give any buoyancy to the rock mass. The relative contact area $A_c$ is practically unknown and not easily defined with direct methods. This makes the concept of effective stress in rock somewhat complicated but with the formulation in equation 2.4 will aid in the modelling in chapter 6.

2.5 Summary and Discussion

The nature of the geometry of a fracture is very elusive. Many authors have attempted to describe the geometry of different fractures with different methods. The results have always been inconclusive and difficult to compare. One of the problems is related to the scale. A fracture can be very large but will
always have a lot of detail at minuscule scales. A consistent measurement of a large fracture all the way from the full size of the fracture down to microscopic levels would be an extremely difficult undertaking and would not necessarily say anything about the geometry of other fractures.

The concept of fractal or self-affine geometries shows great prospect for describing fractures, but the difficulties in measuring a fracture are augmented by the difficulties of a simple-looking but very complex mathematical theory. If the result of a measurement is to be the basis of a model for describing the geometry of a fracture, the span of the fractal parameters should not be as large as they are today. When the parameters obtained from measurements span the entire definition set the usability of the model becomes somewhat limited. There have been several efforts made and they make for interesting reading.

Even though there is room for improvement in the present models, they are more than sufficient for the kind of modelling that will take place later on in this thesis.
3. FLOW IN FRACTURES

3.1 Introduction

In this chapter some different grout-related models for fluid flow inside fractures are discussed. The main interest for research in this field is the propagation of water and oil in fractures but a lot of the research can also be applied to the propagation of grout. The inside of a fracture can be filled with mud, debris, minerals or other non-fluid materials but for most cases the fracture is considered to be empty or filled with water. Areas of interest for grouting-related problems are how to cope with anisotropy and how to utilise the cubic law and other results from "simpler" geometries when describing the properties of fluid flow in fractures.

3.2 Grout Spread Models

Grout is a complicated liquid with scale and time-dependent properties. For short times and large scales it can be considered to be a Newton fluid, but for small length-scales the shear strength of the grout will come into play and it will act like a Bingham fluid. As time passes the grout will harden and the shear strength will increase until the grout has solidified completely.

If the length scales are sufficiently small the grout can no longer be considered to be a fluid but rather a particle suspension. If the grout is modelled as a particle suspension in a Newton fluid, properties such as plugs in small apertures, suspension stability and filtering may be taken in to account.

3.2.1 Special Bingham Properties

A Bingham fluid differs from a Newton fluid like water in one respect: a Newton fluid has no relevant shear strength. If the strain rate in a flowing Bingham fluid is lower than the yield value of the fluid, there will not be any shear and that part of the fluid will flow with constant velocity and with negligible deformation.

The difference in rate of shear and velocity profile between a Bingham fluid
and a Newton fluid can be seen in figure 3.1. Where the shear is at a minimum the velocity profile of the Bingham fluid will be constant. This region is called the *plug flow region*. If the plug flow region is much smaller than the aperture of the channel the difference will become negligible\(^6\). For one-dimensional flow, such as flow in a wide, straight channel with parallel walls, there will be one significant difference. As the grout propagates through the channel the shear strength of the grout will counteract the pressure that propagates the grout forward. As the grout penetrates the channel, the area over which the shear strength of the grout will act increases. This will lead to a continuous reduction in flow rate until the flow completely stops. As the flow rate decreases the plug flow region will grow in size. When the flow stops the plug will fill the entire channel.

![Fig. 3.1: The velocity profile for a Newton fluid (top) and a Bingham fluid (bottom) in a parallel slit flow model. The shear rate is proportional to the change in velocity and where that change becomes too small, the Bingham fluid will act as a solid.](image)

For any given set of flow geometry, grout mix and pressure this will yield a maximum distance from the bore hole that the grout can travel. In some instances this will be the limiting factor of grout penetration.
3.3 Cylindrical Flow

If the parallel plate model is assumed, the grout will spread in a radially symmetric way from the borehole. This model is only valid for fractures with large\(^1\) apertures but is nevertheless interesting since it can be solved analytically. The understanding of the problem that the analytical solution to this model yields is vital to the understanding of the flow of grout in more complex geometries.

When the grout spreads radially from the bore-hole, into the fracture in this model, it will drop in velocity rather rapidly as it propagates away from the bore hole. This is not only due to the increasing area of the grout front but also due to the Bingham properties of the grout mentioned in the previous section. The difference between Bingham flow and Newton flow is however only relevant when the flow velocity is low. The maximum distance the grout can spread from the borehole is called the achievable radius, \(R\) and can be calculated with equation 3.1

\[
R = \frac{bP_E}{2\gamma_0} + r_w \tag{3.1}
\]

The borehole radius is in most cases several orders of magnitude smaller than the achievable radius and can therefore be neglected.

In reality the maximum obtainable penetration distance is smaller than the achievable radius. The grout will propagate very slowly towards the end and will not come close to \(R\) in any reasonable amount of time.

The solution to the time dependent problem is described in great detail by Amadei and Savage (2001)\(^2\) and Gustafson and Stille (2005)\(^29\).

3.4 Non-Symmetric Flow

The symmetrical grout spread pattern result from the parallel plate model is a consequence of the totally symmetric geometry of the model. A real fracture is not likely to have any kind of symmetrical properties in its geometry. The roughness and natural undulations in a fracture will create variations in the aperture of a fracture. With the cubic law (equation 2.2) giving a high precedence for flow in larger apertures the flow of grout will tend to flow in any direction where the aperture is larger. For a rough natural isotropic fracture this will still lead to a grout spread pattern that is only statistically rotationally symmetric.\(^28\).

\(^1\)Large in this context means that the aperture is larger than the local roughness of the fracture so that flow is not dominated by channel flow.
3.5 Network Models

Several authors\textsuperscript{46,9,55} have shown that the flow in a fracture mainly takes place in the high aperture areas of the fracture. This flow behaviour is called channelling and is an important property to consider when modelling grout flow. The formation of channels that will act as superconductors for the grout will radically alter the grout spread pattern from that which is predicted by the parallel plate model. If this is not incorporated in a model that tries to predict grouting results, the model will fail to accurately determine the risk of having too great variations in the sealing efficiency. One way of utilising the properties of channel flow is to model the fracture, not as a large open cavity, but as a connected network of channels or pipes.

Without too much knowledge on the mathematical rigours of correlation functions or self-affine geometry, it is still a reasonable assumption to say that the flow in a fracture is constant if you look at a small enough fraction of the fracture. In this local region of the fracture, the grout will flow in a way determined by the pressure at the boundaries. This fact is used to make a model that accounts for the geometrical variations in a fracture. A model that dissects the fracture into tiny regions, where the variation of the aperture is considerably smaller than the aperture itself, and then connects these regions in a flow network is the basis for all network models.

The basic equations for a network model are the equations for the conservation of mass and the pressure drop over each pipe. Together they will form a linear system of equations with the same number of unknowns as there are nodes in the network. This system can be solved either by iteration or by direct methods. The solution will be the steady-state solution where the shear stress of the grout balances the grouting pressure and all flow stops.

If the system of equations is solved by iteration a pseudo-time-dependent solution can be found. It is not the real time-dependent solution unless time derivatives are added to the equations, but it shows in approximately what order the pipes are being filled. The real time-dependent problem is a multi-phase problem that is difficult to solve both numerically and analytically. It it not clear whether the multi-phase flow at the grout front has any significant impact on the actual grout spread or not. A network model with Bingham flow is therefore often approximated as a sequence of hydrostatic states (see for instance Azevedo et al. (1998)\textsuperscript{4})

3.5.1 Shape of the Network

There is a caveat one should remember when solving a network flow problem for a Bingham fluid. The caveat is that the pressure at a node depends on the way the fluid propagated to the node from the origin. For a Newton fluid, there is no hydrostatic solution since there is no pressure drop along a pipe
unless there is a flow through the pipe. For a Bingham fluid it is therefore essential that the network accurately represents the fracture void geometry, at least statistically.

If there are no directional dependencies of the fracture properties, i.e. the fracture that is being simulated is completely isotropic, the network that represents the fracture should also be completely isotropic.

An ordinary rectangular mesh is extremely anisotropic, especially if all pipes are equal. For the grout to traverse from the starting point to a node $x$ in the network, the distance through the network is only the same as $|x|$ if $x$ is located on one of the main axes of the network. If $x$ is located at any other node in the network, the distance from the starting point to $x$ may be up to $\sqrt{2}$ times longer.

![Fig. 3.2: The difference between the theoretical penetration length (the arc) and the calculated penetration when using a square lattice network to simulate a parallel plate model fracture.](image)

The directional dependence of the network can be reduced. The dependence is greatly reduced if the pipes are selected with random transmissivity but the networks should preferably have a different design than an ordinary rectangular mesh. An unstructured, non-rectangular mesh, like a Penrose mesh has no directional dependencies and should yield favourable results.
3. Flow in Fractures

3.5.2 Length of Network Pipes

A network model has a lot of variables apart from the topology of the network. From a grouting perspective it is important to be able to simulate the flow in a fracture of the same size as one normally grouted. The limiting factor in the size of a network model is the number of pipes. For any given 2D problem the number of pipes will grow as the square of the diameter of the simulated fracture. The computational effort needed to solve a system generally grows as the cube of the number of pipes. It is thus important to chose a pipe length that gives as few pipes as possible but still resolves all relevant features of the geometry it should represent.

In a paper from 1985 Raven and Gale examined how the flow rate of water varied with increasing normal pressure for samples of different size. Although it is not mentioned in their conclusions, their measurements suggest that the flow of water through fractures follows the cubic law closely for flow distances up to the order of a couple of centimeters. For longer distances channel flow will be the dominant flow mechanism and the rate will drop below the cubic law. A network simulating a fracture with a radius of 5 meters would then have on the order of 25000 pipes. Such a system is solvable on any modern desktop computer.

3.6 Summary and Conclusions

Current models for describing grout flow in fractures are perhaps oversimplified in the sense that they do not model several aspects of the flow that could be included in improved models. The largest source of error in the current models is the lack of fracture-similar correlation behaviour and without that all other improvements of the current models may be in vain.

In that respect the current network models are the best available, as they have scaled down the number of parameters to a bare minimum.

When simulating flow in fractures the difference between Newtonian flow and Bingham flow is sometimes large and sometimes very small. This can be exploited to facilitate calculations for some situations when the difference is small.
4. PREDICTING GROUTING PERFORMANCE

4.1 Introduction

A grouting strategy involving jacking may yield a different result than a strategy that does not. This chapter discusses the means and limitations of inflow calculations in order to determine how to determine the actual grouting result. The methods used to predict grouting performance are based on assumptions and models of fluid flow with inherent limitations that must be acknowledged before any statements of grouting performance can be made.

Even with all the limitations of current methods, the effect of jacking may be evaluated. The results show that the usage of jacking may not always yield a favourable result.

4.2 Inflow Calculations

The inflow into an unsealed tunnel per unit length can be calculated using the large well formula

\[ Q_l = \frac{2\pi Kh}{\ln \left( \frac{2h}{r_w} \right)} \]  \hspace{1cm} (4.1)

This equation takes the depth of the tunnel, \( h \), and the tunnel radius \( r_w \) as arguments and requires that the hydraulic conductivity of the rock is known. The equation is derived under the assumptions that the flow is perpendicular to the tunnel, which means that the tunnel is infinitely long and that the ground-water level is undisturbed by the inflow into the tunnel.

If the tunnel is grouted, the sealed rock mass can be viewed as a shell of rock with lower conductivity. In that case the majority of the pressure drop takes place in the grouted zone and the inflow equation becomes

\[ Q_l = \frac{2\pi K_s h}{\ln \left( \frac{r_s + r_w}{r_w} \right) + \frac{K_s}{K} \ln \left( \frac{2h}{r_w + r_s} \right)} \]  \hspace{1cm} (4.2)

Most water bearing features in rock exist in planar zones intersected by the
tunnel. If the water supply to those zones is efficient enough the equations will produce a result that is reasonable close to the measured inflow\textsuperscript{39}.

Most tunnels are not driven underneath lakes or rivers which makes the assumption that the ground-water level should remain undisturbed difficult to achieve. This will overestimate the inflow to the tunnel. The resulting draw down will lower the water pressure at the tunnel which in turn will overestimate the sealing efficiency.

In most cases where water flows into tunnels, the ground-water level will decrease. When it does ground-water will flow in from surrounding regions, creating a cone of depression in the ground-water surface. As the ground-water table decreases the inflow into the tunnel also decreases. Eventually there will be an equilibrium where increased ground-water inflow and reduced water-consumption above the tunnel will equal the inflow into the tunnel and steady state will be reached. If there are no viable sources of ground-water recharge in the vicinity, the cone of depression may become very large and very deep. If the tunnel is located at an unfavourable place in an disadvantageous hydrogeological environment, it is quite possible for the ground-water surface to intersect the tunnel. In such cases, the sealing of the tunnel will only delay the inevitable drainage of the surroundings as the steady state solution may be completely independent of the conductivity of the rock mass.

4.2.1 Hydrogeological Budget

Weather or not the cone of depression will reach a steady state before it reaches the tunnel is dependant on the sources of ground-water recharge available at the site. The hydrogeological cycle of the environment is often at some sort of diabatic balance\textsuperscript{53} before the excavation begins. Any drainage into the tunnel will inevitably remove water from another water-consuming entity in the balance equation. Most often it is the ecology will have to adapt to the new conditions but there may also be wells in the area that will experience lower capacity.

The sources of ground-water in a region are rain and inflow from the surroundings. Ground-water is always moving and a temporary drainage will be refilled with water coming from higher ground and leave a cone of depression downstream. The sources are balanced by the outflow of ground-water to lower grounds and the evapotranspiration, i.e. the consumption of water by plants and the evaporation of water into the air. This yields an equation that, in nature, is balanced.

\[ \sum \text{sources} = \sum \text{sinks} \quad (4.3) \]

A tunnel or other excavation will add to the right hand side of the equation. Since most sources are unaffected by the adding of a new sink, the balance will be the result of a redistribution among the sinks\textsuperscript{34}. 

4.3 Uncontrolled Grout Flow

Ideally all grout would spread in a cylindrical fashion from a bore hole filling all voids in the rock within the achievable radius. In practise it is very difficult to ascertain the actual grout spread pattern. The only indication on how the grout has spread comes from when the grouting has failed in one way or the other. If the grouting fails to penetrate any fractures the resulting inflow will be unaffected but there is also the distinct possibility of a completely different scenario:

The spread of grout will follow the path of least resistance in the fractures. Under some conditions the grout will not spread in a way that will fill the entire fracture in a 2-dimensional way but rather follow one or a few large 1-dimensional features. Under those circumstances the grout can flow long distances. This may disrupt operations far away and the grout can even exit the rock at the ground surface. This type of grout spread will have a negligible sealing effect and is largely a waste of grout. Typical conditions for that kind of flow are fractures subjected to high shear strain. A technique for determining the grout spread dimensionality is described in Gustafson and Stille (2005).

4.4 Grouting induced Permeability Changes

If a fracture is opened by using high injection pressure there will be an increase in aperture for that fracture. The penetration distance is proportional to the achievable distance and the achievable radius is according to equation 3.1 directly proportional to the aperture $b$. The hydraulic conductivity however is proportional to the cube of the aperture (equation 2.2). A first hand analysis using equation 4.2 shows that an increase in radius of the grouted zone, $r_g$, will lower the inflow, but the increase in conductivity may have a larger and reverse impact.

If the conductivity in the grouted zone is unaffected by the increase in pressure, e.g. all groutable fractures are perfectly sealed, the numerator in equation 4.2 will remain unchanged. The denominator in equation 4.2 can be rewritten as the sum of two terms, the first reflecting the grouted zone and the second the ungrouted rock mass.

$$\ln \frac{r_g + r_w}{r_w} + \frac{K_s}{K} \ln \left( \frac{2h}{r_w + r_g} \right)$$ (4.4)

Since $r_w + r_g$ is much smaller than $h$ the second logarithm is virtually unaffected by the increase in $r_g$. That means that for the inflow to be reduced by an increase in $r_g$ the change in the first term must be larger than the decrease of the second term by the increase in $K$, which is the conductivity for the surrounding rock mass.
The first term is a logarithm which argument is always larger than one. For any increase in the argument, the logarithm will have a smaller increase. The second term hover is inversely proportional to $K$ and $K$ will change drastically with changes in $b$. When grouting the fracture that increases in aperture will not do so for just the part that is subjected to grout but for a distance far beyond the reach of the grout. The direct change in $K$ for the rock mass surrounding the excavated area may be difficult to estimate, and it may be lower than that predicted by the cubic law, but even if $K$ was to be directly proportional to $b$ the relative change to the second term would be larger than the relative change to the first term. So unless the second term is much smaller than the first to begin with, a deformation of fractures during grouting will lead to an increase in inflow to the excavation. For most realistic instances the two terms will be of the same order of magnitude.

In order for the opening of fractures to have a negative influence on the inflow, e.g., decreasing it, there are some conditions that should be fulfilled.

- The depth $h$ should be very low in order to make the second term as small as possible.
- The sealed zone $r_g$ should have been very small without the deformation and the deformation should increase it substantially.
- The change in conductivity in the sealed zone should be improved compared to a “normal” grouting.

The third item is possible if the grout spread pattern fills the fractures better or non-groutable fractures are compressed due to the higher normal stresses (see chapter 6). If the normal load on the non-groutable fractures already is high, they may already be compressed in such a degree that the increased load does not affect the flow. This is then probably the reason to why the grout cannot penetrate those fractures in the first place.

The first and the third item are compatible since the stress in the rock mass usually is much lower at shallow depth. The second item is however most often associated with conditions where all water-bearing fractures in the rock are under high normal load making them very tight and difficult to permeate with grout. Those conditions are more often associated with excavations at depth than shallow depth. The usage of high pressures deforming the rock at shallow depth are also associated with elevated risks of failure of overburden.

### 4.5 Summary and Conclusions

Grouting operations are often designed to achieve a specific goal in terms of maximum allowed inflow. This inflow is determined from operational requirements for the tunnel or the impact the drainage has on the surroundings.
The inflow without sealing is then calculated with the large well formula (equation 4.1). To achieve the goal the inflow often has to be reduced substantially. The reduction in inflow will yield the necessary sealing efficiency.

The errors in the approximations in the formulas used to do all this are very large, but the sealing efficiency can be seen as a logarithmic unit as the order of magnitude is the important issue. Together with the geologist’s assessment of the rock mass the difficulties and risks will determine what grouting strategy should be used to seal the rock. The prediction of the grouting performance is therefore not necessarily aimed at giving accurate numbers but more in separating the advantages of different grouting strategies in order to make an economically viable decision.

Residual opening of fractures may have an adverse effect on the inflow in a tunnel-wide perspective. This even though the grout penetration is greater.
4. Predicting Grouting Performance
5. FRACTURES UNDER LOAD

5.1 Introduction

This chapter is a review of the mechanical effects of applying pressure on the inside of a fracture. The different deformation modes of a fracture will be the basis of understanding in the following chapter. There are also different failure modes that must be understood in order to be avoided.

The term jacking is used as a generic term to describe any process involving sudden deformations or fracturing of rock due to excess overpressure inside fractures in the rock mass. In the processes described in this chapter the overpressure comes from pumping grout into the fractures but the same term is used to describe rock mass events under hydraulic dams etc. The processes usually involved in those may differ from the ones described in this chapter. In this thesis the definition of jacking is also more strict.

The question at hand is whether or not jacking is to be considered as a failure or a possibility to increase the sealing efficiency. The increase of aperture in the fracture that is being deformed will increase the penetrability of the fracture and the achievable radius of the grout spread, but it may also increase the transmissivity of the ungrouted parts of the fracture. The added space in the jacked fracture must also come from somewhere. If it is the result of a compression of the rock, it will change the stress distribution in the rock mass surrounding the excavation. This may affect the stability of the rock.

The change in volume may also be the result of a compression of an adjacent, ungrouted, fracture. In that case it may be more difficult for the grout to penetrate that fracture.

Before the conceptual model for the jacking of a fracture is presented, a few other mechanical properties of the fractures and the rock mass will be described.

5.2 Mechanical Modelling of Fractures under Normal Load

As a basic mechanical model a fracture can be considered as a set of linear springs wedged inside an opening in the rock. The springs will represent the fracture’s stiffness and will be able to transmit normal forces through the frac-
ture. This basic model can be used to develop a model for how a fracture behaves under normal load.

Fig. 5.1: A single, small rock sample with a fracture modelled as a rock rod with a spring (left). To the right there are two idealised stress-deformation plots. In the upper one, there is no stress until the spring starts to compress. When the spring is completely compressed, the rock starts to compress as well. In the lower one, the spring is set to zero stiffness. When the spring is fully compressed, the contact area will go from 0% to 100% in one single step.

With the stiffness of the springs being much lower than the stiffness of the intact rock, a uniaxial compression experiment of a small rock sample can be modelled as a compression of the spring or springs inside the rock. The shortest spring representing the fracture will compress completely before any relevant deformation of the intact rock takes place. In its simplest form the model is a single spring between two slender pieces of rock subjected to uniaxial compression. The result of such an idealised experiment can be seen in figure 5.1, the first part of the curve is the compression of the spring and the second part is the compression of the rock. If the size of the rock is sufficiently small the fracture will be almost completely closed and the contact area can be considered to be 100%.

Now consider two such small samples placed next to each other in a uniaxial pressure test. If the samples have different apertures the resulting stress deformation curve should look something like figure 5.2. the first part is the
linear compression of the springs. The second part of the curve comes after the smallest aperture has closed. The contact area in the fracture is now 50% and the stiffness of the fracture increases to half that of intact rock. The third part of the curve is when both apertures have closed completely and the stiffness is now the same as for intact rock.

![Diagram of rock fracture modelled as springs and stress-deformation plots](image)

Fig. 5.2: A larger rock fracture modelled as several rock rods with springs (left). To the right there are two idealised stress-deformation plots. In the upper one, there is no stress until the spring first spring starts to compress. When the spring is completely compressed, the rock starts to compress as well. In the lower one, the springs stiffness is set to zero. As the stiffness approaches that of intact rock, the contact area will approach its maximum value, usually around 30\% \textsuperscript{48}.

Using this line of reasoning to make a theoretical array of samples with different apertures, all being compressed simultaneously in the same uniaxial compression test apparatus, it is possible to imagine the behaviour of a real fracture with varying aperture being compressed in the same machine. As the number of rock samples in the array increases the stiffness of each spring can be set to approach zero. If the spring rate is zero the fracture will have zero stiffness until the smallest aperture is closed. The initial stiffness at that point will then be inversely proportional to the number of samples of intact rock of the same geometry.

There are at least three apparent conclusions that can be made from that model.
• There is a connection between the stiffness of a fracture and its geometry.

• If the fracture is to be closed completely the rock constituting the most protruding asperities, that is the rock around the smallest apertures, must be severely deformed

• The initial stiffness will be dependant on sample size.

In this theoretical model each asperity has been considered to be independent rock samples joined only to its neighbours by the undeformable face of the test apparatus.

A more complicated, but realistic model would model each face of the fracture as intact rock. Each rock face would then follow the elastic line equation, thus distributing the load from a single point of contact to its neighbours. A model very similar to this was presented in 2000 by Pyrak-Nolte and Morris\textsuperscript{48}. Since their model simulates a highly connected network of point loads on a surface with long range connectivity it is computationally very intensive. The main benefit of the more complicated model is that it better shows the behaviour of the fracture during high normal stress. It also becomes quite evident that it will be impossible to completely close the fracture without affecting the structure of the intact rock.

Another benefit of the connected model is that it shows that there is a dependency of the stiffness on not only the aperture distribution but also the spatial correlation of the asperities\textsuperscript{48}. The spatial correlation can be linked to fractal and self affine measures of the fracture. This would indicate that such measures could be important in rock mechanics and also that those measures could be measured indirectly with the aid of the fracture stiffness.

5.3 Opening of Fractures

When a fracture closes due to high normal loads the flow mechanisms inside the fracture shift to a channel dominated flow regime. Likewise does the opposite happen when a fracture is opened due to a high internal pressure. As the fracture faces are moving away from each other, the contact area between the fractures will diminish. As the fracture opens completely, the undulations will become smaller relative to the fracture aperture and the parallel plate model will become a better and better approximation. If a fracture is opened due to an internal overpressure, the grout spread inside that fracture can be expected to be more cylindrical and the sealing of that particular fracture can be expected to be very good. It is however not clear that the total sealing efficiency of the grouting session improves with the opening of a single fracture. If the fracture increases in volume, there will be an increased normal load on adjacent fractures which may lead to more adverse grouting conditions in those fractures.
5.3. Opening of Fractures

The fracture may also propagate which could lead to a higher conductivity in the rock mass surrounding the grouted zone.

In the previous section the deformation of a fracture under normal load was described. If no plastic deformation occurs that model will be completely reversible. The major part of the deformation will take place in the contact asperities. When grouting the fracture, the parts of the fracture that are not in contact will be loaded by the pressure of the grout. The stress distribution in the rock mass will then be completely different from when an empty fracture is unloaded in an uniaxial load experiment. This is described in section 6.3 and it is important to distinguish between these loading scenarios as they will yield entirely different results.

5.3.1 Possible Boundary Conditions

Unclear boundary conditions seems to be a reoccurring problem in rock mechanics. The discrepancy in functional expressions between different type of boundary conditions indicate that the boundary conditions are important.

A reasonable model for the deformation of a rock mass surrounding a grouted fracture should preferably be independent of the size of the system. If the system is made too large, or semi infinite, the stiffness of the rock will be very low. If the system is made too small there will be high residual moments acting on the edges of the model. A suitable model for the movement of an overburden or a rock slab between two fractures should preferably be fairly independent on how much unaffected rock mass is included in the model.

In section 5.2 in this chapter, a model for the stiffness of a fracture was described. If the analogy is extended a bit it can be used to describe a model which would yield a result that is fairly independent on the distance to the edge of the model. In this analogy the rock between two fractures can be described as a slab following a normal Kirchhoff plate model and extending far into the surrounding rock mass (see section 6.4.4). The load on the faces of the slab will be the grout and the fracture stiffness modelled as springs. For simplicity all springs are considered to be of equal stiffness, at least as a starting point.

The pressurised grout will bend the slab creating moments in the slab. In order to solve the Kirchhoff plate equations for this problem the moments or the bending shape at the edge of the plate must be known. This is however not possible in this case. Instead the springs covering the face of the fractures will present an opposite load that will cancel the load from the grout in a way that should cancel all bending moments inside the slab at some distance away from the grouted part of the fracture. The result should be a bending of the slab that will produce rapidly decaying moments and deflection in the slab as the distance from the grouted zone increases.
A special case of this model will be for a horizontal fracture underneath a plate that faces empty space, e.g., an overburden. In that case there won’t be a fracture repelling the load from the grout but the mass of the plate itself. The mass of the overburden can be viewed as a preload of the grouted fracture. Then it will be the ungrouted parts of the grouted fracture that repels the pressure from the grouted zone, by means of decreasing preload. For a plate with reasonable bending stiffness this will yield a counter load that is larger than the weight of the overburden directly above the grouted zone.

5.4 Propagation of Fractures

A fracture subjected to an internal overpressure subjects the surrounding rock mass to a bipolar stress field (see chapter 6). This stress field will introduce tensile stresses around the edges of the fracture. The tensile strength of rock is low and the probability that the fracture will propagate through the rock mass is considerable if the grout fills the entire fracture. If the grout does not fill the entire fracture the load on the fracture edge will be considerably lower.

In a continuum approximation the effect of a higher conductivity surrounding the grouted zone is not so great, especially if the sealing efficiency is high. If the rock mass is modelled as a connected set of fractures the result will be a higher degree of conductivity surrounding the grouted zone. This increase in conductivity may connect water-bearing fractures, that have been sealed at the tunnel wall, with other fractures that previously may have been dry and fractures that may have been grouted less efficiently.
5.5 Failure

The upper limit for grouting pressure in practice is often determined based on the load of the overburden. At least when grouting at shallow depths. A plausible scenario is that a horizontal fracture will act as a cylinder with the overlying rock mass as a piston. If the pressure applied in the cylinder is greater than the downward pressure from the piston, the piston will move upwards. A vertical displacement of the ground is an undesirable event that may have dire consequences. Depending on the quality of the rock this methodology may lead to an overestimate of the risk and an underestimate of the structural integrity of the rock mass.

If the overburden is to be lifted by the grout, the support on the sides must already have failed. This scenario is only likely if the rock mass is very fractured or consisting of blocks with none or low horizontal load. If that is the case, the amount of uplift possible is limited by the amount of grout that is injected. If a higher pressure than the weight of the overburden is desirable, safety against uplift can still be ensured by limiting the volume of grout injected in each round.

If the overburden is intact or is held together by high horizontal stresses the overburden can be viewed as a Kirchhoff plate deforming due to a cylindrical load from below (figure 5.3). This will however lead to a boundary value problem where the boundaries are not well defined.

At large depths a more probable failure mode is cave in. If the jacked fracture is situated too close to the excavation, the load might cause a rock burst or complete failure of the tunnel front. This may present a great working hazard but is perhaps less likely to cause damage to the surroundings.

5.6 Summary and Discussion

This chapter has been a brief overview of some mechanical modes of deformations for a rock mass consisting of intact rock and one or two fractures. The system is simple in the sense there is not many components involved and the deformations are small compared to the scales involved. The complexity and difficulties lies in that the boundary conditions are generally unknown as well as the geometry of the fracture. Without boundary conditions and known fracture properties there is no way to determine the stiffness of the system or any of its parts. With the principal modes of deformation in mind the next chapter will look at experimental results in order to determine the general stiffness behaviour of the system. The result will be a conceptual model for rock mass deformations due to grouting.
5. Fractures under Load
6. GROUTING-INDUCED STRAIN

6.1 Introduction

The deformation of a fracture and thus permeability of a fracture is highly dependent on the normal stress of the fracture. The deformation of the fracture and the surrounding rock mass can be modelled in various ways. It is important to understand the mechanics of this very complex problem in order to ask the right questions and to make the models accordingly. How a fracture deforms under load will not only affect the mechanical properties of that fracture but also adjacent fractures. It may also affect the hydromechanical properties in a non local manner.

The interaction between the applied pressure during grouting, the stresses present in the rock and the stresses induced by excavation creates a very complex stress field. That stress field may in turn affect the permeability of the rock mass. Due to the complex interaction simple models can be used to motivate almost any grouting strategy. With the range of conditions that can be found it is not difficult to imagine that a complex but, in some sense, accurate model also may yield almost any result. It is therefore necessary to always maintain a holistic perspective in these matters.

For all models in this chapter, the grout is assumed to fill a circular section of the fracture. This assumption will bring cylindrical symmetry that will facilitate the calculations. A less covering grout spread pattern will create less leverage on the fracture faces so the assumptions err on the safe side.

6.2 Preloaded Fractures

In a paper by Cornet et al. (2003)\textsuperscript{15} the results of a fracture jacking experiment are presented. The authors of that paper performed an in situ experiment on a fracture using both an internal overpressure from injected water and an external pressure from two flat jacks. Their findings where in line with other documented engineering experience\textsuperscript{22} and very interesting.

In figure 6.1 the load/deformation behaviour of the measured fracture can be seen. During the first stage of the pressure buildup phase there is no measur-
able deformation of the fracture. When the internal pressure reaches a critical pressure (4.6 MPa) the fracture opens. The erratic behaviour of the curve during the vertical jacking is probably due to the limited capacity of the pumping equipment. The jacking stops after approximately 7 µm deformation and the pressure starts to rise again. At higher pressures there seem to be several subsequent jacking events.

Fig. 6.1: The deformation of a fracture, measured perpendicular to the fracture face, subjected to an internal overpressure. From Cornet et al. (2003) 15

During the return phase, the pressure remains constant as the fracture closes. It should also be noted that the closing and the opening of the fracture takes place at nearly the same pressure. The residual aperture indicates that at least one of the jacking events may have been plastic.

The details of the curve indicate that there is something more complicated at work than what can be modelled with the standard solutions to different Boussinesque problems 45.

One possible way to model the jacking of a fracture is to view the fracture and its asperities in contact as a preloaded structure. For that system the load applied on the fracture faces by the injected fluid will unload the asperities in contact. When the pressure increases there will a change in how the normal load across the surface is transmitted but there will not be any additional stresses induced in the rock mass until the critical pressure is reached and the fluid transmits the same stress as the asperities. What happens beyond that point is dependant on how the rock mass will respond to the induced stresses.
In the measurements in figure 6.1 it is reasonable to assume that the pump reaches its maximum capacity as its trying to compensate for the increasing volume of the fracture opens. At approximately $27 \, \mu m$ aperture the rock mass stiffens and the pressure in the fracture starts to increase again. If the jacking behaviour is to be modelled with classical elastic models the pressure in the jacking fluid can not be used as a load on the fracture directly. At least not if the model is to yield results that mimics the behaviour in the measurements.

Therefore a different view is proposed. By looking at the unloading of the fracture and the loading of the rock mass as two adjacent but independent systems this behaviour can be explained without resorting to nonlinear models.

### 6.3 Unloading of the Normal Load on a Fracture Face

The unloading of the normal load across a fracture will be the basis of a conceptual model. The model will describe the sudden change in stiffness in the system without resorting to nonlinearities. The reasoning behind the model can be described as follows:

Consider a single fracture under normal load, with asperities transmitting that load across the fracture.

![Fig. 6.2: The stress in a rock mass transmitted across an empty fracture through the contact surfaces.](image)

The concentration of stress through the asperities will cause the asperities to become more compressed than the surrounding rock. For a rock fracture subjected to moderate normal load a moderate estimate of the contact surface will be between 1% and 10%\(^{48}\). If the fracture is empty the stresses transmitted
through the contact surfaces will according to equation 2.4 be 10 to 100 times higher than the stresses in intact rock some distance away from the fracture.

The volume of compressed rock will extend some distance away from the fracture as well, but the redistribution of stresses will cause this volume to be relatively small. For an overhand engineering estimate the compressed rock can be assumed to be a cylinder of rock extending into the rock 10 times as far as the actual aperture of the fracture\(^\dagger\). For hard intact rock of good quality the Young modulus can be assumed to be on the order of 50 GPa\(^2\). A normal load of 10 MPa on a 100 µm fracture in that type of rock will yield a compression of the contact regions and surrounding rock of 2 µm if the load transmitting area of the fracture is estimated to be 10% of the total area. This approximate calculation would have to be several orders of magnitude too small for the effect to have any impact on the grout flow in the fracture.

During grouting of this fracture the grout will first have to unload the contact surfaces of the rock. During this part of the grouting the pressure inside the fracture will rise from almost zero up to just below critical pressure without any additional stresses being added to the rock mass. The unloading of the contact surfaces will cause them to elongate approximately 2 µm, an almost unmeasurable distance unless in a well equipped laboratory.

![Fig. 6.3: The stress in a rock mass transmitted across a grouted fracture through the contact surfaces and the grouting fluid.](image)

When the critical pressure is reached, the load will increase and the stress will rise in the rock mass adjacent to the fracture and the rock mass will start

\(^\dagger\) A more suitable approximation would have been the distance between the asperities in contact. This distance is however not known. Since the maximum aperture of the fracture is the only measure of the fracture that can be estimated with any certainty, it will have to be the base of the calculations. The result is in agreement with literature values\(^2\).
to deform. The stiffness of a typical rock mass is much lower than that of intact rock, and the geometry is much larger. An added stress of 1 MPa to an intact rock sample of 1m length will yield a deformation of 100 µm, which is of the same order of magnitude as the aperture of the fracture. With this approach to describing the loading of a fracture due to an internal overpressure the unloading of the fracture will render a large part of the pressure unavailable for loading of the rock mass. Only the part of the pressure that exceeds the critical pressure will add load to the surrounding rock mass. The stiffness of the system affected by the overloading of the fracture is however much less stiff than the system transmitting load across the fracture. This will yield a model for the systems where the deformations are very small for all pressures up to the critical pressure and large for any additional pressure added to the system.

6.3.1 Critical Pressure and in situ Stress

The critical pressure is defined as the pressure when jacking occurs\(^{22}\). This is related to the normal stress across the fracture\(^{51}\) but they are not necessarily the same. In the experiment performed by Cornet et al. the confining pressure was set by flat jacks to 2.3 MPa. The fracture was nearly parallel to the flat jacks but no significant movement was measured until the pressure in the fracture was more than twice that of the flat jacks (see figure 6.1).

One possible explanation for this is that the contact surfaces may be much larger than assumed. When the hydraulic pressure inside the fracture reaches the in situ pressure, the stress in the contact surface will be the same as in the rock mass, not zero. To completely unload the asperities in contact the hydraulic pressure must be larger than the in situ stresses. If the contact
area is very small, the difference is very small, but if the ratio of contact area and hydraulically pressurised fracture face is 1:1, eg 50% contact area, the hydraulic pressure must be twice that of the in situ stress to completely unload the contact regions and open up the fracture to a substantial degree.

At that point the jacking may become progressive. A complete unloading of the contact area will most likely make those parts of the fracture available for fluid flow. As the fluid propagates into the previously unreachable parts of the fracture, the load on the rock mass will increase even though the hydraulic pressure remains constant. The actual behaviour will depend on the capacity of the pump. If the pump is unable to supply enough fluid to fill the emerging voids at constant pressure, the pressure in the fracture will drop. This is most likely the explanation to some of the oscillations in figure 6.1.

### 6.4 Stress Field Adjacent to a Grouted Fracture

A point load on a flat surface will create very high stress locally but the stress will diminish over distance. In the same manner the changes in stress will occur in different local regimes during the different stages of jacking.

During the initial stages the load does not increase but it is redistributed over the face of the fracture. The stress field will become less curved and the fracture’s ability to transmit shear loads will diminish. The extent of the affected zone will depend on the poisson ratio of the rock and the geometry of the contact surfaces of the rock. But in most cases it will not extend very far into the rock, the typical scale of the load redistribution will be of the same order of magnitude as distance between the areas of contact. This volume of rock will be very small and the changes in load will therefore not result in any large deformations.

The second stage will take place when the critical pressure has been reached. At that stage the rock can be seen as a elastic homogenous region and the deformations will be proportional to the excess pressure in the fracture. The pressure induced stress changes will however diminish over distance and the deformations will most likely be localised to a region of rock adjacent to the fracture. As the fracture opens the grout may penetrate further into the fracture, this will increase the area over which the grout will apply load on the fracture face, and the load affected volume will increase rapidly (see section 6.4.2).

If the load affected volume reaches any other features, such as a free surface or another fracture, the deformation behaviour may change rapidly. At this third stage, almost anything can happen. This is when the grouting process starts to change the load on other fractures, which may induce large movements governed mainly by the kinematic conditions at those places. If the load affected zone reaches the end of the fracture, the fracture may propagate. Hy-
draulic fracturing of the rock may lead to severe stability problems and should probably be avoided in most instances.

The third stage is probably to be considered to be a failure condition for most operations as almost anything could happen in a manner that is very difficult to control or predict.

It is however important to remember that even if there is negligible deformation of a fracture under normal load while grouting, the shear strength of a fracture is dependant on the load on the contact surfaces and it will diminish as those are unloaded. A fracture subjected to shear stress may slip at pressures far below the critical pressure, but that is a topic that has to be examined on its own.

### 6.4.1 Pressure Distribution while Grouting

The pressure distribution for grout flow is solved by Amadei and Savage for the 1D case and Gustafson and Stille for 2D cylindrical flow. As the flow drops in velocity the pressure distributions in both cases are close to linearly decreasing with the distance from the injection point.

For a large fracture partially filled with slowly moving grout the pressure distribution can be approximated with a conical load on the face of the fracture. As the grout propagates through the fracture the radius of the loading cone will increase but the "height" of the cone is governed by the grouting pressure.

### 6.4.2 Semi-infinite Model

Consider a solitary fracture inside a large rock mass och high quality rock with high strength. The fracture is grouted through a bore hole with a high powered pump that is able to maintain a preset pressure even at high flows. When the grouting pressure has exceeded the critical pressure the excess pressure will cause deformations. If the grout has spread in a cylindrical fashion and does not fill the entire fracture the critical pressure will only be exceed in the central part of the grout spread cylinder (figure 6.5).

As long as the load affected volume of the rock mass can be seen as a homogenous isotropic mass the stress can be approximated with the stresses given in the solution for a conical surface load on a semi-infinite mass\[^{45,30}\]. From that solution the axial stress can be written as

\[
s_{z}|_{r=0} = \Delta P \left[ 1 - \frac{1}{\sqrt{(\frac{r}{a})^2}} \right]
\]

(6.1)

With \( a \) being the radius of the loaded zone and \( z \) the direction perpendicular to the fracture plane. This function dropps of pretty rapidly. The stress induced
6. Grouting-Induced Strain

![Graph showing grouting pressure](image1)

Fig. 6.5: The grouting pressure \( P \) drops to zero as the radius \( I \). The pressure inside the fracture drops below the critical pressure \( P_c \) at the distance \( r_c \)

in the rock mass drops below 10% of the surplus pressure at a distance of 2\( \alpha \) from the fracture. The load affected zone can be approximated by a sphere with a radius shorter than the penetrated distance (figure 6.6). The actual shape is of course more complicated, but for a conceptual model it will give reasonable results without sacrificing comprehension.

![Sphere approximating load-affected rock](image2)

Fig. 6.6: The load-affected rock is approximated as the rock mass inside the sphere with radius \( r_c \), the penetrated distance \( r_g \) is longer unless the \textit{in situ} stress is very low.

The contents of this sphere will be the key in how the rock mass will react to the change in load. If the sphere extends to include other fractures, these fractures may react to the changes in load in different ways.

It should be noted that for any given fracture and grouting pressure the influence on the rock mass is inversely proportional to the \textit{in situ} stress squared.
This as an increase in *in situ* stress will decrease both the radius of the loading area of the fracture and the available pressure $\Delta P$.

If the fracture is completely filled with grout and the fracture extension is considerably shorter than the achievable penetration length the pressure inside the fracture can be considered to be constant. Under those conditions the entire fracture face will reach critical pressure at the same time. This will lead to an instant formation of a large influence zone. This scenario has no further positive outcomes for the grouting efficiency of that particular fracture but can be used as a tool to induce fracturing at the rim of the fracture.

According to Sneddon\(^{52}\) (eq. 3.3.11) the deformation of the fracture face due to an internal load and without fracture propagation has an analytical expression. If the internal overpressure is replaced with $\Delta P$ it will can be written as

$$u_z = \frac{4\Delta P}{\pi E} a \sqrt{1 - \frac{r^2}{a^2}} (1 - \gamma^2). \quad (6.2)$$

Equation 6.2 is the definition of an ellipse. For the shape of a natural fracture, most of the fracture will experience deformation of the same order of magnitude as the deformation at the center of the fracture, which is.

$$u_z = \frac{4\Delta P}{\pi E} a (1 - \gamma^2). \quad (6.3)$$

### 6.4.3 Fracture fracture interaction

If the load affected zone extends to include part of another fracture the load on that particular fracture will change. If the fractures are parallel the second fracture will see an increased normal load that will reduce its aperture. The amount of change in aperture is highly dependant on the *in situ* stress and the stiffness of the fracture. For a fracture under high normal load due to high *in situ* stresses, the change in aperture will be small but a change in *in situ* stresses will have a progressive effect. If the *in situ* stress is lowered, the fracture stiffness will decrease and the excess grouting pressure will increase. The combined effect will make the *in situ* stress the most important parameter for fracture-fracture interaction.

### 6.4.4 Plate Model

As previously discussed in section 5.3.1 the deflection of the rock mass can, under certain conditions, be modelled as a plate of rock bending under the increased load. In order to be able to use standard plate models, the stress $\sigma_{zz}$
should be independent of $z$. This condition can be considered to be fulfilled if
the radius of the load affected zone is larger, or much larger, than the thickness
of the plate. This will be the case when the radius of the loading area is larger
than the plate thickness.

When the rock mass reacts as a slab, the fractures adjacent to the plate may
dilate or contract for much larger distances than other cases. With this type
of modelling the implementation of realistic boundary conditions becomes of
pivotal importance for the outcome of the result.

6.5 Idealised Pressure-Deformation Relation

The three different stages previously discussed can be used to compile an ide-
alised relationship between the grouting pressure and the dilation of the grouted
fracture.

In the first stage the dilation of the fracture will be negligible, which can be
seen as an almost infinite stiffness of the fracture. As the critical pressure is
exceeded, the elastic deformation of the rock begins. The initial stiffness of
an infinite elastic body is zero, if subjected to a point load, so the change in
stiffness for the system may be very large and sudden. The system then acts as
a strain hardening system until the load affected volume reaches a new feature.
If the feature is a fracture parallel to the grouted fracture, the stiffness will
drop again as the next fracture compresses under load.

This behaviour will repeat itself over and over again until the grouting stops or
a failure occurs. As the grout penetration increases rapidly with the dilation of
the grouted fracture the changes in stiffness will occur more and more rapidly
(see figure 6.7).

6.6 Implications for Different Scenarios

Consider two different grouting operations, one at great depth and one in shal-
low rock. For the shallow operation the vertical load is low and the normal
stress on the fractures, particularly horizontal fractures, will be low. Possible
failure criteria for this scenario may include failure of overburden or grout leak-
ages. That scenario may have an idealised dilation-pressure plot looking like
figure 6.8

For the other scenario, the vertical load and rock stresses in general are much
higher. The critical pressure in this situation is likely much higher than in the
previous scenario. The probable failure modes at depth does not necessarily
include failure of overburden but crushing of rock and rock bursts into the
excavated area (see figure 6.9).
Fig. 6.7: In this idealised plot the grouting pressure increase until failure. When the critical pressure A is reached all asperities have been unloaded and the grouting pressure starts to induce additional stress into the rock mass. At pressure point B, the load affected volume has reached another fracture and starts to compress it. The same thing happens at point C. At the point D the overburden fails and the fracture is allowed to expand freely.
If the normal load is low the critical pressure will also be low. The load affected volume will therefore grow rapidly as the grout spreads in the fracture. The initial stiffness at jacking will therefore be very low.

6.6.1 Pump capacity as a limiting factor.

When calculating the force balance necessary to achieve failure under certain conditions, the pressure and the grout spread are used to calculate the load on the rock mass. If the product of those is larger than the opposition mobilised by the rock, failure is achieved. This calculation does however not take into account the strain that has to be achieved in order to mobilise all resistance. With this strain there is an energy associated. That energy has to be inserted into the system by the grouting pump. With that approach the time before failure could be calculated instead. While a typical grouting pump has the ability to achieve quite high pressures, it takes a very powerful pump to be able to maintain that pressure as the fracture dilates.

The idealised stress/dilation-curves in figure 6.8 and 6.9 does not take the time aspect into consideration. If the curve had been drawn as a parametric curve with respect to time it would probably look more like figure 6.1 as each increase
6.7 Possible Grout Flow During Jacking.

Jacking is often mentioned as a tool to open up existing fractures that are otherwise impenetrable by grout. Since the grout has to penetrate the fracture before opening it, this remains an open question. Jacking of a fracture will however lead to a more rapid and more cylindrical grout spread. When this is achieved, two items that differ from "standard" grouting becomes evident.

- The maximum achievable grout penetration distance, $R$, will increase. If the grouting pressure remains constant as the grout propagates, $R$ will

\begin{figure}
\centering
\includegraphics[width=\textwidth]{aperture_vs_grouting_pressure}
\caption{At greater depth the normal load and thus the critical pressure is higher. The adjacent fractures are also most likely very compressed which may lead to less opening of the grouted fracture before failure occurs.}
\end{figure}
also propagate as the load affected volume increases. This will make \( R \) approach infinity and the grouting will only stop if the grout hardens or the operator stop the pump.

- If the pump is stopped before the grout hardens, the grout front will continue to propagate as the fracture closes.

The first item means that if jacking is to be used as a tool, then the stop criteria must be revised and chosen accordingly. The second will be true for both Bingham and Newton fluids. The possibility of having the fracture aid the propagation of the grout could be an important tool but would require tight control of the events.

Up until jacking, the deformation of the fracture is negligible and the network models briefly described in section 3.5 should be sufficient to predict the grout spread. If the fracture is opened, the aperture becomes more or less constant for large parts of the fracture and the parallel plate model may be the most suitable model. At least if it is made to accommodate a pressure dependant aperture.

### 6.8 Discussion

The experiment performed by Cornet et al. was performed in a rather small geometry. A fracture aperture of only 20\( \mu \text{m} \) is small compared to what is normally considered a groutable fracture and the distance to the constraining flat jacks was less than one meter in each direction. The goal of their experiment was not focused on jacking from a grouting perspective but rather to measure the limitations of rock stress measurements.

The experiment is however important and it is reasonable to believe that the experiment and the results should scale well to larger scales. It would be very interesting to repeat the experiment in a scale and manner similar to typical grouting conditions.

### 6.9 Summary and Conclusions

A conceptual model has been developed, based on one of the few relevant experiments found in the literature. The model describes the fracture dilations that take place while grouting with high pressures. A conceptual analysis of the model, based on classical solid mechanics and rock mechanics has been presented. The conceptual model can be summarised as follows.

- At low grouting pressures, the dilation of the fracture and deformation of the rock mass are small and negligible from a grouting standpoint.
• At a pressure related to the in situ stress in the rock and the contact area, the fracture starts to dilate.

• At increasing pressure, the dilation continues. This dilation will be dependent on the loading of the rock mass and may take place in discrete modes of deformation. Initially these deformations will be elastic and reversible.

• As the pressure exceeds the structural strength of any kinematic failure surfaces, the dilation will be unlimited. Unloading of the pressure will yield residual plastic deformations.

Based on this model the implications on a grouting process has been discussed. Below critical pressure, the currently available models are applicable. For isotropic conditions there are analytical solutions available and for anisotropic conditions there are numerical models available that can be applied if the anisotropy is modelled correctly.

The deformations that will develop in the rock mass as the pressure exceeds the critical pressure will be dependent on several factors. The boundary conditions that govern the principal deformation modes are usually not known. The deformations will be elastic at first but may become plastic locally. If the pressure is lowered, the dilation may revert, at least partially. This will however influence the grout spread in a way that is not accounted for in current grout spread models. The pressure that will cause ultimate failure can in principle be calculated with current rock mechanical models.

Dilation of fractures during grouting will facilitate penetration of the fractures by the grouting fluid. As the penetrated distance is longer in the larger fractures, the load on the rock mass will be uneven. This will lead to a complex behaviour of opening and closing of fractures. The dynamic behaviour of the fractures and the grout spread will be dependent on the characteristic behaviour of the pump. The pump has to be very powerful in order to maintain the grouting pressure as the fracture dilates.

Under certain conditions the overall result of the grouting may be negatively influenced as large dilating fractures may close small fractures. The dilation of a fracture may also take place at distances far beyond the spread of the grout, which will affect the conductivity of the rock mass in a way that increases the inflow into the excavation.

Grouting pressures that will cause fracture propagation should be avoided as the process is difficult to control and will only seldom have a positive influence on the sealing efficiency. At what pressure the fracture starts to propagate will vary from rock type to rock type, but it is always higher than the critical pressure.
6. Grouting-Induced Strain
7. FUTURE RESEARCH

The main work so far has been focused on identifying and understanding different mechanisms that are at work while grouting. The result is a set of conceptual models for different phenomena. These models need to be examined closely in order to distinguish the conditions under which they may be valid. If the conditions and the models can be isolated well enough to make them easily distinguishable, the models might become ready for experimental verification.

The isolation of conditions that may lead to certain positive or negative results according to the models needs to be investigated thoroughly in order to implement the results as design criteria for real world grouting. This is probably the most difficult challenge of all, as every grouting operation is unique, and has its own problems and conditions. The rock mechanics of some type conditions should be the basis for analytical models of jacking. If the boundary conditions are resolved, elastic models can be formulated and integrated in the existing rock mechanic models.

The focus of the coming work should be to verify the existence of a critical pressure. If there is a pressure that marks the transition from a high stiffness regime to a low stiffness regime. This would mean a new way of looking at grouting pressures and it would be a good definition of what is to be considered high pressure grouting. This should be modelled more carefully with extended efforts to find applicable experiments and theories from related research fields. There are more experiments in the literature that could be reevaluated from with respect to this new model.

To measure movements within the rock mass that are on the order of hundreds of micrometers are not a common task in modern underground excavations. During grouting most measurements are of hydrological type and associated with either the inflow of water into boreholes before grouting or the flow of grout during grouting. These measurements yield little data regarding the changes in stress inside the rock mass and regarding any dilation of any fractures. There are ongoing research efforts that are looking into using seismic attenuation methods for measuring rock fracture stiffness inside the undisturbed rock mass. If these methods could be used to verify grout spread and sealing efficiency during grouting, the opportunities for improved grouting techniques based on active design will increase drastically.
A different method could be to look at the level of microseismic events during grouting. If the conceptual model is correct, the level of microseismic events should increase drastically once the critical pressure is exceeded. This would be a good indication in support of the conceptual model.

When the mechanics of jacking is understood, the effort can be shifted towards understanding the flow of grout inside the fracture during jacking. Will the reverse jacking process cause the grout to propagate further inside the fracture? Will the opening of the fracture aid the grout to fill the minuscule voids around any asperities in contact?

The time remaining in this project is too short for all these leads to be followed. The key issue remains to be the critical pressure but the decision on what to focus on beyond that can not be made at this time. A field experiment may become very large and time-consuming but it is ultimately that which may give answers to the questions that arise from the theoretical analysis. A less time consuming way could be to device a scale experiment that could be performed in a lab.
REFERENCES


Part II

PAPER I
A. EXPERIMENTAL MEASUREMENTS

A.1 Introduction

In August 2005 a fracture trace in a borehole at the Åspö hard rock laboratory was carefully photographed. In this chapter the experiment, the apparatus and the results are described.

The planning for the experiment started in spring 2004 when the opportunity for access to a tunnel at the Åspö hard rock laboratory arose. Several other experiments have been and are being performed in the tunnel and the co-scheduling of several experiments made it economically feasible to look at some of the several interesting phenomena manifested in the tunnel.

The tunnel was designed explicitly for the Åspö pillar stability experiment and both the excavation and the sealing of the tunnel where carefully designed, performed and monitored.

The tunnel is intersected by a water conducting fracture zone, that intersects at a close to normal angle parallel to the main stress direction. This zone was successfully grouted with a cementitious grout.

The aim of this experiment was to excavate a fracture in this zone to see if the grout spread in the fracture could be ascertained.

A.2 Measurement Apparatus

The measurement apparatus is basically a custom made high resolution borehole imaging system, but built from common consumer electronics and household items. The apparatus was inserted into a large core-drilled hole and manually navigated inside the bore hole.

The imaging system consist of a Canon EOS 350D digital camera with a Canon EF 100mm Macro lens and A Canon EZ-14 ring flash. The camera was mounted eccentrically in a 250mm diameter and 1000mm long lexan tube. A surface reflecting mirror was placed in a holder in front of the lens and a flash reflector was placed behind the reflector. The lens was chosen because of its ability to take high resolution pictures at a scale of 1:1 at a distance of 14cm from the
A. Experimental measurements

Fig. A.1: A schematic rendering of the measurement apparatus inside the borehole. The camera is mounted inside a tube with a mirror and a flash reflector in front. The tube rolls on ball bearings to facilitate operation.

front of the lens.

The camera was remote controlled from a computer outside the bore hole and all images were stored directly on the computer. The same camera equipment was also intended for usage when studying the surface of the fracture inside the core.

On order to facilitate the navigation of the tube inside the hole wooden spheres where placed between the tube and the fracture wall to act as a bearing.

A.3 Site Selection

The requirements for selection of a fracture to be excavated where that it should be in the grouted zone, and not conduct to much water as the camera would not operate if submerged.

A suitable candidate fracture was found in April 2005 when Ann Emmelin
(SKB), Rolf Christiansson (SKB), Björn Magnor (SKB) and Rikard Gothäll (KTH) visited the site. The fracture that was selected intersected several boreholes and the trace was visible on the tunnel wall from floor to ceiling.

The placement of a bore hole was made at approximately 1m above the tunnel floor to facilitate the drilling and measurement procedures. The hole was also given a slight upward tilt to ensure that there would be no buildup of water in the hole. This placement also put the hole roughly 30cm above one of the injection holes for the closest grouting fan.

A.4 Procedures

Drilling

The drilling took place in April 2005 and was performed by smålands betonghåltagnings under supervision of Björn Ljunggren from Thyrens.

During the drilling the core split several times in what could be described as a core discing manner. The presence of a fault zone and high stresses are probably the explanation for this. The broken core did however necessitate several restarts of the drilling since the drill had to be free from freely rotating rock. The restarts caused some irregularities on the wall of the borehole, but none of these irregularities have had any impact on the measurements or the results.

Measurements

The measurements were performed on August 9, 2005. The installation of the equipment went fast and smoothly, much thanks to the excellent support provided by the Åspö hard rock laboratory. Once the equipment was installed and tested, the actual measurements went quite fast. Each photograph covers the area of a small stamp and on average, more than two pictures per minute could be shot. Most of the time was spent navigating the tube inside the fracture. The usage of mirrors made the navigation somewhat counterintuitive and the fracture trace was often lost due to errors in navigation. As the operator got more used with the navigation, the time spent navigating dropped significantly.

The fracture was not completely dry due to a small seepage through the fracture and through the bottom of the hole. This posed no problem for the camera or optics, but the moisture caused the flash light to reflect in the fluid surface. This problem was remedied by re-balancing the light from the two flash tubes. Much of the moisture was also removed by a hot air fan that was set to warm up the rock surface inside the hole. The inflow was however not completely stopped by the heat and the rock was wet again in a few minutes. Unfortunately the fan and camera could not operate simultaneously. For the next version a
Fig. A.2: The fracture has caused an approximately 2 dm wide outcrop from the tunnel wall. The outcrop intersects an incompletely filled grouting hole that leaks a dark fluid. The white calcite is being deposited on the tunnel wall by water leaking very slowly from the fractures. The fracture in the image is parallel to the one in the measurements.

hot air fan could probably be integrated in the measurement apparatus. The camera also had some troubles focusing on the wet rock if there was no fracture trace in the picture to provide contrast. This could also be remedied with the usage of structured focus illumination.
A.5 Results

Fig. A.3: The drilling of the bore hole for the measurement. Right below the drill the remains of one of the grouting holes is visible, extending along the tunnel wall a few more meters.

The distance between the front lens and the fracture trace was approximately 16 cm for most of the measurement. At this distance, the imaging system gave an approximate scale of 1:1.4. One caveat that was difficult to ascertain that the focal plane was parallel with the fracture trace. The focal depth was large enough to accommodate most of the deviation but for future measurements a method for controlling this during operation should be developed.

A.5 Results

The fracture trace was very accurately imaged with great detail and the imaging process can be viewed as a success. The images are sharp and colourful and the fracture aperture variations are easy to gauge from the images.

The fracture in itself was however somewhat of a disappointment. That which from the fracture trace on the tunnel wall looked like a completely grout-filled
fracture turned out to be a calcite filled fracture. The calcite covers a large part of the fracture but there is also what appears to be a water conducting channel network inside the fracture. From the measured trace, no grout seems to have penetrated this channel network.

A.6 Discussion

The aim of the experiment was to try to ascertain the spread of grout inside the fracture and to try to chart the geometry of the fracture. If the conditions had been ideal, there could have been a correlation between ungrouted parts of the fracture and the geometry of the fracture. There was also some hope that filter plugs or separation of grout could have been found if those phenomena had been present.

Since no grout at all had penetrated the fracture the spread was obviously zero. The aperture of the fracture is not easily determined from the images. The difficulty lies not in the imaging but in the definition of the aperture. The fracture shows signs of diagenesis and filling of calcite. Should the aperture be defined as the empty space or the distance between the faces of original rock?

The mechanical aperture is as large as one millimeter in some places but the hydraulic aperture must be much smaller as the inflow along the entire length of the bore hole was just enough to wet the surface of the rock.
Fig. A.5: A typical image from the measurements. The imaged height represents an approximately 20 mm long patch of the borehole surface. The aperture of the fracture varies between about 2mm to zero. The definition of the aperture is also very vague.

A.7 Acknowledgements

The author would like to thank the personnel at the Åspö facility. The efficiency and quality of their assistance was exceeded only by their friendly and welcoming attitude. Everyone involved in the project made the visits to the facility both pleasant and highly productive. Thank you very much!