KTH Architecture and the Built Environment

Remedial Injection Grouting of Embankment Dams With Non-Hardening Grouts

Johan Lagerlund

Licentiate Thesis
Department of Civil and Architectural Engineering
Division of Soil and Rock Mechanics
Stockholm, Sweden 2009

TRITA-JOB LIC 2013
ISSN 1650-951X

Table of Contents

Abstract

The focus of this thesis is to study the possibility of injection grouting of embankment dams affected by internal erosion. Internal erosion is a process where certain soil material from an embankment dam is removed. This phenomenon occurs in the central core of the embankment dam. If the internal erosion is allowed to continue over a longer period of time, the dam might face a fatal situation. Since the dam core is washed out, larger voids are created, thus lowering the geotechnical stability of the dam. If the voids become larger, more seepage is allowed to pass and if more seepage passes, the internal erosion process is accelerated.

The central core in an embankment dam is preferably constructed with till. Till is a natural soil that origins from the ice age. The till contains a wide range of grain sizes, basically anything from clay to blocs. The mixture of grain sizes does however give the till characteristics that are highly desirable for a water retaining construction. It is cohesive, has a low permeability, a high angle of internal friction and can be found practically anywhere in Sweden. In an embankment dam the core is the water barrier. The core alone is however weak and cannot withstand the large external forces put on a dam construction. Because of this, several zones are constructed on both sides of the core. The first zone outside the core is the filter. The filter has no cohesion and is constructed with a coarser material than the core. Outside the filter, the shell is found. The shell is constructed with even coarser material than the filter and supports the entire dam structure. Outside the shell the riprap is found. The riprap protects the dam from erosive forces such as wave erosion, ice loads and heavy rainfalls.

The filters main task is to protect the core from being washed out. Since the till in the core has a wide range of grain sizes, a constant rate of seepage may start to move its finer particles (clay, silt). If the filter doesn’t catch these moving particles, a loss of material will occur. This is the basis for internal erosion.

If the till has a smooth particle size distribution curve it is less prone to internal erosion. The smoothness of the curve ensures that the different grain sizes involved are evenly distributed. The finer particles are mechanically locked in place by coarser particles, which in turn are mechanically locked by even coarser grains. Finally, the soil structure is more able to withstand the erosive forces provided by the seepage. If the finer particles aren’t mechanically locked and eroded by the seepage, the filter must be designed to catch them. Therefore, internal erosion occurs only if both the till and the filter flaws.

If the internal erosion is continuous, the loss of material must be replaced. By doing so without dismantling the dam, injection grouting can be performed. The grout will replace the lost core material and restore the dam. The type of grout can basically be divided into two sub groups:

1. Hardening grouts;
2. Flexible grouts.

Hardening grouts involves cement or sodium silicates. The cement binds the grout together, thus creating a hard, impervious intrusion that fills the void. Flexible grouts contain no cement that binds the material together. The material involved is soil of various particle sizes. Hardening grouts seem to be more commonly used than flexible. The basics of internal erosion show that hard intrusions can cause various problems to a dam such as low compaction of the core soil and fracturing.

For this thesis, an evaluation of different grouting techniques has been done. Experiments have been performed in order to examine whether flexible grouts can be used for remedial grouting of an embankment dam. Experiments have also been performed to reveal if hardening grouts cause damaging to an embankment dam. Therefore, two experimental series have been designed: Permeability testing and filter box testing.

Compaction grouting technique has been found to offer the biggest advantages when injecting a flexible grout. The primary reason for this is that compaction grouting technique does not cause cracking of the original core soil.

The permeability testing has been done in order to examine if a flexible grout can remain durable if grouted in a dam core. If a flexible grout is injected, it will be subjected to the same erosive forces as the original core soil of the dam. Because of this, the flexible grout must be tested in an environment where it can be surged. Different flexible grouts with different particle size distribution curves have been surged. Evaluations have been done by comparing the particle size distribution curves of the flexible grout before and after surging. The results from the permeability tests show that the original particle size distribution curve of the flexible grout has a large influence on its long-term stability. Thus, it is possible to obtain a durable flexible grout that remains at its injected location over a longer period of time.

The filter box tests have been done in order to examine how the injected grout affects the surrounding original core soil. Both hardening and flexible grouts have been used. The filter box models an embankment dam with both a core and a downstream filter. The central part of the core soil has been grouted with either a hardening- or a flexible grout. Results from these experiments show that hardening grouts generate cracks in the core soil. The primary reason for the occurrence of cracks is the shrinking ability of the hardening grout. Flexible grouts do not generate cracking but cooperates with the original core soil.

Sammanfattning

Den här uppsatsen avser att studera möjligheterna att med injekteringsteknik reparera fyllningsdammar ansatta av inre erosion. Inre erosion är en process där visst jordmaterial från en fyllningsdamm spolas bort. Detta fenomen inträffar i de centrala delarna av dammen. Om inre erosion fortgår under en längre tidsperiod kommer fyllningsdammens säkerhet mot brott att minska. Eftersom material transporteras bort, skapas håligheter inne i dammen vilket även minkar dess geotekniska stabilitet. Om håligheterna blir större ökar även läckvattenflödet. Ett ökat flöde leder till att erosionsprocessen accelereras.

Den centralt placerade tätkärnan i en fyllningsdamm är vanligtvis konstruerad av morän. Morän är ett naturligt material med ursprung från istiden. Moränjorden är bredgraderad med partikelstorlekar från lera till block. Bredgraderingen ger moränen egenskaper som gör den lämpad för användning som tätkärna i en fyllningsdamm. Den har hög kohesion, låg permeabilitet, hög inre friktionsvinkel och är vanligt förekommande i naturen. I en fyllningsdamm verkar tätkärnan vattenmothållande. Tätkärnan i sig själv är dock svag och kan inte själv motstå de krafter som verkar på fyllningsdammen. På grund av detta utformas en fyllningsdamm till att ha flera zoner på båda sidor om tätkärnan. Den första zonen utanför tätkärnan är filtret. Filtret saknar kohesion och utgörs av ett grövre material än tätkärnan. Utanför filtret ligger stödfyllningen som håller uppe dammkonstruktionen. Stödfyllningens material är grövre än filtrets. Utanför stödfyllningen finns olika former av erosionsskydd som slänt- och krönskydd. Dessa skyddar dammen från erosionspåverkan av till exempel vågor, is och regn.

Filtrets huvuduppgift är att skydda tätkärnan från urspolning. Eftersom moränjorden i tätkärnan är bredgraderat är det känsligt för genomströmmande vatten. Då vatten passerar genom jordkroppen kan dess finpartiklar suspenderas i flödet. Om filtret inte fångar dessa suspenderade partiklar kommer en materialförlust att inträffa. Detta är grunden för inre erosion.

Om moränjorden har en jämnt fördelad kornfördelningskurva är den mindre känslig för inre erosion. ”Mjukheten” på kurvan försäkrar att de ingående kornstorlekarna är jämt fördelade i jordkroppen. De finaste partiklarna låses fast av de något större, vilka i sin tur låses fast av än större partiklar. Om någon partikelstorlek inte är låst uppstår det en risk att dessa förflyttas och det ligger då på filtret att fånga dessa. Därför uppstår bara inre erosion då relationen mellan filter och tätjord brister.

Om den inre erosionen fortgår bör det förlorade materialet ersättas. För att ersätta förlorat material utan att gräva ur dammen kan reparationsinjektering utföras. Bruket ersätter det förlorade tätkärnematerialet och återställer tätkärnan. Det finns primärt två typer av bruk:

1. Härdande bruk och,
2. Flexibla bruk.

Härdande bruk innehåller cement eller vattenglas. Cementen binder ihop brukets olika beståndsdelar och bygger upp en hård, impermeabel kropp som fyller upp håligheten. Flexibla bruk innehåller ingen cement som binder bruket. De ingående materialen i det flexibla bruket är jordpartiklar av olika storlekar. Härdande bruk verkar vara mer vanligt förekommande än flexibla vid reparationsarbeten. Härdande bruk kan dock skapa olika följdproblem för dammen som låg kompaktering av omgivande tätjord och sprickbildning.

En utvärdering av möjliga injekteringsmetoder har utförts. Försök har utformats för att undersöka om flexibla bruk kan användas vid dammreparationer. Försök har även utförts för att undersöka om härdande bruk orsakar följdproblem för en fyllningsdamm. På grund av detta har två försöksserier utformats: Permeabilitetstestning och försök med filterlåda.

Kompakteringsinjektering har funnits ge de största fördelarna vid damminjektering med ett flexibelt bruk. Huvudsyftet med detta är att tekniken inte orsakar uppspräckning av befintlig tätjord.

Permeabilitetsförsök har utförts för att ta reda på om det är möjligt för ett flexibelt bruk att ha en god långtidsverkan i en damm. Om flexibla bruk används kommer det att utsättas för samma erosionskrafter som den ursprungliga tätjorden. På grund av detta måste det flexibla bruket testas i en miljö där det kan genomströmmas av vatten. Olika flexibla bruk med skilda kornfördelningskurvor har testats. Utvärdering har utförts genom att jämföra brukens originalfördelning med utfallet av kornkurvorna efter en tids genomströmning. Resultaten från permeabilitetstesterna visar att det är möjligt att minska ett flexibelt bruks känslighet mot inre erosion genom att modifiera dess kornfördelningskurva. Det verkar därför möjligt att utforma ett flexibelt bruk som kan verka långtidsstabilt för dammreparation.

Filterlådetesterna har utförts för att undersöka hur ett injekterat bruk påverkar dammens omgivande, ursprungliga jordstruktur. Både härdande och flexibelt bruk har använts. Filterlådan modellerar en fyllningsdamm i det avseende att den har både tätkärna och nerströmsfilter. Den centrala delen av tätkärnan har injekterats med både härdande och flexibla bruk. Resultaten av dessa försök visar att härdande bruk ger upphov till sprickor i tätjorden. Den primära anledningen till detta är det härdande brukets krympning. Flexibla bruk uppvisar inte dessa egenskaper utan verkar som en del av den ursprungliga tätkärnan.

Preface

The thesis is treating the questions about remedial injection grouting of embankment dams.

The thesis is divided into two main parts. The first part, *chapter 2 – chapter 4* constitutes a literature review of the embankment dam, the internal erosion process and possible grouting techniques. The literature review points out certain factors that are necessary to know in order to design experiments. The internal erosion process has in particular been helpful when to determine important parameters of a grout material. Without this knowledge, laboratory work wouldn't be possible. The second part, *chapter 6 – chapter 7* constitutes the performed laboratory work. The last chapter concludes the thesis and point out the guidelines for future work.

The work has been carried out at Vattenfall Research and Development at Älvkarleby, Sweden and at the Division of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm. Daniel Eklund, PhD from Vattenfall Research and Development and Professor Håkan Stille at the Royal Institute of Technology, has supervised the project.

The project is financed by ELFORSK AB and Vattenfall AB, hydropower and has been a part of research projects within the frames of Swedish Hydropower Centre, SVC.

Acknowledgements

I would like to offer my warmest thanks to my to supervisors Daniel Eklund and Professor Håkan Stille. Without their guidance this project would be impossible. A special thanks to Peter Viklander Vattenfall, Håkan Bond WSP, Martin Johansson Skellefteå Kraft and Malte Cederström Vattenfall who have given me very good advices and provided me with experience based facts that I do not personally have. Professor Anders Wörman Royal Institute of Technology for his help in finding fruitful literature and discussions about internal Erosion. Hans Rönnqvist Vattenfall for his help and ideas when evaluating laboratory results. Anders Salomonsson at the Swedish Geotechnical Institute for finding and providing me with literature.

I want to thank the staff at the Civil and Materials engineering at Vattenfall Research and Development for making my work situation better. I'd like to point out Niklas Hansson who has helped med with measurements during the laboratory work during times I couldn't be there. Peter Skärberg who always find optimal solutions to various practical problems. Susanne Ericsson who has helped me to solve all the problems that comes with the responsibility of administration.

Thanks to the Division of Soil and Rock Mechanics at the Royal Institute of Technology and in particular Lena Wennerlund and Associate Professor Stefan Larsson who has become my mentor in soil mechanics, probably without knowing it.

On a personal plane I would like to thank my dear friends Jindar Türkan Sweco and Johan Szabo for showing me a life outside the world of studies. My family who have supported me without exception and to whom I dedicate this thesis.

Finally, since music is a big part of my life, my warmest thanks to my fellow friends in the rock band Dr. Livingdead. We've begun a journey that might very well end up at the very top. Without you, my life would be less interesting.

Notations and Symbols

Roman Letters

A	Area [m^2]
c	Cohesion [kPa]
d_{50}	Diameter at which 50 percent of the core soil is finer [mm]
D_{50}	Diameter at which 50 percent of the filter soil is finer [mm]
D	Diameter [mm]
e	Void ratio [-]
E	Youngs Modulus [MPa]
F	Load [N]
g	Acceleration of gravity [m/s^2]
H	Height; Hydrostatic head [mm]
k	Permeability coefficient [m/s]
k_v	Vertical permeability [m/s]
k_h	Horizontal permeability [m/s]
K_a	Coefficient of active pressure [-]
K_p	Coefficient of passive pressure [-]
l	Length [mm]
n	Porosity [%]
P_a	Resultant active pressure [N]
P_p	Resultant passive pressure [N]
q	Rate of water [m^3]
Q	Volume of water during time [m^3/s]
r	Radius [mm]
t	Time [s]
u	Pore water pressure [kPa]
V_a	Volume of air [m^3]
V_s	Volume of solids [m^3]
V_v	Volume of voids [m^3]
V_w	Volume of water [m^3]
V_{tot}	Total volume [m^3]
W_s	Weight of solids [g]
W_w	Weight of water [g]
W_{tot}	Total weight [g]
w	Water content [%]
w/c	Water to cement ratio [-]

Greek letters

β	Factor of adhesion [-]
Δ	Difference [-]
γ	Unit weight [kg/m^3]

ρ	Mass density [kg/m^3]
σ_v	Vertical stress [kPa]
σ'_v	Effective vertical stress [kPa]
τ_{fu}	Shear stress at failure, undrained [kPa]
τ_u	Shear stress undrained [kPa]
ϕ	Friction angle [°]

1 Introduction

1.1 General

The society of today relies on a constant supply of electricity. With a growing awareness about the negative effects of fossil fuels on our environment, more attention is given to renewable energy sources. Among renewable energy sources is hydropower. Hydropower provides a simple way of extracting energy from water and does so by using the force of gravitation. Globally, hydropower has a very small impact on the environment but locally it affects the environment due to its need to store water in a reservoir. It has an effect on animals such as fish but in terms of emissions of carbon dioxide it is considered clean.

To extract energy from water a dam construction is needed. There are primarily two types of dams; concrete dams and embankment dams. Both have subclasses and the subclasses for embankment dams will be described further in *chapter 2*. The design depends on several factors such as the topography on the site, climate and closeness to building material. In Sweden, about 90% of the hydropower dams are embankment dams.

An embankment dam is a complex geotechnical structure. It is constituted out of several zones. Each zone has a specific task and depend on its neighbouring zones to stay in place. A large embankment can be over one hundred meters high and kilometres in length, built over a time span of several years. During its time of usage it may be subjected to various kinds of stresses. These stresses are generated from external forces such as temperatures ranging from cold winters to warm summers, different water levels in the reservoir, ice loads, heavy rainfalls etc. All these external forces will act negatively on the expected life span of the dam, speeding up the aging process.

Besides forces acting on the dam, there are also forces acting within the dam. These forces are often the result of an exterior action such as settlements in the underground or elevation of the water level in the reservoir. Settlements may cause a cracking of the central core of the dam and an elevation of the water level may raise the pore water pressures within the dam. An elevation of the reservoir may also cause a larger seepage flow through the dam, which is undesired since it may initiate internal erosion within the dam.

There are several factors that may cause a dam breach. A dam breach is when the dam failures. Failures of dams are rare but do happen and depend not only of the aggressiveness of the external or internal forces. It is also depending on the type of embankment. Often, a combination of factors leads to a breach.

A survey made from acquired data from real dam breaches in America 1900 - 1970 gives the following table, *table 1-1*:

Factors resulting in dam breach	
Factor	**Percentage of factor [%]**
Seepage through dam	30
Seepage through the under ground	14
Overflow	23
External erosion	18
Instability	6
Remainders	9

Table 1-1. Factors resulting in dam breach for embankment dams in America. Survey based on 77 actual breaches (risknet.foi.se 2002)

Table 1-1 shows the influence of internal erosion as a trigger mechanism for dam breaches. Internal erosion in the underground and through the dam alone is behind almost half of the recorded dam breaches (30% + 14% = 44%)

Another study focuses on the probability of a dam breach given a certain causing breach factor. *Table 1-2* shows the result made of an inventory made by ICOLD based on 14.700 dams, taller than 15m throughout the world excluding China. The total amount of breached embankment dams was 0,83%.

Causing breach factor	**Percentage of factor [%]**	**Probability during life span (30 years)**
Overflow and breach of sluice boards	48	$1/250$
Slope instability	8	$1/1500$
Internal erosion in the dam	28	$1/425$
Internal erosion in the under ground	12	$1/1000$
Remainders	4	$1/3000$
Total	100	$1/120$
(Internal erosion in general)	(40)	$(1/300)$

Table 1-2. Probability of dam breach factor throughout the expected life span of an embankment dam (risknet.foi.se 2002)

Table 1-1 and *table 1-2* focus on breaches but the main focus should be to prevent the breach factor from evolving.

Potential breach mechanisms, following internal erosion are (Fell et al 2005):

- Gross enlargement of pipe hole;
- Unravelling of the toe;
- Crest settlement or sinkhole on the crest leading to overtopping;
- Instability of the downstream slope.

In order to do prevent breach mechanisms to evolve, changes of the dam must be intervened and stopped. A change in the dam refers to a change from the normal state. *Table 1-3* lists actual changes observed in Swedish dams. The base for *table 1-3* is an enquiry by VASO sent to different dam owners. 68 dams with a height of more than 15m and 16 dams lower than 15m had changes to report.

Type	Name of change	Amount of changed dams
1	Seepage through the under ground	16
2	Seepage through the dam	19
3	Damages in the riprap	45
4	Abnormal movements (settlements or side movements)	16
5	Internal erosion in the under ground	5
6	Internal erosion in the dam	13
7	Slope sliding (up- and down-stream)	5
8	Changes in the interface concrete/dam	16
9	Changes in drainage system or filters	1
10	Changes in pore water pressures	4
11	Changes in grout curtains	1
12	Sinkholes	27
13	Remainders	3

Table 1-3. Different known changes recorded in Swedish dams. Some dams suffer from more than one change (risknet.foi.se 2002)

However, it should be considered that all changes are not in *table 1-3* since it is virtually impossible to detect all. Therefore, it should be considered more likely that a larger number of dams suffer from different changes. The largest number of changes can be found in the riprap but this can partially be explained by the method of discovering them. The riprap is highly visible to the naked eye while it is more difficult to detect changes inside the dam or in the underground.

The conclusion from the different tables is that changes in dams do occur. Dams suffers from accelerated aging and this, sometimes, lead to dam breach. Internal erosion is a common source for aging related changes, thus it needs to be examined further.

According to Dahlin et al (1987) there are a number of events that causes damages in dams:

- Poor compaction;
- Poorly constructed filters;
- Separation in core soil or filter;
- The core soil is sensitive to erosion;
- Frost heaves;
- Slope sliding;
- Vibrations from heavy traffic;
- Wash out of core soil through the underground.

An embankment that suffers from damages must always be considered at a risk.

Therefore, the main focus of the thesis is interventions that include injection grouting of the core in an embankment dam.

1.2 Objective

The objective of the thesis is to characterise important factors when using remedial methods on dams damaged by *internal erosion*. The performed studies assume that remedial actions in form of injection grouting will be done in order to mend the damages. Following factors will be examined:

- Injection method;
- Characteristics of the non-hardening grout;
- Studies of how the injected grout affects the original structure of the embankment dam.

Different injection methods will be examined and weighed against its possible use as a remedial method in an embankment dam.

A characterisation of the grout will be done in order to find key parameters of an injection material, of non-hardening characteristics, which will repair and remain within the dam over a longer period of time.

1.3 Hypothesis

The main hypothesis is:

> ***"Flexible grouts are better suited than hardening if used in an embankment dam"***

The term flexible grout relates to a grout constituted of solely soil particles and water. Since a dam core is sensitive to any form of intervention it should be considered that a grout with basically the same mechanical characteristics as the core soil is better suited. The main hypothesis is the basis for the tests described in *chapter 7*.

During laboratory studies in *chapter 5*, yet another hypothesis has been formed:

> ***"Designing the particle size distribution curve of a flexible grout will reduce the risk of internal erosion"***.

Since gap graded soils may erode a soil with a smooth particle size distribution curve should exhibit a greater buffer against internal erosion.

1.4 Limitations

Only damages caused by internal erosion in the core will be considered. Even though internal erosion occur in the under ground of a dam this will not be considered.

The actual grouting procedure will not be included. During each experiment, the tested grout has been put in place from the start of the test procedure.

1.5 Disposition

The disposition of this thesis is as follows:

Chapter 2 describes different types of embankment dams. The embankment dams are the environment, in which the grouting will be implemented (grouting of the core). The basic functions of each zone within an embankment and the filter criterions are also explained

Chapter 3 describes the internal erosion process within the embankment. Why internal erosion occurs and how. It gives a description of how damages look and also damages in special cases where the mechanisms of internal erosion are boosted.

Chapter 4 introduces different remedial methods and describes the basics of injection work. It also describes why injection work is desirable in order to repair. Parameters such as constitution, pressures, drilling and monitoring are presented here. Finally, an evaluation of the different methods will be done.

Chapter 5 will describe the decision making progress of choosing experiments and explain the parameters, targeted for testing and evaluation.

Chapter 6. This is where the performed permeability experiments are presented and concluded.

Chapter 7. This is where the results from the second laboratory, the filter box testing, is presented and concluded.

Chapter 8 concludes the thesis and point out possible future directions of research.

2 The Embankment Dam

This chapter will describe the embankment dam. The embankment dam is the environment where this research will be implemented. This chapter will describe the building material used in an embankment, different types of embankments and the different zones within the embankment.

2.1 Soil types

Since an embankment is constructed of soil it will be important to fully understand the aspects of the material. The geotechnical aspects will be of highest importance.

There are two types of soil; organic- and mineral soil.

- **Organic soil** – Made from decomposed organic material such as parts from plants. There are three sub types; peat, mud and muck;
- **Mineral soil** – Made from weathered and eroded rock parts. This soil is the most common in Sweden and can be found practically everywhere.

Since organic soil never is used in an embankment this will not be discussed any further.

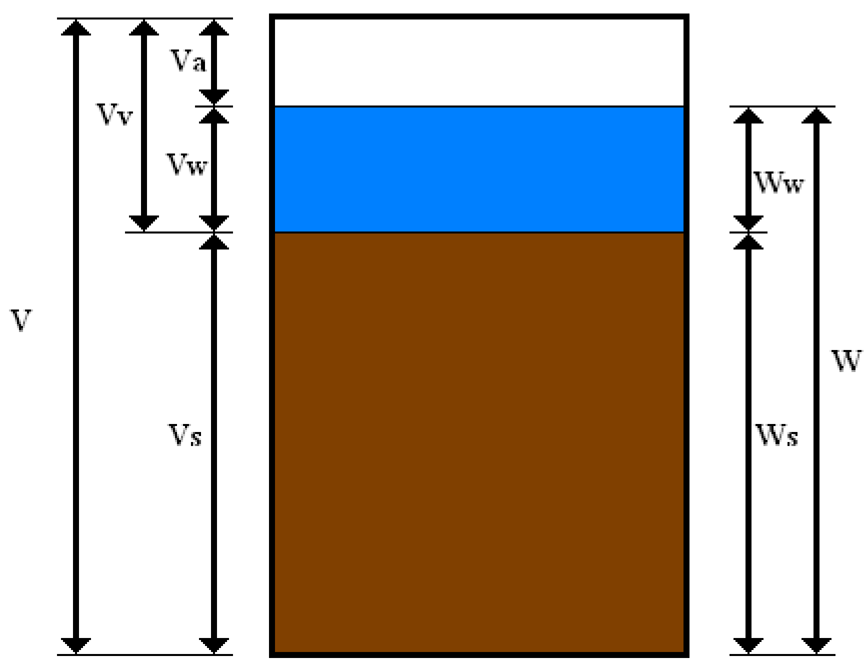


Figure 2-1. Composition of soil. Solids, water and air. Water and air together is also known as the voids.

2.1.1 Till

The most common soil type in Sweden is the till. Till originates from the ice age and is constituted of mineral soils. The soil contains a large variety of particle sizes, everything from clay to blocs. Only mineral soils are used in an earth fill embankment dam.

The characteristic of the till is mainly dependent of its origin. If the soil is constituted of weathered granite it will be rich in blocs, sand and silt. If the soil has a high content of clay it originates from weathered rock types such as slate. There is also a simple rule of thumb to predict what type of till to expect at a building site: The higher the altitude, the less content of blocs. This is due to the fact that gravity has a larger influence on blocs. Blocs are more often found in valleys and hollows. In other words the till on higher altitudes has a lower content of blocs while the till at lower altitudes has a higher content of blocs (Alexandersson 2004).

The geotechnical aspects of the till are:

- High capillarity;
- Low permeability;
- Stiff with a τ_{fu} of about 100kPa. Till with a high content of clay can have a τ_{fu} between 200-300kPa but this is seldom the case;
- Inconsiderably frost-sensitive to very frost-sensitive depending on the percentage of silt;
- The angle of internal friction is high.

The **high capillarity** could cause problems when the soil is subjected to freezing. Due to this fact it is not recommendable to construct a core during the winter season. If this would be done the till would be exposed to freezing and thawing thus making the core less compact.

The **permeability** of the till is very low which makes it suitable to act as a water barrier.

The **high undrained shear strength** could cause a problem because the soil becomes less flexible to work with. In the prolonging this can make it easier for cracks to develop within the core.

The **high angle of friction** enhances the safety factor in order to avoid surfaces of rupture.

The **availability** of the till makes it very suitable as a building material from an economical point of view.

The characteristics mentioned above makes the till suitable as a building material of the core in an embankment.

Besides the previous aspects, the till is commonly found. In average there is 7m of till evenly distributed over an area equivalent to the area of Sweden.

2.1.2 Fraction sizes

Mineral soils are often divided into two main groups, fine grained (degree of fines > 50%) and coarse grained (degree of fines < 50%). Stones and boulders are excluded from these groups. The term fraction sizes refer to the specific size of each grain of soil obtains when weathered or eroded. Slates provide soil with smaller particles while clastic stone types such as conglomerate offers coarser soil types. The measurements of particle sizes can be found in *figure 2-2*.

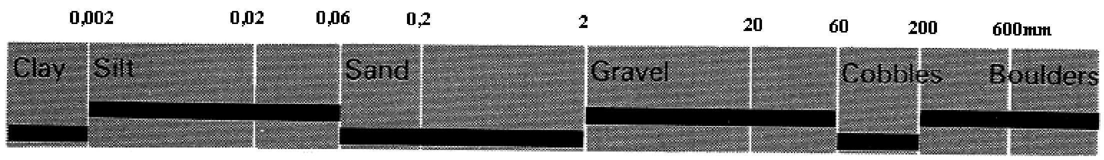


Figure 2-2. Different fraction sizes with their names and sizes (Vattenfall 1988).

It is important to distinct the different sizes of fractions from one another since the distribution play an important part in building up a stable soil structure. In this thesis the term stable refers to its possibility to withstand seepage without the fines being washed out. This will be discussed further later in this chapter.

2.2 Different types of embankment dams

There are two main types of embankment dams:

- Earth fill embankment dam;
- Stone fill embankment dam.

As the names suggest, an earth fill dam is manly built with soil material and the stone fill dam is mainly built with stone material. The dams are divided into multiple zones where each zone has specific characteristics and a specific task to fulfil.

In an *earth fill dam* the main zones are:

- **The core** - Constructed of till and has a very low permeability between 10^{-6} to 10^{-9} m/s. The core lowers the pore pressures in the dam and limits the seepage;
- **The filter zones** - Constructed from relatively fine non-cohesive material and are supposed to protect the core and keep it in place. It is also acts as a draining system of the construction;

- **The stabilizing zones** – Rocks from blasting that stabilizes and keep the entire construction in place;
- **The protective zones** against erosion – The building material is large (>600mm) and protects the dam against various types of erosion such as waves, heavy rainfalls, ice, inundations etc.

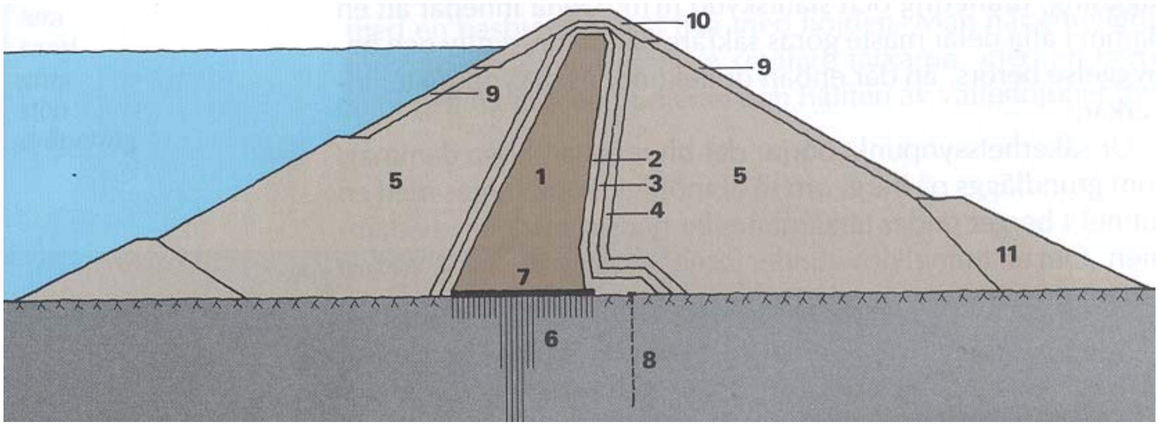


Figure 2-3. A large earth fill dam (>15m) with a central core of till at display (Vattenfall 1988).

In *figure 2-3* each number describes a zone. The zones are:

1. The core;
2. Fine filter;
3. Medium filter;
4. Coarse filter;
5. Shell;
6. Grout curtain;
7. Concrete plate;
8. Filter-well;
9. Riprap;
10. Crest protection;
11. Dam toe support.

In a *stone fill dam* the zones are different. In this case a stone fill dam with a face slab will be at display (see *figure 2-4*). A stone fill dam with a face slab can be constructed in areas where there is a shortage of till. It is also a wise choice in arctic zones since there is no water in the stone fill that can cause freezing and thawing. The zones are primarily:

- **The face slab** – Usually constructed of concrete or bitumen. This zone acts as a water barrier
- **Stone fill** – Large boulders that constitutes the main part of the dam

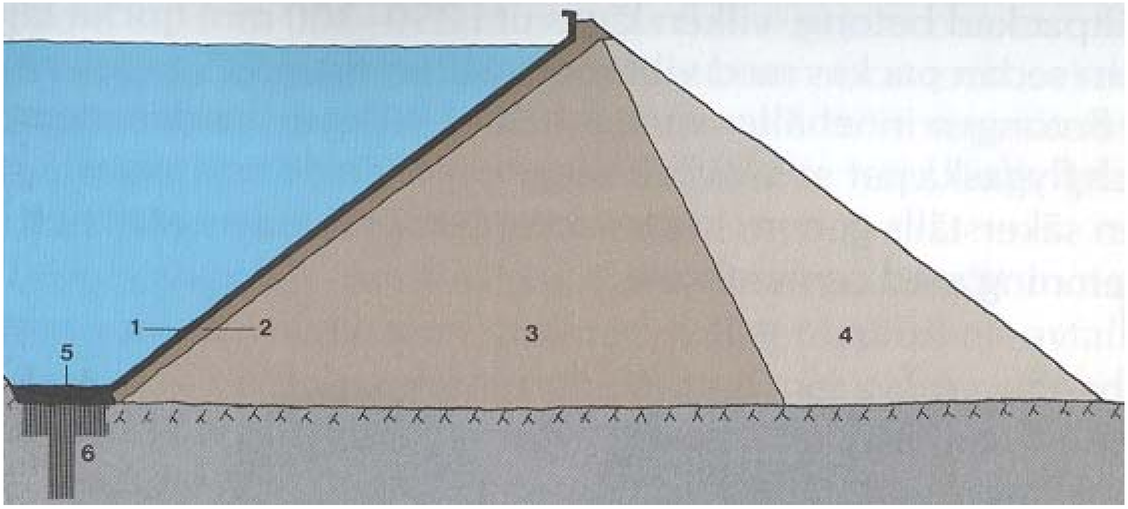


Figure 2-4. A typical stone fill dam with a face slab (Vattenfall 1988).

In *figure 2-4* each number describes a cone. The zones are

1. Face slab, concrete face;
2. Transition zone;
3. Stone fill, 1,0m/layer;
4. Stone fill, 1,5m/layer;
5. Seepage seal;
6. Grout curtain.

If the two dam types are compared in a water retaining point of view, the water retention in the earth fill dam take place in the core while the same action take place at the face slab of the stone fill dam.

We will not study the latter construction any more but only concentrate on the earth fill dam.

2.3 The core

As mentioned before the material used in the core is well sorted till. The main characteristics can be found in *chapter 2.1.1*. The design of the core depends not only on factors such as the availability of the till and its characteristic but also on geometrical aspects. A wide core is supposed to be safer than a thin when considering formation of cracks. A wide central core is desirable in a high and narrow valley since the high water table will give raise to excessive pore water pressures. If the till has a very low permeability in combination with a high water ratio there can be problems in handling the soil. The soil is almost similar to concrete and cracks more easily. In this case it is preferred to construct a thin core since it is easier to build this correctly (Vattenfall 1988). A leaning core (4:1, 2:1) offers better-distributed pore water pressures on the downstream side. It also diminishes the danger of slips and suspension against the filter (Vattenfall 1988). Suspensions of the core is dangerous since it lowers the effective vertical stress,

σ'_v, on the soil since the soil looses its bearing capacity due to the relative augmentation of pore water pressure, u.

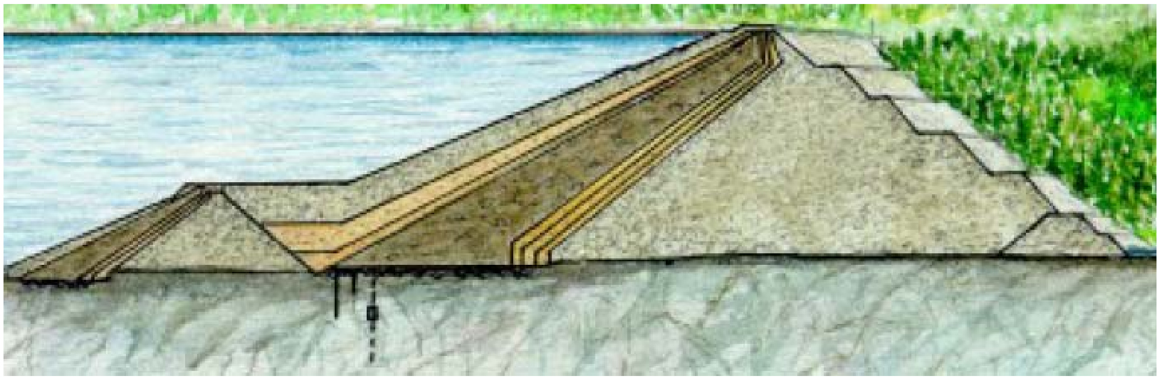


Figure 2-5. A leaning core in an embankment dam.

2.3.1 Desirable properties of the core

The till in the core is optimal if it has the following properties (Vattenfall 1988):

- Silty, sandy till with a content of material < 20mm between 15-40%;
- A permeability between $3x10^{-7}$ to $3x10^{-9}$ m/s ;
- A water ratio about the same as the optimal obtained via laboratory tamping;
- Good compactibility according to test packing;
- Reasonable low stone ratio;
- Small share of blocs.

In addition to this it is desirable to have the till extraction point relatively close to the construction site.

If the till isn't found close to the dam, other solutions of the design should be made.

2.4 The filter

The main objective of the filter is to keep the core in place. This refers to prevent the fines from the core to be washed out with the seepage. The filter must also allow the seepage to be drained (Fell et Al 2005). In larger embankment dams there are up to three different filters but normally only two: fine filter and coarse filter. The fine filter is made from sandy material and the coarse filter is made of material with about the same size as macadam. If there are three filters, this third filter is placed between the fine- and the coarse filter. The size of the grains in this filter is normally between 2-60mm. A third filter can be needed in dams where the core has a high content of fine material such as clay.

The final zone between the filter and the shell is a transition zone. It is important that the filter has low cohesion in order to prevent the forming of cracks. In large dams there is a rule of thumb to construct each filter with a width of at least five meters because wide filters offers better protection against cracks in the core (Vattenfall 1988).

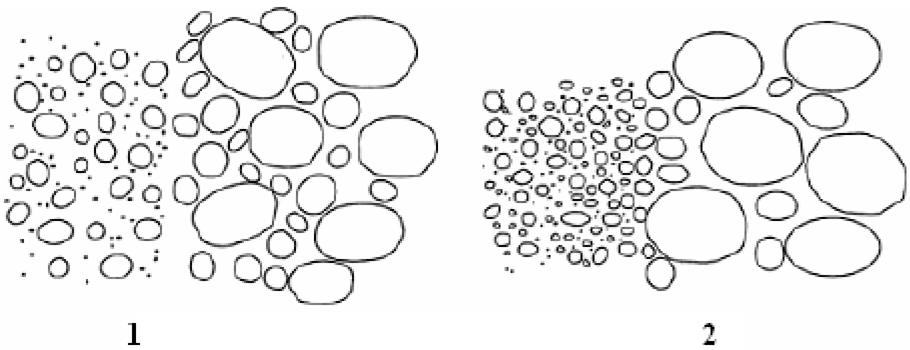


Figure 2-6. This figure above shows the principle function of a filter material. On the left side, marked 1, a poorly graded base soil with large differences between the fines and the coarse material. On the right side, marked 2, a well functioning filter with smaller differences between the fines and the coarse material (Vattenfall 1988).

A filter has mainly three demands in order to work satisfying:

- The permeability must be higher in relation to the core to make drainage possible;
- The porosity must be low to prevent fines from the core to be transported through the filter;
- The differences in distribution of particle sizes should be minimized to prevent separation of the filter material during time of construction.

During normal conditions the downstream filter work both as a protection against wash out of finer particles and as a draining system. The upstream filter do not have the same demands regarding wash out of fines but must be designed to manage a sudden sinking of the reservoir. In this case the seepage through the dam is at risk of being inverted, thus creating negative pore water pressures. If the core is washed out of its fines the upstream filter must be able to mend the damage and does so by "sacrificing" its own material into the damaged part of the core.

Furthermore the upstream filter must:

- Offer protection against wave erosion;
- Offer protection against thawing and hydraulic fracturing of the core;
- Offer a transition zone in order to protect the outer surface of the core that is extremely sensitive to erosion (see *figure 2-7*);
- Function in the same way as the down stream filter in case of a sudden sinking of the reservoir.

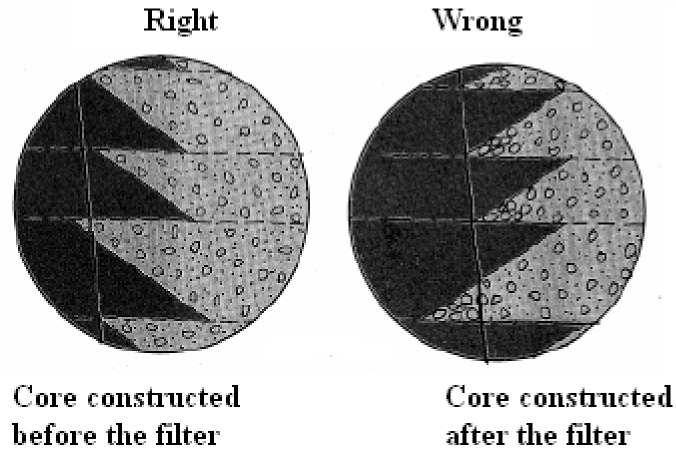


Figure 2-7. The figure shows the border between the filter and the core. The core is shown in dark colour while the filter is shown with a pale colour. On the left the core is constructed before the filter. On the left the filter is constructed first. If the filter is constructed before the filter there is a risk of separation that greatly enhances the risk of erosion at this point (Vattenfall 1988).

2.4.1 Criterions of the filter

The criterions of the filter are very important when designing an embankment dam. There are clear connections between poorly constructed filters and dams damaged from internal erosion (Rönnqvist 2006).

Filters are normally graded after their distribution of particle sizes. They must be fine graded to keep the fine material in the core in place. Meanwhile, they must also be coarse to act as drainage systems. This is the basis to the idea of having different zones in a dam. Roughly, from the core and out finer particles are gradually substituted by coarser material. A coarser zone should always protect a more fine graded zone inside itself from being washed out.

To meet these demands there are certain demands on the “ideal filter” (Nilsson et Al 1999) and (ICOLD 1994):

- It must not separate at the time of construction;
- It must not change its particle size distribution curve during anytime of construction or usage;
- It must not be cohesive or consolidate in order to prevent cracks;
- It must be internally stable (not gap graded);
- It must have a suitable permeability to allow drainage;
- It must be able to seal an eventual leak in the core.

In order to construct these filters in a correct manner there are criterions, which are based on the distribution of particle sizes of the filter material. The definitions will be discussed further in the next segment.

2.4.2 Filter rules

Karl Terzaghi developed the first rules in the 1940s. The rules at this time where lacking clear definitions such as the maximal allowed size of particles in the filter. Because of this the rules have been revised over the years. A further problem of not having clear rules is the possibility of each designer to make own interpretations when deciding the design of the filter. With the release of RIDAS 1998 these rules have been specified even more and today, filters tend to be finer graded than their predecessors. Today there are also specific rules for the allowed maximum particle size in order to prevent separation (see *table 2-1*).

It has been evident that the material used in cores (till) mainly is constituted of silt and sand fractions where stones (>60mm) are embedded. Not until the 1970s it became clear that the filters played an important part in order to prevent erosion of the core. Until the seventies the filter was only thought to play a second role of the protection of the core. The core alone was thought to mend itself in case of damage and the definitions of the filters were sometimes changed to suite the available building material near the site.

It must be mentioned that erosion of the core cannot only be blamed on poorly constructed filters but is mostly caused by a combination of multiple events (Nilsson 1995).

2.4.2.1 Definitions of filter rules

Definitions

d = Particle size of the core material (core material according to sieve testing < 20mm).
D = Particle size of the filter material (according to sieve testing. Fine filter <60mm and coarse filter <100mm).

Degree of fines = amount of material <0,06mm, based on a sample <20mm

Core material with a degree of fines < 30%	**degree of fines 30-80%**
$4 < D_{15}/d_{15} < 40^*$	$4 < D_{15}/d_{15}$
$4^{**} > D_{15}/d_{85}$	D_{15} <0,7mm
$25 > D_{50}/d_{50}$	$25 < D_{50}/d_{50}$
D_{max} <60mm	D_{max} <60mm

* Decisive factor for a widely graded material.
** Decisive factor for a narrowly graded material.

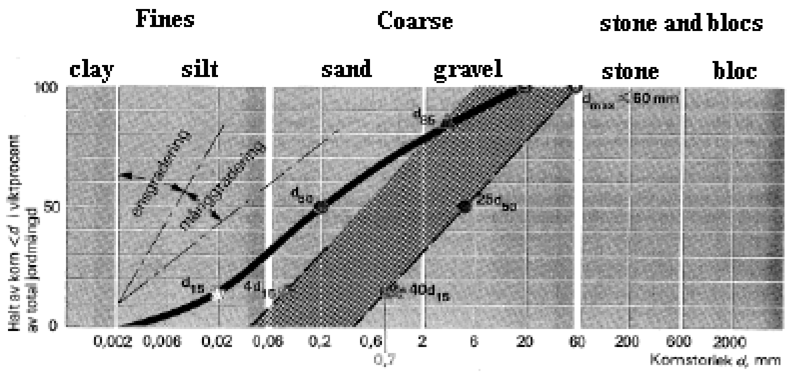


Figure 2-8. The curve represents a given base soil from the core and within the darker zone are the borders of a suitable filter. This figure meets the demands of a widely graded base soil (Vattenfall 1988).

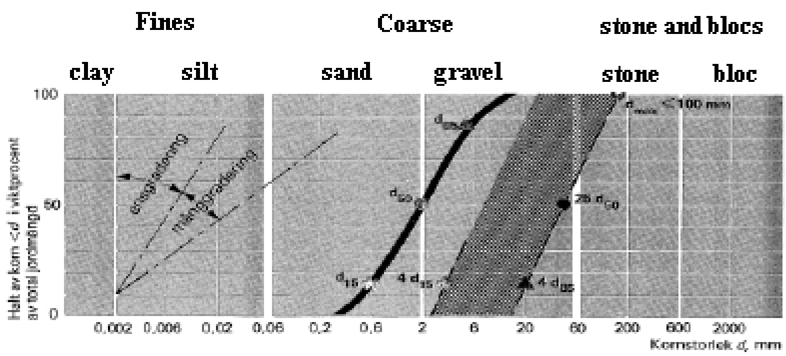


Figure 2-9. The curve represents a given base soil from the core and inside the darker zone are the borders of a suitable filter. This figure meet the demands of a single graded material (Vattenfall 1988).

In order to fully understand the factor D_{15} it is best described as the largest particle size (mm) that constitutes 15% of the total amount of soil in the filter (<60mm or <100mm). The same principle is adapted on the base soil.

Min D_{10} (mm)	**Max D_{90} (mm)**
<0,5	20
0,5-1,0	25
1,0-2,0	30
2,0-5,0	40
5,0-10,0	50
10,0-50,0	60

Table 2-1. Maximum allowance of stones in a fine filter in order to avoid separation (RIDAS 1998)

2.4.3 Gap grading

Sometimes the soil is gap-graded. This is also known as suffusion. If this is the case there is a greater risk of the soil being exposed to erosion. The term gap grading originates from the shape of the particle size distribution curve obtained via sieve testing (see *figure 2-10*). Since a specific particle size is missing, the interaction between all the different sizes is at risk of being avoided. It also induces a higher risk of separation.

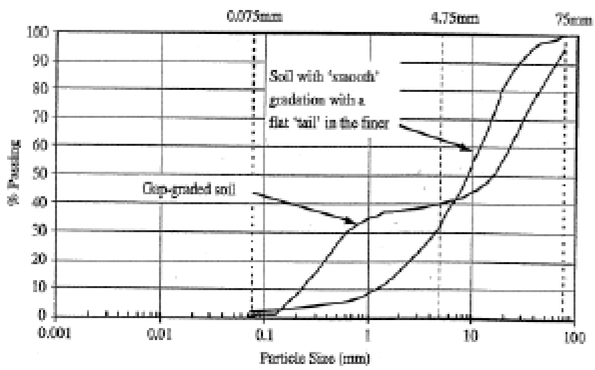


Figure 2-10. Gap-graded soil where particles ranging from 1 to 10 mm are missing (Fell et Al 2005).

2.5 Shell

The shell can be said to be the skeleton of the dam. It constitutes the last line of defense in order to prevent a rupture of the structure. In stone fill dams the material is normally large blocs and boulders, preferably stones from blasting since the sharp angels offers a higher angle of friction thus greater geotechnical safety. A disadvantage with blocs from blasting is its sharp edges. These edges are easily crushed during construction and deposited between the blocs. Through time the crushed material will be washed away and in the prolonging this can cause severe settlements of the structure. However, this can be avoided by watering the blocs during the time of construction (Vattenfall 1988). Sometimes the material can also be coarse non-cohesive material. In this case the design of the riprap protection becomes important.

In case of leakage it is up to the shell to keep the dam intact until remedial work has been performed (Vattenfall 1988). The shell is often over designed to attain a higher safety factor.

2.6 Riprap protection

The riprap protection protects the dam from various types of erosion such as wave erosion and ice. Slate stones should be avoided since they are easily eroded. The most widely used materials are blocs and natural stone but in some cases concrete blocs might be used.

The **upstream side** must withstand erosive forces from waves and ice. The waves in particular might wash out the shell. The blocs must be designed so that the waves may not move them. Filter criteria must be attained between the shell and riprap.

The **downstream side** must withstand erosive forces from rain, snow melting, frost and ice. The toe of the dam is often heavily fortified because this part of the dam is most often the origin of heavy seepage, if this occurs. A fortification of this part enhances the resistance towards rupture. The size of this material can sometimes be up to a meter in diameter.

3 Internal Erosion

As briefly mentioned earlier in the thesis, all dams are subjected to seepage and in certain cases this seepage can cause internal erosion of the core. The internal erosion of a core is a process that people working with embankment dams are familiar with, but no one knows exactly what triggers it. This chapter will explain the mechanisms of the process, the formation of cracks, damages at the interface of neighbouring concrete structures/core soil, damages near and in the vicinity of surveillance equipment and damages caused by heavy traffic.

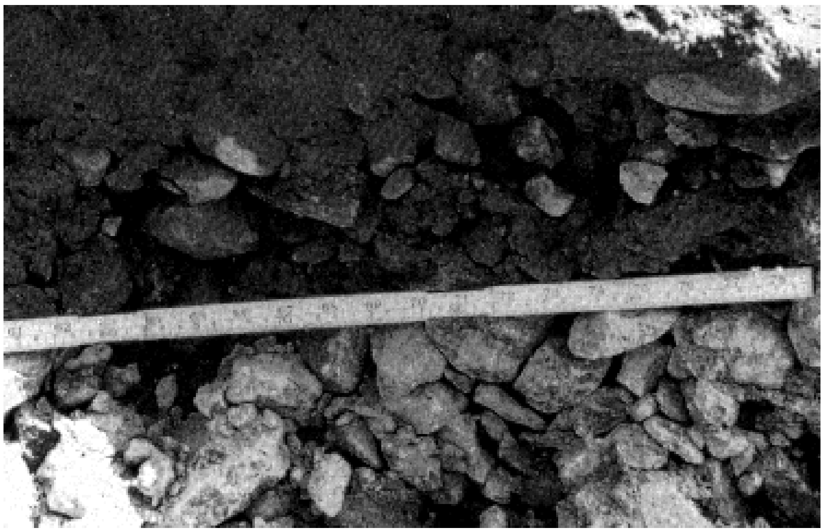


Figure 3-1. This is a photo showing internal erosion. Above the folding rule the finer material is still present but below it, no fines are to be found (Vattenfall 1988)

Internal erosion is an event where the finer particles from the core escape the dam construction. If the erosion isn't discovered in time the core will gradually loose its primary function, to act as a water-retaining barrier. As the seepage-flow accelerates, the forces acting on the fines provided by the seepage water becomes higher. At a certain point these forces become stronger than the fines ability to withstand them, thus internal erosion initiates.

For internal erosion to occur, three conditions must exist (Fell et al 2005):

- There must be a seepage flow path and a source of water;
- There must be erodible material within the flow path and this material must be carried by the seepage flow;
- There must be an unprotected exit (open, unfiltered), from which the eroded material may escape.

In addition, to form a pipe the following condition must exist:

- For a pipe to form, the material being piped, or the material directly above, must be able to form and support a “roof” for the pipe.

The conclusion of the conditions is that internal erosion only occurs if the core and filter simultaneously flaws. This can be shown in a fault tree, *figure 3-2*:

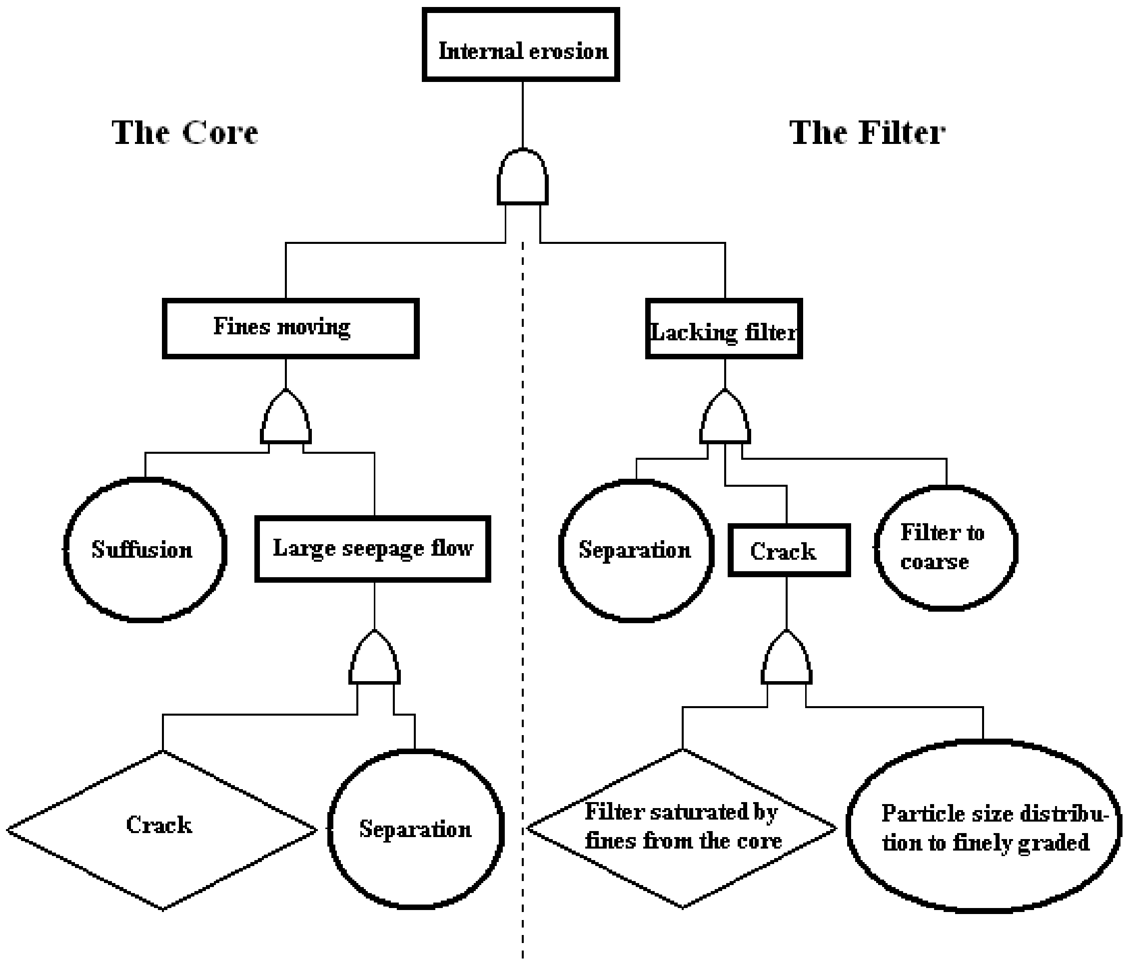


Figure 3-2. Over view fault tree of internal erosion. The actions that might take place in the core to the left and vice versa for the filter on the right side.

In *figure 3-2* it is possible to see what causes the internal erosion. A single event cannot start the internal erosion process alone, thus key events must occur. By doing a top-down approach these key events are more easily found.

The most common type of internal erosion in the core is the forming of a pipe (Fell et al 2005). If the backward erosion is allowed to act over a longer time sinkholes might occur on the crest of the dam either directly over the core or the up-stream filter.

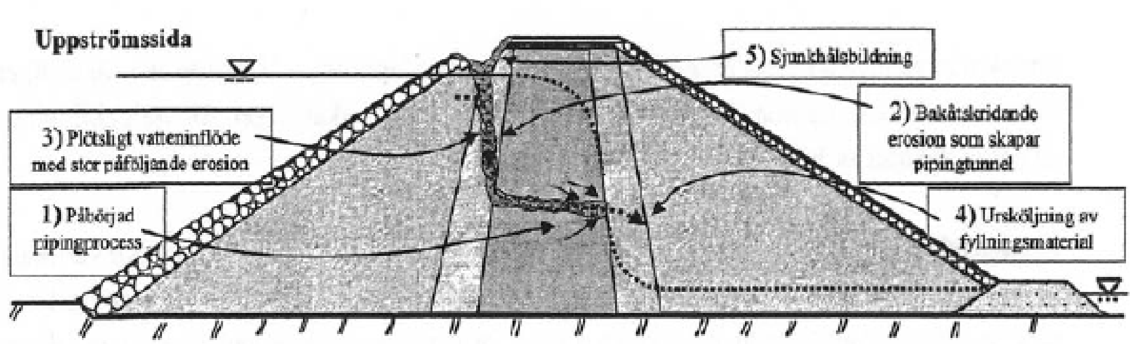


Figure 3-3. The progression of backward erosion can be seen on the figure above. 1- initiation, 2 – Backward erosion, 3 –Sudden inflow of water, 4 –Fines escaping, 5 – Sinkhole (Hans Rönnqvist 2002).

Internal erosion is a concept when the soil is exposed to a larger external shear force than it can withstand. In this case the external shear force is provided by the seepage. If the internal erosion is allowed to progress the core will be damaged. Since material is escaping the core, a pipe, smaller conduits or a high permeable stratum might form. Through these formations the fines may escape the core. The damages can roughly be divided into three types:

1. A pipe;
2. Smaller conduits;
3. Diffuse stratums.

The cause for internal erosion can originate from three periods. During each phase human error or natural causes act on the dams final resistance to internal erosion. The phases are:

- Design phase;
- Building phase;
- Usage phase.

Design phase – During this stage the material used in the dam is evaluated. Problems may occur in the testing process, faulty conclusions may be drawn based on tests on core soil and the under ground. If this is done the dam will be built and designed based upon wrong facts.

Building phase – If the construction is large there might be problems with the uniformity of the different zones (compaction, water ratios, separation). The magnitude of the dam will extend the construction time up to several years. However, building during winter is not recommended, thus the dam will be exposed to thawing. This may create high permeable layers horizontally through the dam.

Operation phase – Different settlements may occur in the dam causing cracks, pore pressures may vary and filters may be saturated of fines from the core and cause cracking of the filter.

3.1 Continuation of internal erosion

The continuation of internal erosion depends highly on the zoning of the dam e.g. filters. Homogenous embankment dams without filters are more likely to allow a continuation of internal erosion. All core soil posses an ability of self-healing and they may (Fell et al 2005)

- **Seal with no erosion** – rapid sealing of the concentrated leak with no potential for damage and no or only minor increases in leakage, continuation will be very unlikely;
- **Seal with some erosion** – sealing of the concentrated leak but with the potential for some damage and minor or moderate increases in leakage, continuation will be likely;
- **Excessive or continuing erosion** – slow or no sealing of the concentrated leak with the potential for large or continuing erosion losses and large increases in leakage, continuation will be highly likely.

Even if the filter design is well done there might be “blind spots”. Blind spots refer to points in a dam where the filter is unable to protect the core. This may bring the dam into a dangerous situation during flooding. If the filter has been design to protect the core, only to a designed reservoir level, the filter may be overtopped. If this happens, pipes might form in the upper part of the dam and in combination with cracks, a breach situation might rapidly form.

3.2 Piping

Piping is a process where the fines are washed out from the core. Meantime the cores cohesion is sufficiently high to prevent the forming pipe to collapse. The core soil may also exhibit arching capability (see *chapter 3.4.1*). The word piping refers to the form of the damage, which can be seen as a tunnel inside the core soil with various dimensions.

If piping continues over a long time the dam might breach.

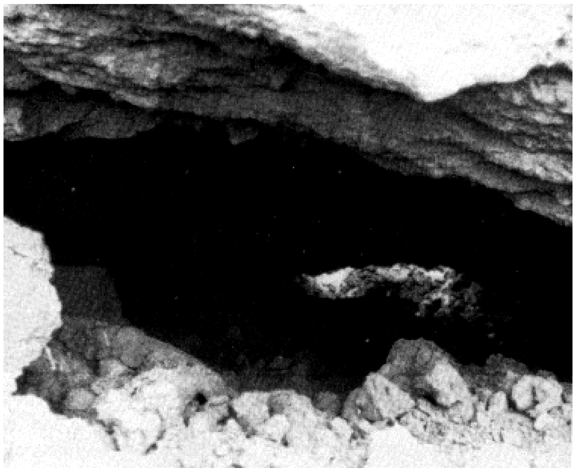


Figure 3-4. Photo of a piping damage (Vattenfall 1988)

3.2.1 Piping in an embankment

Piping in the core may be initiated by any of the following processes (Fell et Al 2005):

- Backward erosion;
- Concentrated leak;
- Suffusion.

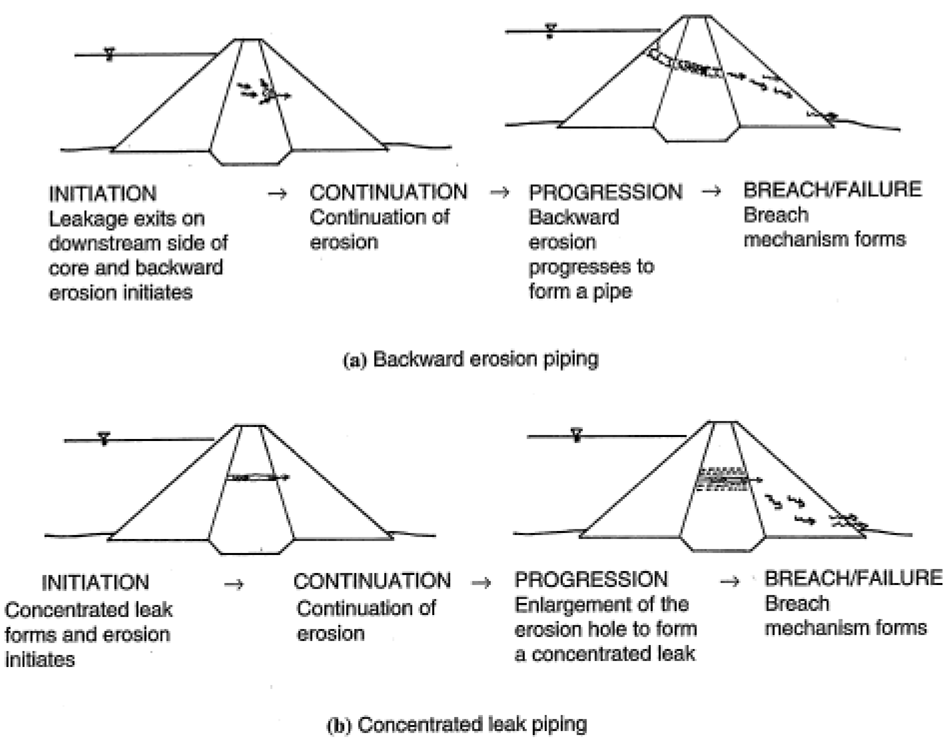


Figure 3-5. Different piping, backward erosion piping and concentrated leak piping (Foster et al 2005).

Backward erosion piping starts at the interface between the core and the filter or between the fine- and coarse filter. The erosion propagates backwards, towards the upstream side of the dam into the core. Fines from the back of the forming tunnel are constantly suspended in the seepage, thus creating a backward enlargement of the tunnel. As the formed damage grows in width and height, the erosion is accelerated.

Concentrated leak piping happens when a crack is forming in through the core between the up-, and down-stream side. It may also happen if a layer of high permeable material cut the core. Cracks can form if the core soil is very stiff and the dam is subjected to a movement. Cracks can also form due to differential settlements and the fact that the stiffness of the core may vary from point to point. Cracks may not only form in cores but can also exist in very fine graded filters. Fine graded filters can be caused by a constant rate of fines from the core being captured by the filter.

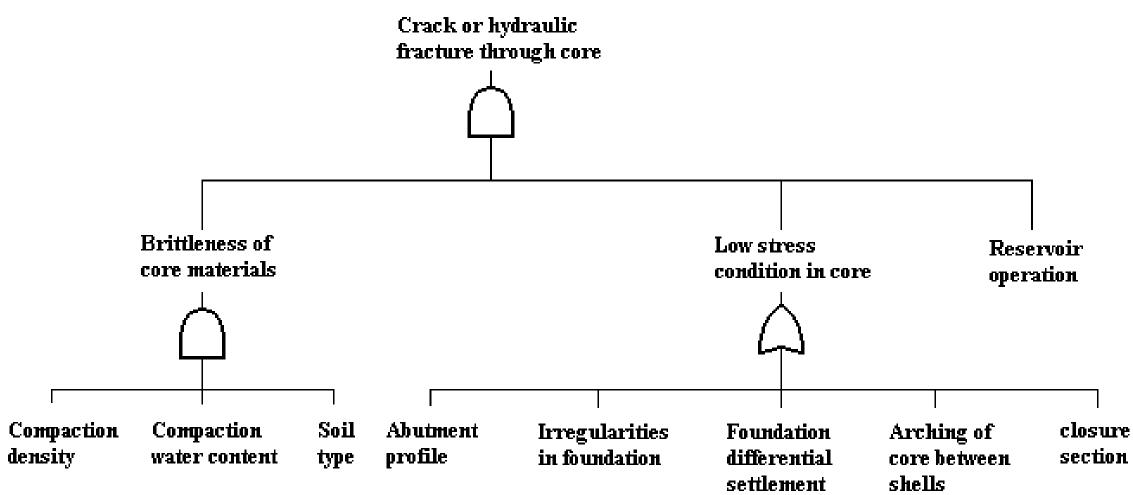


Figure 3-6. Fault tree diagram for the formation of a crack or hydraulic fracture through the core (Foster 1999).

Suffusion is the washing out of fines from internally unstable soils, which is described in *chapter 2.4.3.*

Potential breach mechanisms due to piping are according to Fell et al (2005):

- Gross enlargement of the pipe hole;
- Unravelling of the toe;
- Crest settlement or sinkhole on the top on the crest leading to overtopping;
- Instability of the downstream slope.

Pipes may evolve through the entire core, spanning from downstream to upstream. In some cases piping progress but collapses. If this is the case, core soil above the pipe will be damaged. In some cases subsidence of soil propagates up to the crest. If the pipe reaches through the entire core without collapsing in combination with reaching the upstream filter, the filter material may enter and seal the pipe.

Figure 3-7. Pipe found in core soil during dismantling of the Lövö dam, Sweden (Ericsson et al 1998).

3.3 Sinkholes

In embankment dams, sinkholes are a severe warning that the dam has been exposed to excessive internal erosion. Depending on the location of the sinkhole it is possible to tell what kind of internal erosion is occurring inside the dam. There are mainly two positions on a dam where sinkholes might be found:

- On the crest, over the filter or;
- On the crest, over the core.

Sinkholes may also occur in the up-stream or down-stream slopes.

If the sinkhole is found on the crest over the filter, a pipe has formed within the core and the core has remained intact except from the piping damage. If the sinkhole is found on the crest over the core, piping has not occurred but the core is damaged from the point of internal erosion up to the crest.

Figure 3-8. A sinkhole on the crest of Bennet dam, B.C Canada (Garner 2006)

The process behind the forming of sinkholes is simple. During the internal erosion process material is constantly transported out of the dam. A loss of material occurs and is replaced by the overlaying soil, pulled downwards into the cavity due to gravitational forces.

3.4 Cracks

An embankment dam is constantly exposed to different loads. The water level in the reservoir constantly changes, the weather varies between warm summers and cold winters, the soil in the embankment can consolidate and the underground might move. All these actions create a movement of the dam. Since the stiffness of the core soil varies the movement will create different tensile forces on the core soil, which may crack the core. Cracks can also form close to concrete constructions and at the interface filter and core since the core soil tends to suspend itself towards its neighbouring zone, thus allowing tensile forces pulling the soil.

Cracks might also form due to earthquakes. However this is a minor risk in Sweden. It takes at least an earthquake with a magnitude of 6,5 on the Richter scale to create a crack in the core (Fell et al 2005).

According to (Ekström et al 2001) cracks form due to:

- Settlements in a dry compacted core during its first filling;
- Settlements in a low permeable, wet-compacted core. Through time the core drains, thus creating a settlement;
- Suspension towards neighbouring joints and constructions;
- Creation of arches between differentials on the surface of the underground.

Yet another explanation can be found in the construction technique. Since the core is built in several layers, all these horizontal layers form a horizontal weakening of the core soil.

The knowledge of cracks is in a way disregarded since the dam owner quickly mends this type of damage if visible, leaving hardly any time for research (Sherard et al 1965).

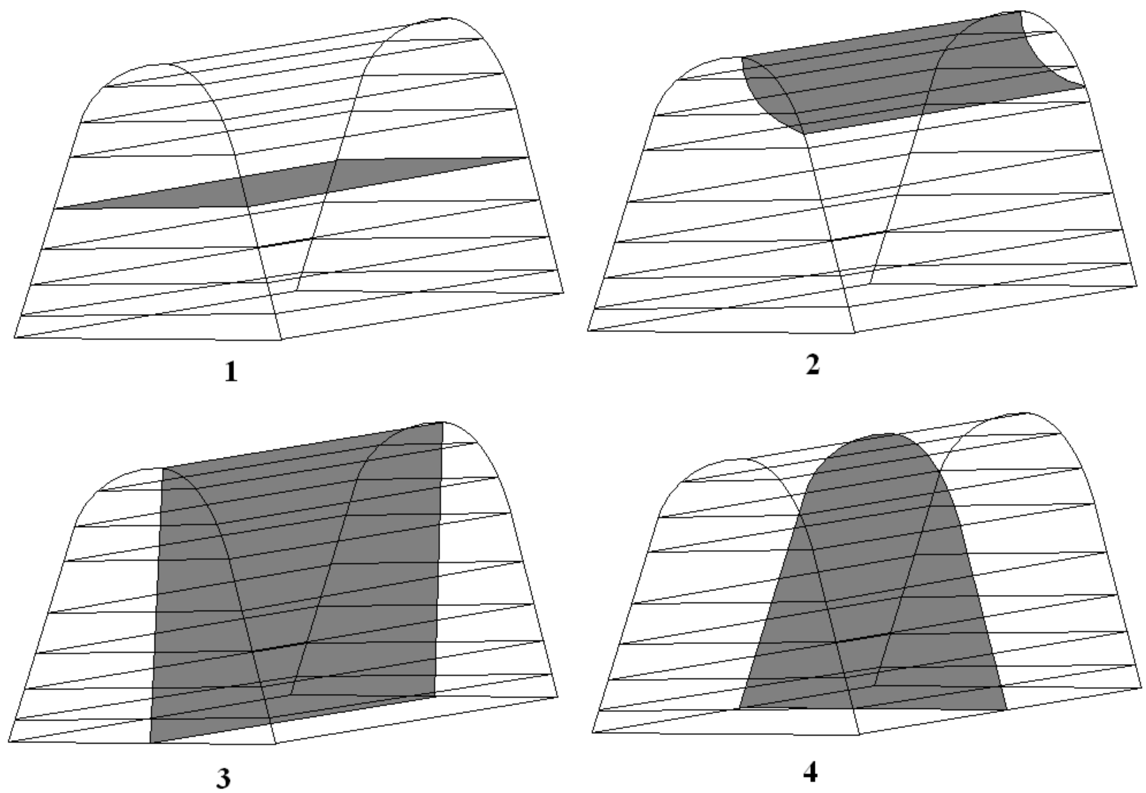


Figure 3-9. Principle orientation of cracks in an embankment dam.

The general orientation of cracks can be seen in *figure 3-9*. Different orientations expose the dam to various degrees of danger. Number one and number two offers a greater risk since it allows the seepage a fast passage through the core. Number three and four does not. Cracks may also be local and not traverse the entire core.

A fault tree for cracking can be seen in *figure 3-6.*

Heavy traffic may also enhance the risk of cracking. Embankment dams are often used as natural bridges, allowing traffic to cross the river. However this risk should be considered unlikely since the traffic-induced vibrations only affects the very upper part of the dam (Dahlin et al 1987). The traffic-induced vibrations are similar to earth quake loads. Vibrations can have a profound influence on the behavior of some non-cohesive soils that may be found in an embankment dam. The vibrations can cause an increased loss of fines (Kenney et al 1985).

3.4.1 Hydraulic fracturing

Hydraulic fraction can take place in the presence of cracks, arching or a high permeable connection between the reservoir and the inside of the core. The reason for this is the fact that it allows the pore water pressure to become greater than the soil pressure.

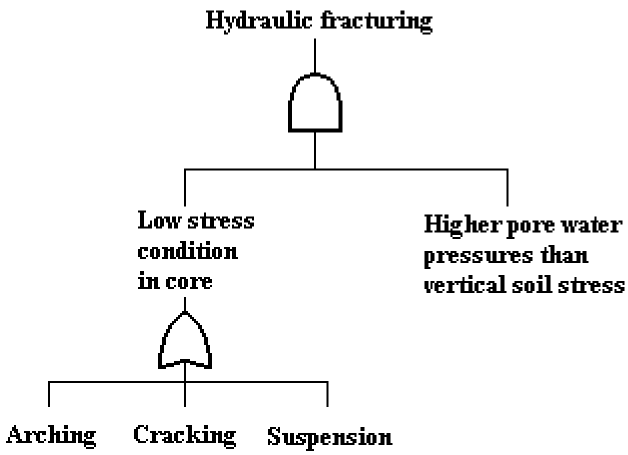


Figure 3-10. Fault tree, hydraulic fracturing.

The result is similar to an explosion when the soil particles are pushed apart, in a very slow rate. The result is that all interacting forces between the soil particles cease.

It can be described by the formula:

$$\sigma'_v = \sigma_v - u \qquad \textbf{(1)}$$

Where in **(1)**:

$$\begin{cases} \sigma'_v = effective\ vertical\ stress\ [kPa] \\ \sigma_v = vertcal\ stress\ [kPa] \\ u = pore\ water\ \ pressure\ [kPa] \end{cases}$$

If u is greater than σ_v hydraulic fracturing will occur.

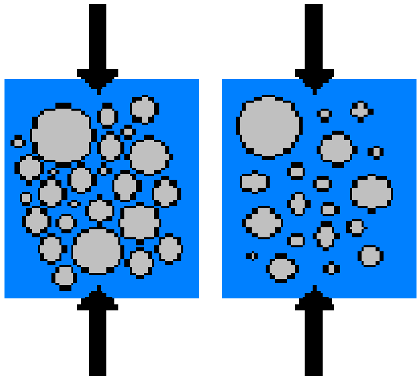


Figure 3-11. On the left the vertical stress from the soil is greater than the pore water pressure while the pore water pressure is greater than the vertical stress on the right side.

Figure 3-10 shows a fault tree when hydraulic fracturing occurs. As seen, cracking, arching and suspension cause low-stress conditions. Cracking has been described in *chapter 3.4* but arching and suspension are different from cracking.

Arching is a phenomenon of pressure transferring. (Terzaghi et al 1996). Arching is shown in *figure 3-12*.

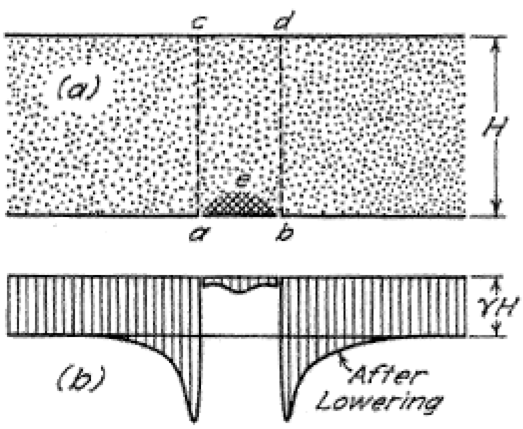


Figure 3-12. Apparatus for investigating arching in a layer of sand (Terzaghi et al 1996).

Figure 3-12 above shows a simple test, discovering arching. A layer of sand is placed inside a box with a small trap door at the bottom. The trap door is slowly opened downwards. The bottom part of *figure 3-12* demonstrates the pressures acting on both the bottom of the box and on the door. The pressure on the trap door is considerably smaller than the pressure on the bottom of the floor. This occurs because the descent of the prism of sand located above the yielding door is resisted by shearing stresses along its lateral boundaries, a*c* and *bd* (Terzaghi et al 1996). The practical effect of this in a body of soil is a lowering of σ_v (see formula **(1)**).

Suspension takes place at locations where the core soil interface with either concrete structures (spillways) or abutments (rock). The interface of core soil and concrete structures/abutments results in a local relief of the vertical stress in the soil. This is due to the roughness and rigidity of the wall. Since the core soil may move and the walls do not, an upward resultant due to active earth pressure will act on the soil. This will relieve the soil, thus the σ_v (see formula **(1)**) will diminish, see *figure 3-13* below:

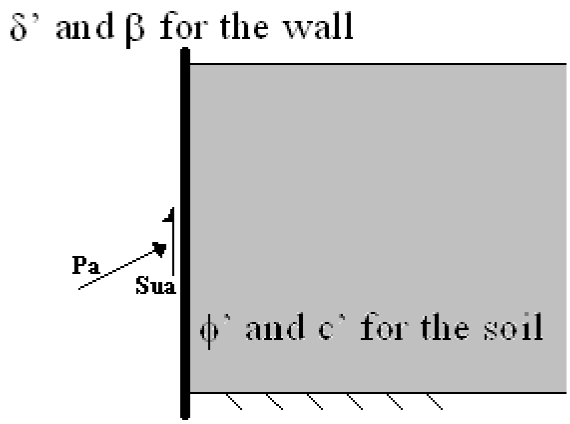


Figure 3-13. Influence factors on suspension against a wall. P_a is the active earth pressure given by δ', wall friction. $S_{u,a}$ is the active adhesive force given by β, factor of adhesion.

3.5 Damages in specific cases

Until now only damages in general such as internal erosion and formation of cracks have been discussed. However there are parts of a dam that is more commonly exposed to damages that may lead to internal erosion.

3.5.1 Damages in the interface dam/concrete structures

Damages at the interface core and concrete structures, such as spillways are common problems in dams. This is primary due to difficulties like compaction of the soil during time of construction at these locations. At this specific location, the concrete act as an impermeable hinder that redirects the seepage along the surface of the concrete. The result is a larger flow of water and in combination with poor compaction plus a possible suspension of the core soil this particular area will be subjected to erosion.

According to (Nilsson et al 1999) this is because:

- Difficulties in compaction of the soil;
- Suspension of the core soil towards the concrete wall;
- The core soil has been dry compacted in order to facilitate construction work. When the reservoir is filled this area will be more prone to consolidate in comparison to the rest of the core.

3.5.2 Damages in the connections between dam/surveillance equipment

Damages in the connections between dam and surveillance equipment is a paradox. Surveillance equipment with the task of detecting anomalies may actually work as a catalyst for internal erosion. The reason for this is the same as the reason for damages to occur between the dam and concrete structures – poor compaction.

Surveillance equipment is incorporated in the dam from the start or dug in place after the time of construction. The equipment normally consists of cables or tubes, which are installed in a desired part of the dam. This poses normally no threat unless the equipment is installed within the core due to its higher cohesion.

4 Remedial Grouting Methods

This chapter will present various forms of possible remedial methods. Some of them have never been implemented in an embankment dam but they still need to be examined in order to decide whether they are of interest or not. Studying remedial methods may also give an idea of how to design and optimize a possible grout. It is mentioned in *chapter 1.2* that the objective of the thesis is to use injection grouting to compensate for a loss of material in the core. Since the internal erosion process creates a loss of material inside the core the best way of replacing it without dismantling the dam is via injection grouting.

The choice of grouting technique depends on the intention. In different soils different methods are used. One grout work might diminish seepage and another might raise the geotechnical stability. It is therefore vital to know that different problems demand different solutions. This means that the same grouting procedure from one dam to another should not be done without consideration.

The principal procedure of remedial work is to:

1. Seal the leak;
2. Mend the damage.

The first step is often done with a coarse, hardening grout. This will stop the major part of the leaking seepage and make way for the second step, mending the damage. These mending actions will be described in detail and evaluated.

It is important to know that the scale of the leakage may vary greatly. Sometimes the first step is unnecessary.

4.1 Different interventions

Depending on during which stage the internal erosion is discovered, different interventions need to be taken, the interventions are divided into three steps:

- Long-term interventions;
- Short-term interventions;
- Heroic/crisis interventions.

The interventions for each step is proposed below (Bradley et al 2007):

- **Long term** - Filters/drains, positive cutoffs, **grouting**, filters/drainage berms and relief wells;
- **Short-term** – Modified operations, reservoir drawdown, reservoir restrictions, **grouting**, filters/drains/berms, and relief wells;

- **Heroic/crisis** – Emergency drawdown, evacuation of down-stream residents, filters, down-stream gradient reduction and **crack or pipe filling**.

Grouting could be applicable for all three types of interventions: grouting for short- and long-term and crack-/pipe filling for heroic/crisis situations.

4.2 Grouting

The effectiveness of grouting depends on several factors. These factors are (Brantberger 2000):

- For a **grout curtain** – distance between drilled holes, look-out, depth of drill work, drilling equipment and circulation flush;
- For **the grout** alone – penetration ability, hardening, filling of the voids and durability;
- For the **equipment** – blender, mixer, pumps, dosage- and recording equipment;
- For the **injection methodology** – criterions for grout pressures and stopping, criterion for the grout, order of injection, several holes injected at the same time and intervals of injection.

The grout material can be cement, soil or chemical solutions.

4.2.1 W/C ratio

The viscosity of the grout is important and it is modified depending of the structure of the damage. The viscosity is modified by variation of the water-to-cement ratio. If cement isn't used as a grout but soil material only, the expression water-to-cement can be exchanged to soil-to-water ratio. In fact this will give the water content of the soil.

The definition for water-to-cement is:

$$w/c = \frac{m_w}{m_c} \qquad \textbf{(2)}$$

Where in **(2)**:

$$\begin{cases} m_w = mass\ of\ \ water[kg] \\ m_c = mass\ of\ \ cement[kg] \end{cases}$$

Furthermore the w/c ratios for grouts are (Ekström et al 2001):

- For smaller cracks a w/c ratio of 1 to 3;
- For larger cracks and voids a w/c ratio of 0,5 to 1.

Different problems are associated with different w/c ratios. At low ratios the grout becomes stiff, thus harder to work with and there is a possibility that all the cement does not react with the water. A higher w/c ratio cause larger quantity of excessive water and the grout is not as durable as grouts with lower w/c ratios. In other word there are large contrasts between lower and higher w/c ratios. Lower w/c ratios make the grout more durable and denser. Meanwhile, its penetrability is lower and its time to harden is longer compared with grouts with higher w/c ratios.

The main focus of this thesis is to examine remedial work with grouting and its possibility as a future method.

4.2.2 Drilling

If grouting is performed, holes into the dam must be drilled. This will avoid excavation work. There are various drilling methods but there are similarities between them. In operation, a drill head rotates a string of drill rods, which are constantly lengthened as the depth of the drill hole increases. At the end of the drill string the drill bit is found, see *figure 4-1*.

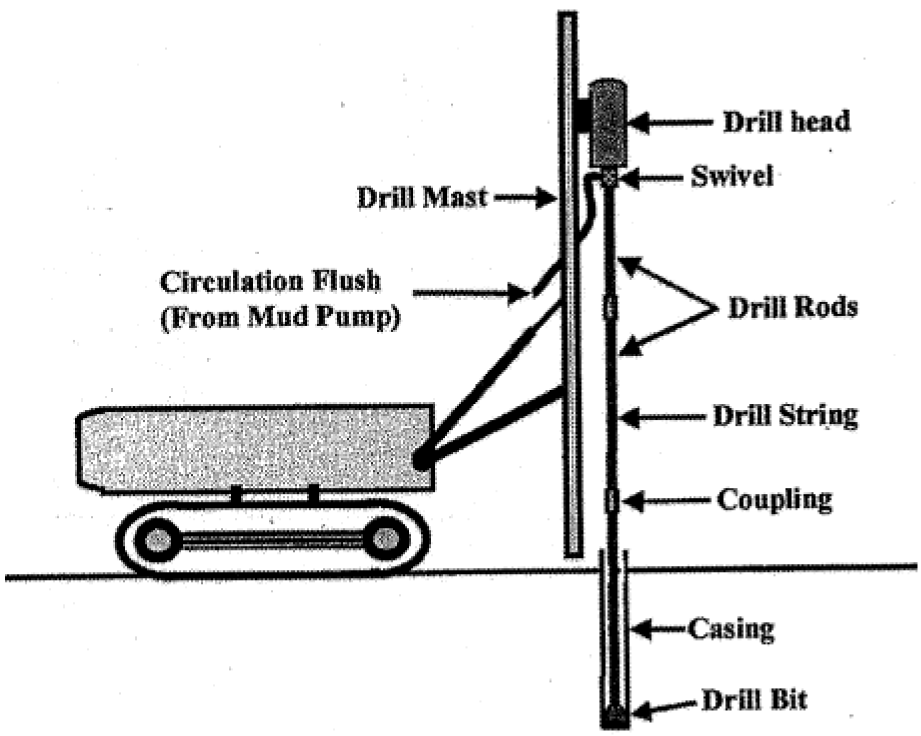


Figure 4-1. Basic elements of a drill rig (Warner 2004).

As the work continues, casings must be installed to prevent the hole from collapsing. The casings also fulfil another function: the flushing out of drill cuttings. The drill string is hollow and connected to a swivel. Air or fluid is pushed into the drill string and escapes at the very bottom of the drill bit. At this point, it suspends the drill cutting, transporting it upwards inside the casing to ground level, see *figure 4-2*. This action, done with water or fluid, is known as circulation flush.

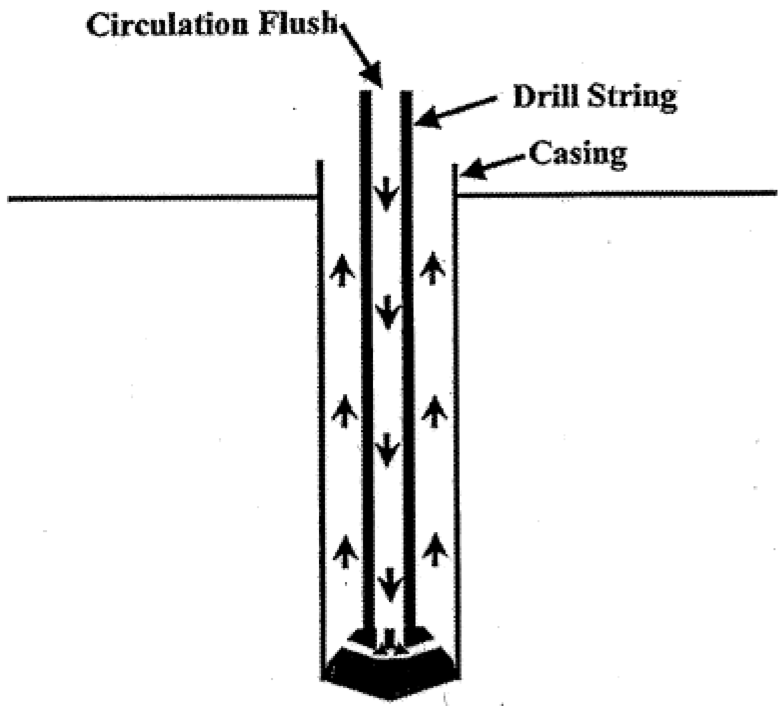


Figure 4-2. The drill cuttings are removed by the flow of the circulation flush (Warner 2004).

There are three general classifications of drilling work for grouting (Warner 2004):

- Rotary;
- Rotary percussion;
- Driving.

Rotary is commonly used and if *percussion* is added, the work will normally be done in a faster rate. Percussion is included if the material is harder than normal soil but is unnecessary if drilling is not done in rock or concrete. The expression percussion relates to a rapid pounding action through e drill string. Rotary drilling only is the most frequently used type of drilling in soil only (Warner 2004). Handheld rotary equipment can, and has been used for drilling holes in soil with depths to 30m. From this point of view, a drill rig shouldn't be necessary for shallow drill holes if considering the desired depth alone. However, if drilling in a dam the main goal is not only to reach a desired depth but also to reach the actual damaged part. This will ensure a greater probability of success of the grouting procedure. Monitoring of the drilling should be done in order to guarantee this.

The third class in the classification relates to drilling that involves physical *driving* of a casing into the ground. A hydraulic or a pneumatic type of hammer provides the driving force. No drilling is actually involved so in other words, this is not a drilling method. Drivers that supply high energy, low-frequency pounding have been found to be much less damaging to its environment in comparison to high frequency pounding (Warner 2004).

4.2.3 Grouting pressures

Depending on the chosen method, a good knowledge about grout pressures is important. Grouting pressures may vary depending on the injection depth and closeness to other zones in the dam. A key issue is not to harm the dam when performing injection, however, higher grout pressures will ensure more durable reparation but also elevate the risk of damaging. Necessary grout pressures must be carefully evaluated before performing remedial work.

4.2.4 Surveillance of the grouting procedure

In any kind of injection work careful surveillance and monitoring must be performed since it is impossible to actually see the event underground. Surveillance offers a simple way of determining whether the procedure have performed as intended or not. Monitoring is important because the grout procedure must not damage the dam structure any further. Creating further damages on the soil when grouting must be avoided in particular if grouting is used in dams.

The most commonly used method is the surveillance of the pressure- and flow curve (Sjöström 1999). At any injection work, the grout pressure changes through time depending on the size of the grouted void. The void is injected and when a pre-determined pressure is reached the injection is aborted. This is done in order to avoid unintentional fracturing of the surrounding soil. Necessary criteria for aborting the injection should be carefully evaluated before the injection procedure starts. If the pressure of the grout suddenly lowers during injection work, the void must have been rapidly enlarged - fracturing of the soil has occurred. To reach the intended pressure is of high importance if considering the durability and permeability of the grouting.

The rate of the flow is as important as the monitoring of the pressure. A low, constant flow of grout is desired if grouting in dams since the grout must allow the pore water to escape the surrounding soil structure. This rate should be modified according to the permeability of the soil around the damaged part. If this is not properly done the soil might fracture. The build up of the pressure is decided by the rate of injection. The build-up of pore water pressures may be monitored with piezometers (Garner et al. 2000).

A log from injection grout work at Bennet dam, B.C., Canada can be seen in *figure 4-3*. Monitoring of specific energy [kPa], collar pressure [kPa] and take $\left[{}^{l}\!/_{0,3m} \right]$ have been performed.

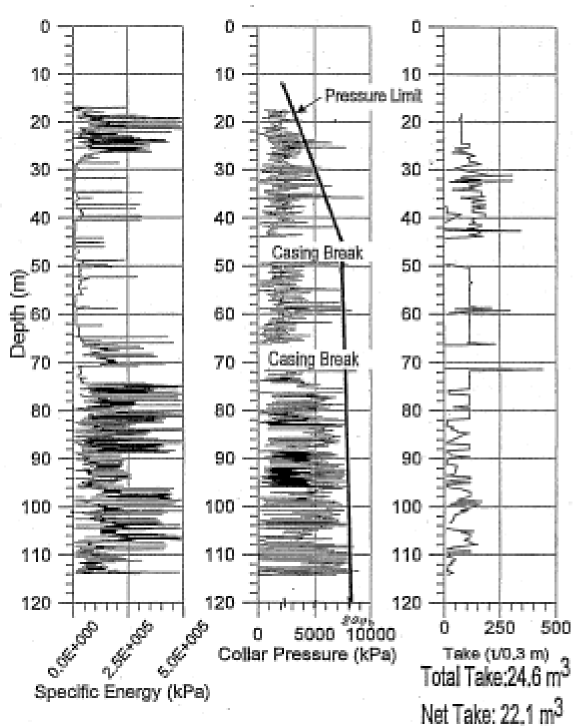


Figure 4-3. Summary from log during compaction grouting at Bennet dam. The black, straight in the graph marked "Collar pressure (kPa)" represent the maximal allowed pressure at any depth (Garner et al 2000).

4.3 The remedial methods

Most of the described methods are widely used today and have been so for many years. It is important to know that all of the methods have not been implemented in liaison with remedial work on dams. Except from retaining walls and diaphragm walls, all of the methods have in common that they include injection work.

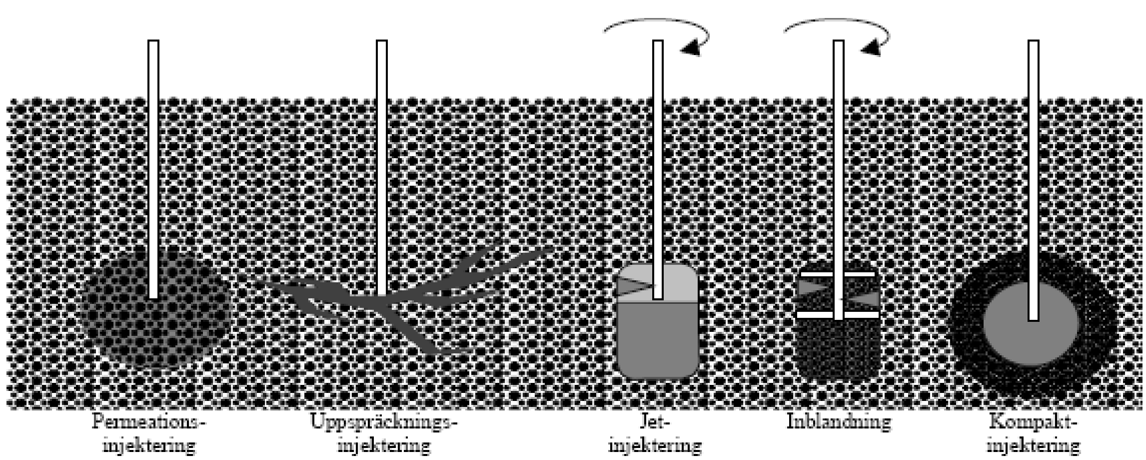


Figure 4-4. Different types of injection methods. From left to right: Permeation grouting, fracture grouting, jet grouting, mixing and compaction grouting (Hansson 1999).

Examined methods are:

- Permeation grouting;
- Fracture grouting;
- Compaction grouting;
- Soil grouting;
- Inert grouting;
- Jet grouting;
- Diaphragm wall.

A short description of bentonite clay will also be given since they often are used as additives in soil grouting.

4.3.1 Permeation grouting

Permeation grouting is most often used in permeable soils. The method does not rupture the original soil structure. The expression permeable soils relates to non-cohesive soils where the porosity and permeability is higher. The grout has high w/c ratios and penetrates the pores of the soil plus eventual larger voids and is left to harden. If grouting for water control, essentially 100 percent of the pore void system must be filled. For strengthening operations a lesser degree of filling is acceptable. In strengthening operations, the primary mechanism of improvement is through adhesion that glues the soil particles together (Warner 2004).

As a rule of thumb the grout particles shouldn't be more than one fifth of the size of the pores in order to ensure proper penetration. Permeation grouting is suitable for soils with permeability > 10^{-6} m/s . If the method is used in soils with a lower permeability there is a risk of hydraulic fracturing, since the grout cannot enter the pores (Widing 1987).

Important geotechnical parameters for permeation grouting are:

- Permeability;
- Porosity;
- Knowledge of the layering of the soil;
- Seepage.

In a dam, seepage is difficult to predict but in and around a core-damage it must be considered high. This makes it difficult to inject the intended area since the grout is easily washed away.

The largest single use of permeation grouting in soils is in short-term strengthening, which is used during construction work. Accordingly, the strength and durability of the grouted mass is pertinent only until the end of constructed. This is an important factor, as most chemical solution grouts do not provide long-term or permanent strength (Warner 2004). The fact that the long-term factor isn't satisfying makes the method costly.

4.3.1.1 Permeation grouting with chemical solution grouts

A widely used chemical solution grout is water glass, also known as sodium silicate. The water glass function in two steps:

- Penetration;
- Hardening.

Water glass gains solidification through interference of an acid. In injection work with water glass it is unpractical to mix the grout with the acid instantly since this will diminish the intended efficiency of the penetration. Instead a hardener is added, which after a short period of time forms acid (Hansson 2005).

The impression of water glass is more similar to water than ordinary grouts. Thanks to this the water glass penetrates the soil and its voids easily. The primary strengthening function is through adhesion.

As already mentioned, there are many questions about the life span of permeation grouts and in particular, water glass. The water glass also tends to shrink during and after the process of solidification. Shrinkage of the grout is highly undesirable since it leaves the pores and voids in the soil untreated.

If the water glass is disturbed during its hardening phase, which might happen due to large seepage flow, the process of hardening can be stopped (Hansson 2005).

4.3.1.2 Permeation grouting with cement

Cement based grouts are also used, most commonly in form of slurrys with fine graded cement. Fly ashes might also be used. If the soil is relative coarse (d_{min} = 0,4mm) ordinary cement can be used. The characteristic of the slurry can be controlled via variation of the water-to-cement ratio and different additives. This is where the rule of thumb is important.

4.3.2 Fracture grouting

When grouting via fracture grouting the soil is intentionally fractured. The grout is injected under high pressure thus creating cracks in the soil, which is sealed by the grout. After the procedure the grout is left to harden. This will enhance the strength of the soil through a network of interconnected grout lenses. It will also lower the permeability of the soil since the grout lenses force the seepage to take longer routs through the soil.

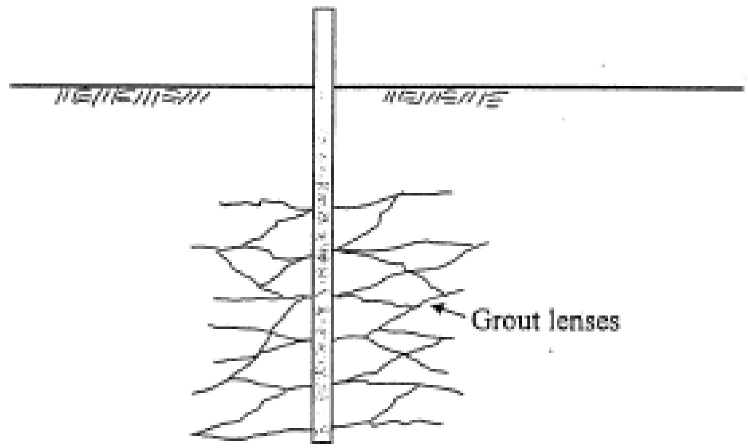


Figure 4-5. Grout lenses in the soil (Warner 2004).

The most common grout materials are cement based with high w/c ratios. As an alternative, high-calcium or hydrated lime slurrys are sometimes used.

The grout alone cannot handle tensile forces, thus polymer fibres are mixed with the grout (Warner 2004). Sleeve port pipes are generally used for the grout injection, with the injection ports spaced at fairly small intervals. In order to prevent excessive fracturing the quantity of grout placed at one location is usually limited from the start. The grout limitation is based upon the appearance of the soil structure.

The greatest difficulty with fraction grouting is the problem of controlling the direction of the fractions. Exceptional care must be taken to ensure that the intended location of the grout is reached (Warner 2004). Excavations after test injections also show that the lenses are seldom connected as intended.

Fracture grouting is most often used in low permeable soils due to their nature of fracturing, which should make it suitable in the core of a dam. However, since the control of the fracturing is virtually impossible this makes it unsuitable as a remedial method in a dam. There is also a risk of "building bridges" between the core and filter with the grout since the grout might cross the two zones.

4.3.3 Compaction grouting

Compaction grouting is the most common soil grouting method (Warner 2004), hence the best known. The name of the method describes its function, to compact the damaged soil. Compaction grouting is versatile and is applicable in any kind of soil type but most commonly in soils ranging from sand to silt. Compaction grouting can also be performed in clays. Precautions must be taken after the cavity is filled to avoid fracturing of the soil. This requires a skilled operator to abort the injection work when achieving the desired grout pressure.

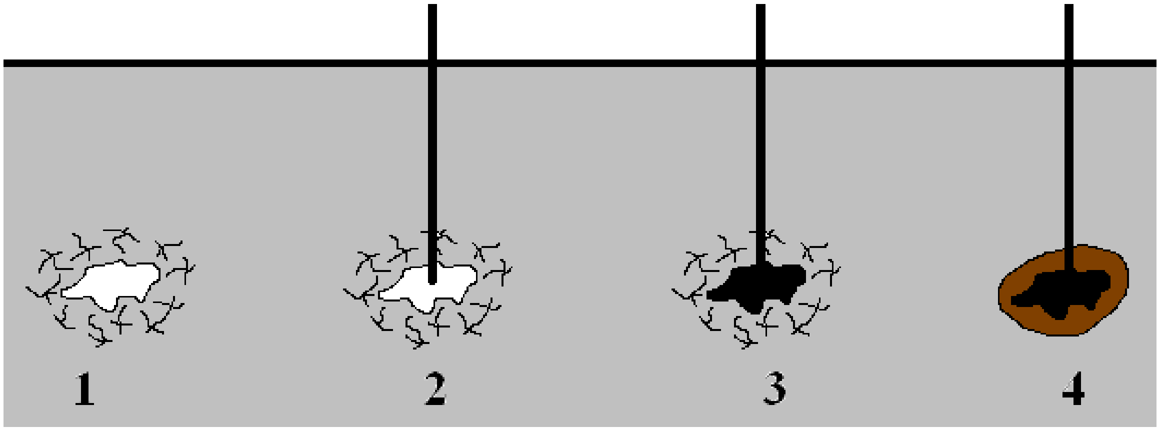


Figure 4-6. Methodology of compaction grouting. 1 – A cavity, the vicinity is loosely compacted, 2 – Drilling work and a nozzle put in place, 3 – Grout work begins, filling the cavity, 4- The cavity is filled and the adjacent soil is compacted.

The grout is denser in comparison with the grout used for permeation- or fracture grouting. This fact allows the grout to undergo radial expansion, similar to a balloon. The expansion can however cause problems in the original soil since shear forces may evolve at the interface of the expanding grout and the soil. If the original soil is highly cohesive it might be fractured. However, since the soil around the cavity is already loosely compacted this problem may be considered less serious – The fracturing has already occurred.

Since the grout expands and compact the surrounding soil it is important that the grouting is done at slow rate. This fact will allow the pore water to safely emit without cause a hydraulic fracturing of the soil. Rapid compaction grouting compresses the pore water into the original soil where high pressures may cause damage if not drained properly (Warner 2004).

The most significant difference attained with compaction grouting is, excluding the injected cavity, the degree of compaction of the soil around the cavity (Hansson 1999). The grout is never intended to interfere with the surrounding soil such as permeation grouting and jet grouting.

4.3.3.1 The composition of the grout

The grout must be rigid since it may not enter the pores of the soil. The grout can be based on either cement or soil. An important factor is that the water-to-cement or water-to-soil factor is low in comparison with the relative high water-to-cement ratio for fracture grouting. The angle of internal friction and the viscosity must be high. If soil is used as a grout, there are several particle sizes that need to be included. Sand is added to raise the angle of internal friction and silt is needed to attain the right pump ability of the grout.

4.3.3.2 Grouting pressures

In a dam subjected to internal erosion the fines have escaped the structure. This leads to a loss of material within the dam structure. The appearance of the damage may differ but they have in common an elevation of the void ratio, **e,** either alone or around a pipe.

$$e = \frac{V_a + V_w}{V_s} = \frac{V_v}{V_s}$$
$$0 < e < \infty$$

(3)

Higher void ratios indicate a higher content of water or air in a given body of soil, thus leading to a higher permeability. This relation can be seen in *figure 4-7*. As already discussed in *chapter 3*, high permeability leads to a larger rate of seepage, which in turn elevates the risk of internal erosion.

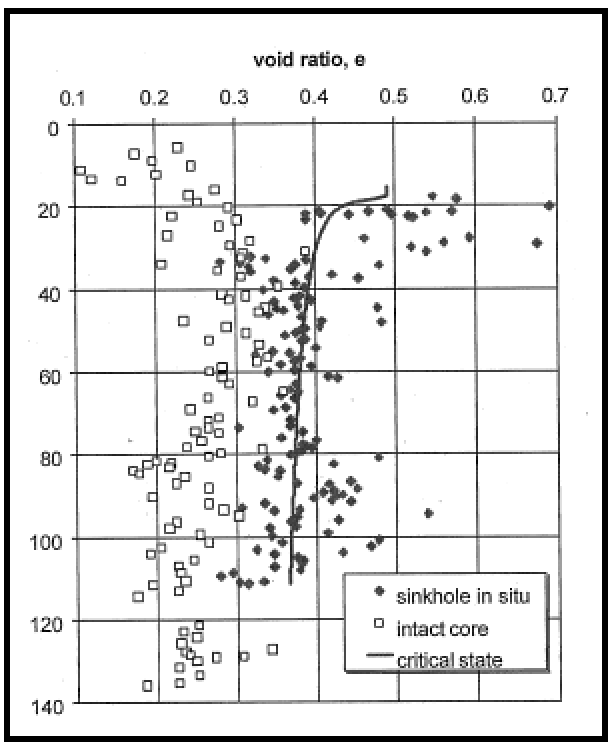


Figure 4-7. Relation between void ratio and the occurrence of sinkholes at Bennet Dam, Canada. The y-axis indicates the depth (Shuttle et al, *Prediction and validation of compaction grout effectiveness*).

Compaction grouting can and has been used to efficiently reduce the void ratio.

Numerical models should be included in the evaluation of allowable maximum grout pressures (Warner et al 2000).

4.3.4 Soil grouting

Soil grouting has many similarities with fraction grouting. The main difference is the grout used. With soil grouting, the grout used is primary clay. This method has been successfully used in embankment dams along the Yellow river in China (Chen 1982). The method is cheap and the swelling ability of clay makes it suitable for remedial work in dams. The swelling functions in the same way as compaction grouting – the soil around the grouted part densifies. The usage of clay is however linked with a lesser degree of durability (Warner 2004). Clay is easily washed out from the dam because of the seepage. Now days, grouting with clay only is seldom done without an addition of standard Portland cement.

The method is applicable in loose soils. The loose soil is easily fractured and the soil grout seals these fractures. Since the grout swells, a compaction will occur that keep the soil in place by mechanical locking. After a correct procedure the permeability of the injected area can be as low as $10^{-8}\,{}^{m}\!/_{s}$ (Chen 1982). If the grout intercepts smaller conduits within the soil structure, it can follow these and seal them.

4.3.4.1 The composition of the grout

The grout consists of soils with various particle sizes and sometimes cement. Clay is the most common material since it is easier for clay to fracture the original soil. The mixture is defined as clay-to-cement and its density is usually around 1,2 – 1,4 ${}^{t}\!/_{m^3}$. Via experiences using this method, a more dense mixture is desired since it enhances the durability of the remedial work.

If the injection is done in a core with a lower too much lower water ratio than optimum, the injection can be done in two steps. First, a grout with a less content of clay will be injected. This will raise the water ratio in the original soil. After this an injection is performed with a grout that contains more clay that more easily penetrates the original soil (Chen 1982).

4.3.4.2 Bentonite

Since bentonite is commonly used in grout work it must also be described. Bentonite is a clay of expansive nature. It is used as an additive in order to prevent separation during the grout process. Bentonite is today widely used all over the world.

The bentonite possesses strong swelling characteristics in contact with water and is mainly constituted of the mineral montmorillonite with an addition of other minerals from the smectite group. These in turn consist of aluminium silicates with iron, magnesia and sodium or calcium. The montmorillonite is formed when a basic stone material such as volcanic ash is sedimented in a marine environment. From a geotechnical point of view the bentonite is considered weak with low bearing capacities. There are two main types of bentonite:

- Sodium-based bentonite;
- Calcium-based bentonite.

The *sodium-based bentonite* is well known for its extreme swelling potential. There are reports of swelling between 15 – 18 times its own original volume when wetted. The swelling is caused by the absorption of water molecules in the molecular structure of the bentonite. As an additive in grout work the bentonite raise the viscosity of the grout, thus preventing excessive sedimentation. It can also lower the strength of cement if mixed with cement bound grouts. The *calcium-based bentonite* also swell if wetted but to a lesser degree if compared to the sodium-based bentonite.

The amount of bentonite as an additive in concrete grout depends on the w/c ratio. For grouts with a w/c ratio of 0,5, about 2% is normally added. Grout with a higher w/c ratio, about 5 – 7% is added (Widing 1987).

4.3.4.2.1 Sensitivity to erosion

The hydraulic conductivity of the bentonite is very low. The sodium-based bentonite varies between 2×10^{-5} to $4\times10^{-14}\ {}^{m}/_{s}$ and the calcium-based bentonite varies between 1×10^{-5} to $2\times10^{-13}\ {}^{m}/_{s}$. The main influence of the hydraulic conductivity is the presence of organic matter and the pH-value of the water. The clay structure is normally very homogenous at a low concentration of salt. If the concentration of salt in the water is raised the clay might coagulate, thus pores will be formed raising the hydraulic conductivity. The presence of organic matters affects the bentonite in a similar way.

Since the velocity of seepage is a decisive factor for the probability of erosion, an augmentation of the hydraulic conductivity is dangerous. Clay in general is easily eroded since the particle fractions are extremely small. If the hydraulic conductivity is higher than $10^{-6}\ {}^{m}/_{s}$ the clay starts to erode (Ekström et al 2001).

4.3.4.2.2 Swelling pressures

The swelling pressures of bentonite always have to be exceeded by exterior forces such as lateral and horizontal effective soil pressures (Ekström et al 2001). The swelling pressure of "clean" bentonite has two decisive factors:

- The density of the bentonite;
- The salinity of the water.

Type of bentonite	Density [kg/m^3]	Swelling pressures, p_s [kPa]	
		Fresh water	Salt water
Sodium bentonite	1400	100	0
Sodium bentonite	1600	300	40
Sodium bentonite	1800	800	300
Sodium bentonite	2000	4000	4000
Calcium bentonite	1400	0	0
Calcium bentonite	1600	20	0
Calcium bentonite	1800	500	200
Calcium bentonite	2000	4000	4000

Table 4-1. Swelling pressures of smectite clay with montmorillonite content of 70 – 80%. (Ekström et al 2001).

Since the density of bentonite in average is $2350\,{}^{kg}\!/_{m^3}$ the ***"Density column"*** can be recalculated into a w/c ratio, but in this case a water-to-bentonite ratio using the formula:

$$\rho_{Bentonite+H_2O} = \frac{m_b + m_{H_2O}}{{}^{m_b}\!/_{\rho_b} + {}^{m_{H_2O}}\!/_{\rho_{H_2O}}} \tag{4}$$

Where **(4)** can be re-written:

$$\rho_{Bentonite+H_2O} = \frac{m_b + m_{H_2O}}{V_b + V_{H_2O}} \tag{5}$$

where in **(5)**:

$$\begin{cases} m_b = mass\ of\ bentonite\ [kg] \\ m_w = mass\ of\ water\ [kg] \\ V_b = volume\ of\ bentonite\ [m^3] \\ V_w = volume\ of\ water\ [m^3] \\ \rho_b = density\ of\ bentonite\ \left[{kg}/{m^3}\right] \\ \rho_{H_2O} = density\ of\ water\ \left[{kg}/{m^3}\right] \end{cases}$$

The usage of **(5)** gives *figure 4-8*:

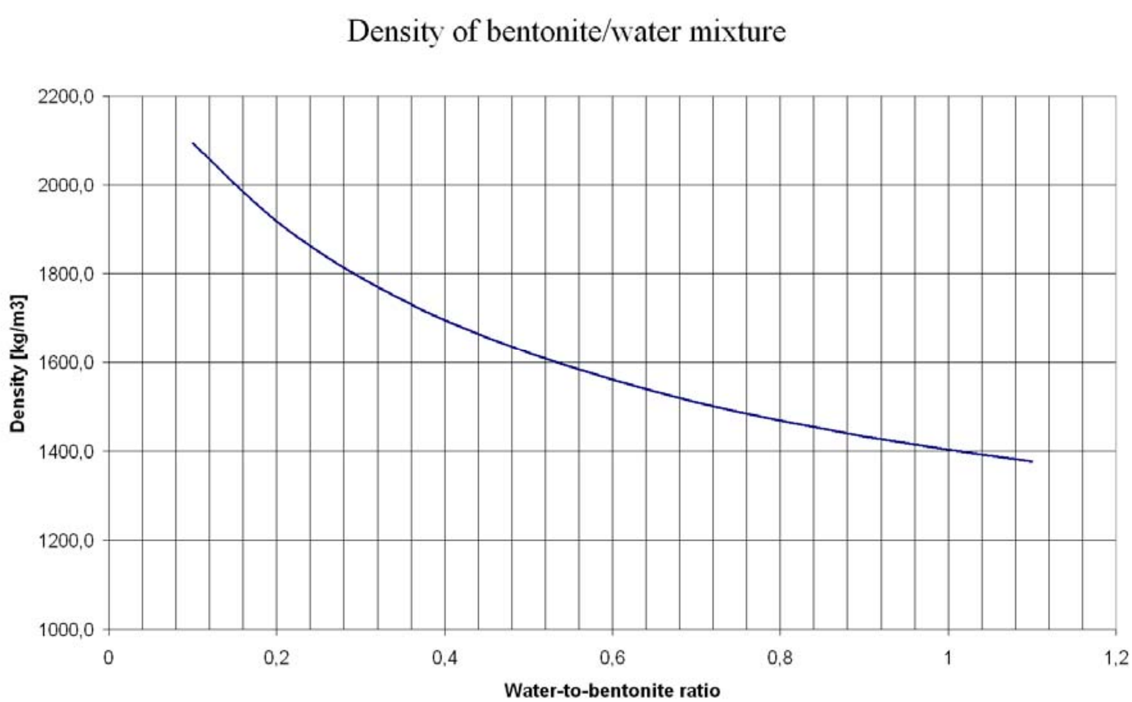


Figure 4-8. Water/bentonite mixture with its water-to-bentonite ratio given the density of the mixture.

Furthermore, *figure 4-8* indirect shows the amount of added water to attain the desired density:

Density of mixture $\left[kg/m^3 \right]$	Water-to-bentonite ratio $[-]$
1400	1,00
1600	0,50
1800	0,30
2000	0,15

Table 4-2. Water-to-bentonite ratio. Should be compared with table 4-1

Reading *table 4-1* gives an impression of the expansive forces of the bentonite. Under the "right" circumstances, swelling pressures can grow as large as 4.000 kPa within the bentonite. *Table 4-2* gives an idea of the necessary amount of water for each mixture of bentonite/water.

4.3.5 Grouting with inert material

An inert material does not give raise to any kind of chemical reactions in the grout. Today only pilot tests have been performed and the material used have been grounded dolomite. The particle size distribution during these pilots has been equivalent to ultra fine cement 16. The degree of compaction can be controlled by the addition of superplastisisers, used for concrete and grouting (Hansson travel report)

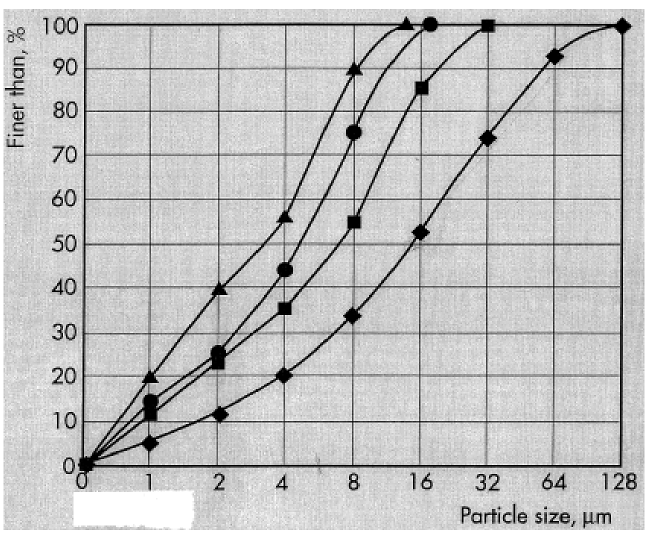


Figure 4-9. Particle size distribution curve for Ultra fine cement 16 (curve with circles imbedded) and Ultra fine 12 (Heidelberg Cement)

4.3.5.1 Principal function of the inert grout

As already mentioned the grout does not undergo any chemical reactions before, during or after the injection. The dolomite is mixed with water and superplastisisers. It is injected and the material is left to deposit and self-compact. It is important that the excess water is drained; otherwise the process of sedimentation and self-compaction will not be efficient. The excess water should be drained with special drainage holes (Hansson 1999).

When the material has undergone compaction it has thixotropic characteristics.

4.3.6 Jet grouting

Jet grouting is a method where the fines of a soil structure are replaced with hardening particles under high pressure. The coarser part of the soil forms a new, harder structure together with the grout (Windelhed 2001). The methodology is to drill a hole at the intended location. A nozzle is placed into the hole. The nozzle spins around its own axis and while doing so, it washes out the fines from the soil with the use of water or air. After the washing out of fines, a cement- or cement/bentonite grout replace the pores, recently filled with the washed out fines. The whole process is manageable by using one nozzle only. During the process the nozzle constantly moves upwards.

The final product is a pillar formed structure with a permeability of 10^{-7} to 10^{-9} m/s, which is basically the same as the core in a dam (Windelhed 2001).

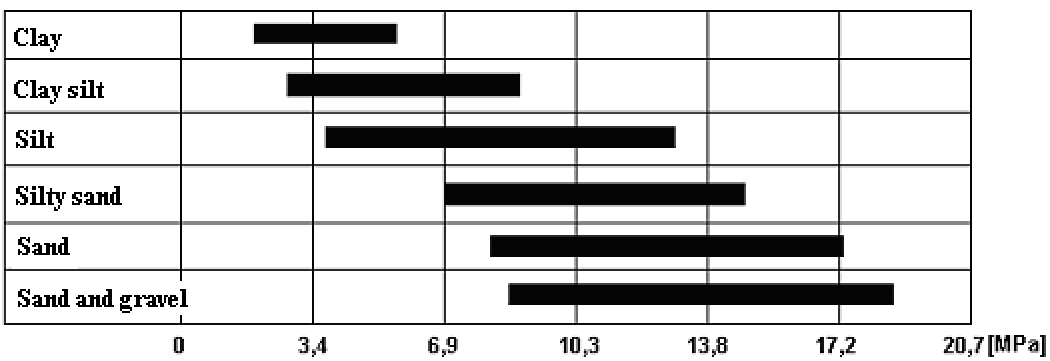


Figure 4-10. Final strength of jet grouted pillars in different soils (Warner 2004)

Since clays are less pervious the exchangement between fines and grout, the method is less suitable for clays. Another problem with jet grouting is that the washed out fines must be permitted to escape the soil structure. This is done by an uplifting action provided by the constant elevation of the nozzle. If the soil is too dense, this action will not be completed, thus the soil may be unintentionally hydraulic fractured.

4.3.7 Diaphragm walls

Although not an injection grouting technique, diaphragm walls are often the choice if an embankment dam suffers extensive damaging. The execution of this technique is well known since the method is widely used in ordinary construction work. It is well suited for clay rich environments (Charles et al 1996). A trench is vertically dug with a special excavation along the axis of the dam, see *figure 4-11*, and is continuously filled with bentonite slurry. The slurry will prevent the walls of the trench to collapse. In small dams the slurry also contains cement. In this case, replacement of the bentonite only slurry will not be needed. This method is also known as the single-phase method. A further development of the single-phase method is the double-phase method where the original slurry is replaced by plastic concrete (Charles et al 1996).

4.3.7.1 Single-phase method

In smaller dams where the soil pressures are smaller, single-phase method can be used. The supportive cement/bentonite slurry used during excavation is left to harden without any further modifications. The water ratio of the slurry is normally between 250 – 400%, thus shrinkage will occur if the slurry is not covered. The shrinkage can also cause severe cracking in the wall. The final product is brittle at low horizontal pressures and plastic behaviour is only attained during drained conditions in combination with high effective pressures of at least 100kPa (Charles et al 1996). The weight of the hardened wall is as low as 1,1 - 1,2 t/m^3, which indicates a high content of water.

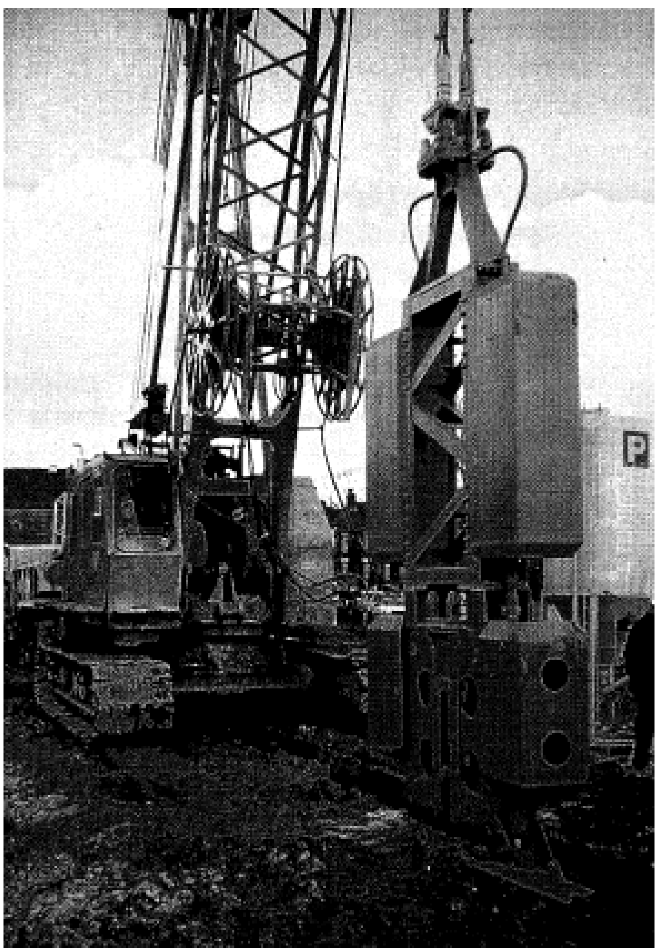


Figure 4-11. Excavator during construction phase of a diaphragm wall (Charles et al 1996)

4.3.7.2 Double-phase method

The only difference between single-phase and double-phase method is the substitution of the original slurry. The slurry is exchanged with a mixture of cement, ballast, bentonite, water and sometimes even pulverizes fuel ash (fly ash). The concrete is placed in the trench by using a tremie pipe, thus replacing the slurry. The moulding is done in sections with smaller, connected panels as a result.

The usage of the two-step method has its advantages within larger dams and work has been performed to depths exceeding 100m. The final product is less sensitive to erosion and stronger in comparison with the single-phase method.

4.4 Conclusions of the grout method study

The conclusion of the performed literature study of the grouting methods will be done. This refers to find the most suitable grouting technique that may be suitable in an embankment dam subjected to internal erosion. It will be done in two steps:

1. Evaluation of the grouting techniques;
2. Evaluation of important factors for the injection process.

4.4.1 Grouting techniques

Some evaluations of advantages and disadvantages for possible implementation in an embankment dam have already been made in this chapter. However, these evaluations need to be concretized, which will be made in this segment of the thesis.

4.4.1.1 Permeation grouting

Advantages

- Permeation grouting does not damage the original structure of the dam;
- Is better suited in soils with a loss of fines.

Disadvantages

- The durability is questionable;
- Difficulties for the grout to penetrate all the desired pores and voids before hardening;
- Loss of volume of the injected grout during and after hardening.

There are specific question marks with the durability of water glass. It is hard to evaluate laboratory tests done with this type of grout (Hansson 2005). Since the w/c ratio is higher the grout suffers greater risk of shrinkage during and after hardening. Furthermore, good knowledge of the extent of the damage is needed. This method will probably not be suitable for damaging by piping.

4.4.1.2 Fraction grouting

Advantages

- Lowers the permeability of the soil, thus diminishing the velocity and rate of the seepage;
- Suitable for low permeable soils due to its nature of fracturing.

Disadvantages

- Difficulties in controlling the fractures;
- Cracks are not desired in the core.

Fraction grouting should not be used in a dam. Cracking of the core is not desired in the core due to the risk of creating faster seepage routs. Faster passage of the seepage raises the risk for internal erosion in the core. The grout filled cracks may also create short cuts between the filter and the core. This may result in jeopardizing of the filter rules.

4.4.1.3 Compaction grouting

Advantages

- Compacts the soil, thus lowering its permeability;
- Well suited for any kind of loosely compacted soil type;
- Flexible. The grout may consist of various materials.

Disadvantages

- High demands on the operator in order not to fracture the soil;
- It may damage the inner structure of the dam.

The method is widely used hence well known. Compaction grouting put demands on the operator in order to prevent unintentional fracturing of the core soil. Furthermore the method requires good knowledge of the orientation of the damaged part.

4.4.1.4 Soil grouting

Advantages

- Material similar to the core is used as grout;
- The method is very cheap in comparison with other methods.

Disadvantages

- Fractures the soil;
- Low durability depending on the water-to-soil ratio.

Grouting with soil is a well-known method and has been used in large-scale remedial projects. As long as the seepage velocity remains low there is a lesser degree of erosion of the injected soil (normally clay). An interesting factor for this method is the swelling ability of the clay.

4.4.1.5 Grouting with inert material

Advantages

- Injected material does not interact chemically with the water;
- Self compacts.

Disadvantages

- Drainage of excessive water is difficult;
- The method is not well tested.

The inert grouting seems interesting because the procedure does not disturb the core soil while compacting. The final characteristic of the soil is plastic, which means that it is flexible. Unfortunately, the knowledge of this method is very limited. There might also be problems when completely filling a larger void.

4.4.1.6 Jet grouting

Advantages

- Final attained permeability is low. Similar to the original permeability of the core;
- Well suited for soil within the range of sand to gravel.

Disadvantages

- Difficulties with the drainage of the fines;
- The strength of the grouted pillar becomes too high.

This method is not well suited inside a dam core. It would create hard vertical curtains inside the dam and the migration of fines is difficult in soils with low permeability. The presence of boulders in the core soil may create untreated sections.

4.4.1.7 Diaphragm walls

Advantages

- Can be performed at great depths;
- Works well with the high-cohesive core soil.
- Can be used if the dam suffers extensive damaging.

Disadvantages

- The final product is prone to shrinkage;
- Requires large-scale linings.

This method is not a grouting method but may be the only solution if the dam suffers from extensive damaging along its length. Diaphragm walls efficiently blocks excessive seepage through the dam and hinders fine material from being washed out. The method does not repair the damage/damages inside the dam but it hinders the result of it. The method is well known worldwide.

4.4.2 Important factors for the injection process

If grouting inside the core of an embankment dam damaged by piping, *compaction grouting* technique seems interesting. Compaction grouting may be used in several types of soils, is flexible and offers a wide variety of possible grouts. It has been used many times before. Compaction grouting material can cooperate with the original core soil and when used, the soil is more restored than mended. Compaction grouting technique should have the possibility of restoring dam cores suffering from various kinds of internal erosion damages (piping, smaller canals and suffusion). Compaction grouting will furthermore, if performed correctly, not fracture the original core soil.

However, a main problem to solve is to obtain better knowledge of the location of the damage and developing better suited drilling methods for dam cores. If a remedial grout process intends to be successful with compaction grouting technique, the grout must reach the damaged part inside the core without penetrating the core soil.

4.4.2.1 Grout holes

In order to get the grout into its intended place, drilling must be performed. Great caution must then be taken if the core soil above the damage is intact, which may be the case for piping. Careless drilling work may fracture the original core soil. The main reason can be found in the circulation flush. Circulation flush can raise the pore water pressures in the soil, thus damaging the soil in the vicinity of the drill hole. Circulation flush using air creates similar problems as the previous case. If circulation flush is not used another way of removing the drill cuttings must be used. It should furthermore be necessary to assure that the end of casing joins the actual location of the damage. This can be done through careful surveillance of the drilling work.

Since the core soil is made from till there is a possibility of the presence of boulders and sometimes blocs. Blocs are harder to penetrate hence the drill and/or casing may drift away from its planned trajectory. The result may be a miss of the location of damage inside the core, see *figure 4-12*. However, straight, perfect drill holes are in reality rare. Enhanced accuracy requires a heavier drill string and much greater care taken by the operator. It is thus significantly more time consuming and if a deviation survey is added, the cost may be doubled (Warner 2004). Rotary drilling generally offers straighter holes.

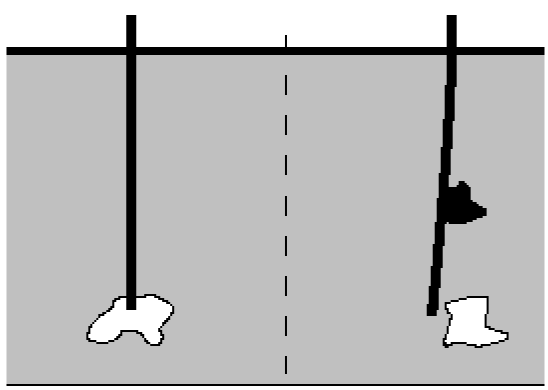


Figure 4-12. Possible deflection of drill/casing in drilling work.

4.4.2.2 Grout material

Many of the characteristics of the grout are controlled by the w/c-ratio. A high w/c-ratio provides a grout with lower viscosity. A low viscosity grout is more easily eroded but offers greater penetration of the damaged soil. Penetration should however not be attained since it may fracture the core soil. A high viscosity grout possesses a low w/c ratio, thus it is more rigid. This type of grout will have a greater durability but not the same penetration ability as a low viscosity grout. Furthermore, it should not be subjected to severe shrinkage, in comparison with grouts with higher w/c-ratios.

The material used as grout must be thoroughly examined. An introduction of high strength zones inside the core can cause cracking and suspension of the core soil. A high-strength body of grout can furthermore hide newer damages from being discovered and provide greater difficulties in performing future remedial work. These types of rigid intrusions within the dam are the result of an introduction of hardening grouts.

The grout may also be constituted of different soils. Clay is commonly used but there are other options as well. Sometimes a mixture of soil with different fraction sizes has been used, such as the case for Bennet Dam, Canada. In this particular case the grout consisted of a mixture of water, pea gravel (20%), sand (50%) and silt (30%) (Hansson 1999). The grout in this case was designed to resemble of the original core soil (particle size distribution curve designed to resemble to the original curve of the core soil). The usage of this type of grout is in this thesis named flexible grouts.

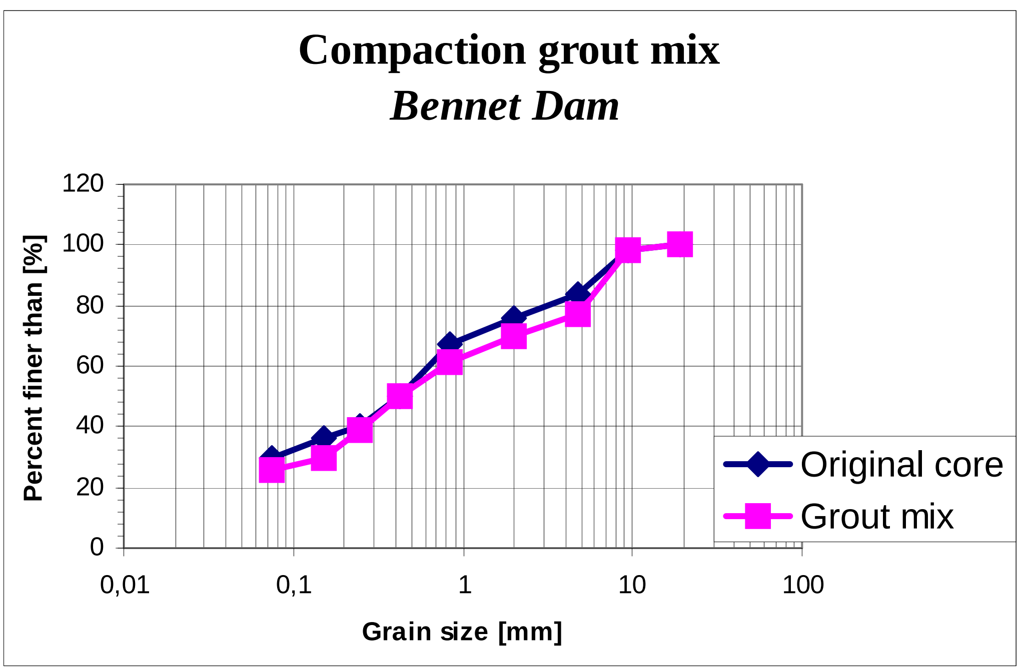


Figure 4-13. Particle size distribution curves of both the core and used grout at Bennet dam (Garner et al 2000)

4.4.2.3 Grout pressures

If grouting is performed in a core, caution must be taken in order to not put extensive damage on the core. This may however be controlled by the chosen method, grout pressures and an experienced operator.

In order to control the grout pressures as much as possible the diameter of the grout holes should be designed to give a loss of head by friction in pipes. This will reduce the pressure at the end of casing, thus offering a wider range of possibilities to control the pressures throughout the injection process.

For compaction grouting, two main characteristics should be considered:

- Rupture of the core soil due to high grout pressures;
- Extensive tensile forces put upon the soil.

Not only the passive earth pressure should be considered but also the extensive tensile forces, de facto put upon the soil in liaison with compaction grouting. When compaction grouting is performed, an expanding bulb of grout subjects the surrounding soil to tensile forces. Analysis of this sort is difficult. A numerical model should be made that treats different shear behaviour under a constant change of density since soil behaviour is density dependent. The change of density is explained by the compaction of the soil, thus lowering the void ratio in the soil. The model must be chosen according to the actual appearance of the soil at the site (Jefferies 1993).

5 Planning for Laboratory Work

This chapter will point out main characteristics that need to be considered when performing laboratory work. By combining knowledge from the literature study, *chapter2, chapter 3* and *chapter 4,* it will become easier decide characteristics that must be included and tested when designing the characteristics of a grout.

This chapter will furthermore describe how the experiments have been performed and how they have been evaluated.

5.1 Deciding experiments

In order to design the experiments, key data needs to be extracted from the literature study. The three chapters constituting the literature study implicates three important guidelines:

1. An embankment dam is a complex structure and all its components are highly dependent on each other. Therefore, the grout alone cannot be considered only. Experiments must be designed that take in count not only the grout, but the surrounding original core soil as well and how the injected grout affect the environment. Since all incorporated zones in a dam are dependant on each other, there should furthermore be a desire to design a grout to have similar geotechnical characteristics as the original core soil.
2. If similar characteristics between a grout and core soil is desired, the grout shouldn't contain any kind of hydrating material such as cement. The grout should contain only soil particles of various sizes and water. This will be referred to as a ***flexible grout***. If a flexible grout is used in a dam, it will be subjected to the same erosive forces as the original core soil.
3. If a flexible grout is used, its viscosity is of great importance. Low viscosity provides a lesser degree of penetration and a high viscosity a higher degree of penetration. Rupture of the core soil should always be considered as an unwanted scenario hence, grout used in an embankment should have a low viscosity.

The three previous stated points leads to the following decision:

1. Studies of a flexible grout and how it reacts to a constant rate of seepage must be done– ***Permeability testing***;
2. Studies of how the injected grout affects the surrounding original core soil when implemented inside a damaged part of the dam must be done – ***Filter box testing***.

Thus, Permeability testing and Filter box testing will be performed and their respective results will be presented in *chapter 6* and *chapter 7*. The two experimental series and their evaluated parameters will be described further in this chapter.

5.2 Permeability testing

If a flexible grout is used as an injection material inside an embankment dam, it will be subjected to the same erosive forces as the original dam. Therefore, a flexible grout must be tested in an environment where it can be surged. If a flexible grout is surged, it is subjected to erosive forces. A good way of surging a flexible grout can be done via permeability testing. The permeability testing allows any type of flexible grout mixture to be surged. After the flexible grout has been surged, it must be evaluated if soil particles within the grout structure have moved or not. If soil particles within the flexible grout move, the grout must be considered more prone to erosion. If particles soil particles within the flexible grout move its expected durability in situ will more likely diminish.

Designing the particle size distribution curve is a concept that origins from *chapter 2.4*. If a soil, or flexible grout is gap graded, it is less stable in terms of resistance to erosion. Hence, if the "right" particle size distribution curve is obtained, the flexible grout should become more stable. The movement of the soil particles constituting the flexible grout should be traceable by using sieve analysis. If a given particle size distribution curve of a soil is known before surge, it may be compared with its particle size distribution curve after surging. This comparison should tell weather particle movements have occurred or not. If the particle size distribution curve after surging differs from the original curve before surging, a rearrangement of the particles inside the soil should have occurred.

5.2.1 Goal of the permeability testing

The goal of the permeability testing should be to design a grout with a certain particle size distribution curve where the movements of the finer particles are minimized.

Permeability tests are primary used to describe the permeability of a given soil. In this test series, it will primary allow the opportunity of surging a flexible grout. The flexible grout will then be evaluated how the surging affected its original structure. When the water pass the grout it may suspend fines and transport it from one location to another inside the permeameter.

In order to choose a grout material that is constituted uniquely from soil material, an analysis regarding particle transport needs to be performed. If the presumptive grout for injection in a dam is not cement-bound, following factors should influence the durability of the grout in-situ:

- Grading of the grout;
- A low permeability of the grout.

Both of the factors mentioned above should be able to be tested via permeability tests. An evenly distributed particle size distribution curve should offer a greater resistance against internal erosion. A low permeability should diminish the risk for higher shear forces due to water flow to develop.

The aim of the permeability testing is to:

- Spot a diminish of the particle transport of fines by moderating the particle size distribution curve of the tested grout;
- Obtain a grout with a maximum permeability of $1\times10^{-6}\,{}^{m}/_{s}$, which in theory represent the lowest hydraulic conductivity where clay particles may erode (Ekström et al 2001).

The following segment will describe the basics of permeability testing and the tested parameters after each test run. It will also be explained why these parameters have been tested and evaluated.

5.2.2 Hypothesis for permeability testing

The hypothesis for the permeability testing is connected with the grading of the grout. Since a flexible grout is thought to resemble the original core, the same mechanisms damaging the core will also damage the injected grout.

The hypothesis is:

"Designing the particle size distribution curve of a grout will reduce the risk of internal erosion".

An evenly distributed particle distribution curve will enhance the materials own stability against internal erosion. If the grout is gap graded the grout is more easily subjected to internal erosion. A shortage of a specific fraction size within the grout sample may jeopardize the stability of the entire soil structure of the flexible grout.

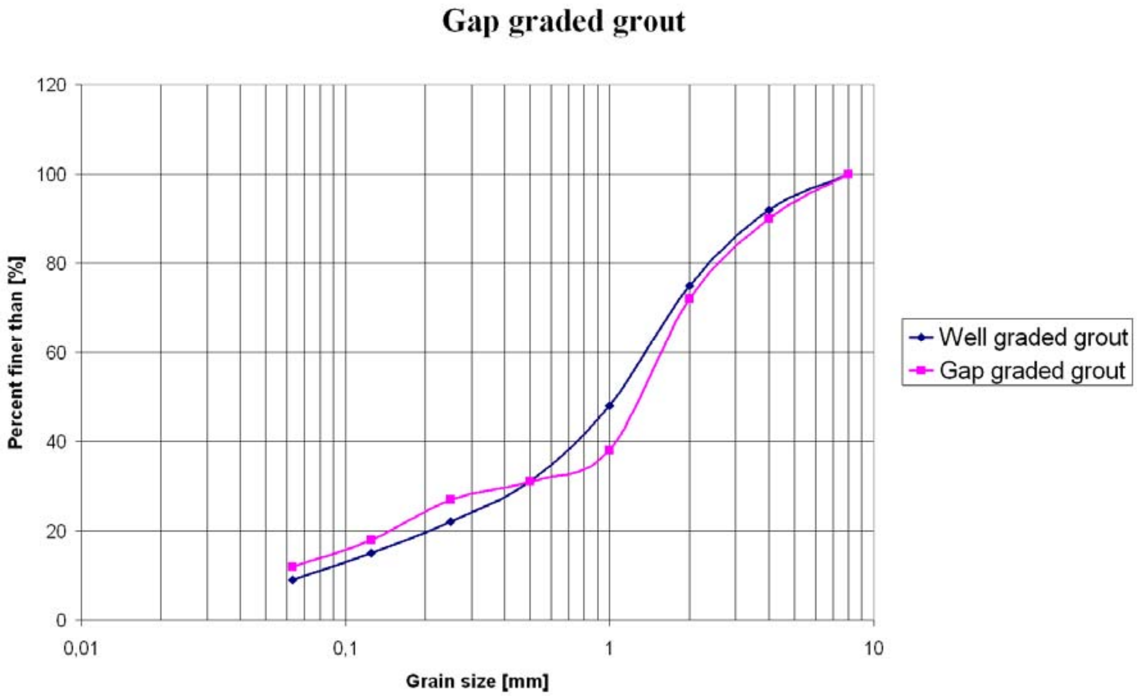


Figure 5-1. Gap graded grout vs. a well-graded grout.

There will be three test series in total. Since the flexible grout should have similar characteristics as an original core soil, data from a sieved core soil in an embankment dam will be used. This data will provide a first particle size distribution curve of a presumptive flexible grout. The second and third tested flexible grout will be a further development of the first tested flexible grout.

The three different tested flexible grouts (different particle size distribution curves), grout 1, grout 2 and grout 3 have particle size distributions that can be seen in *figure 5-2*.

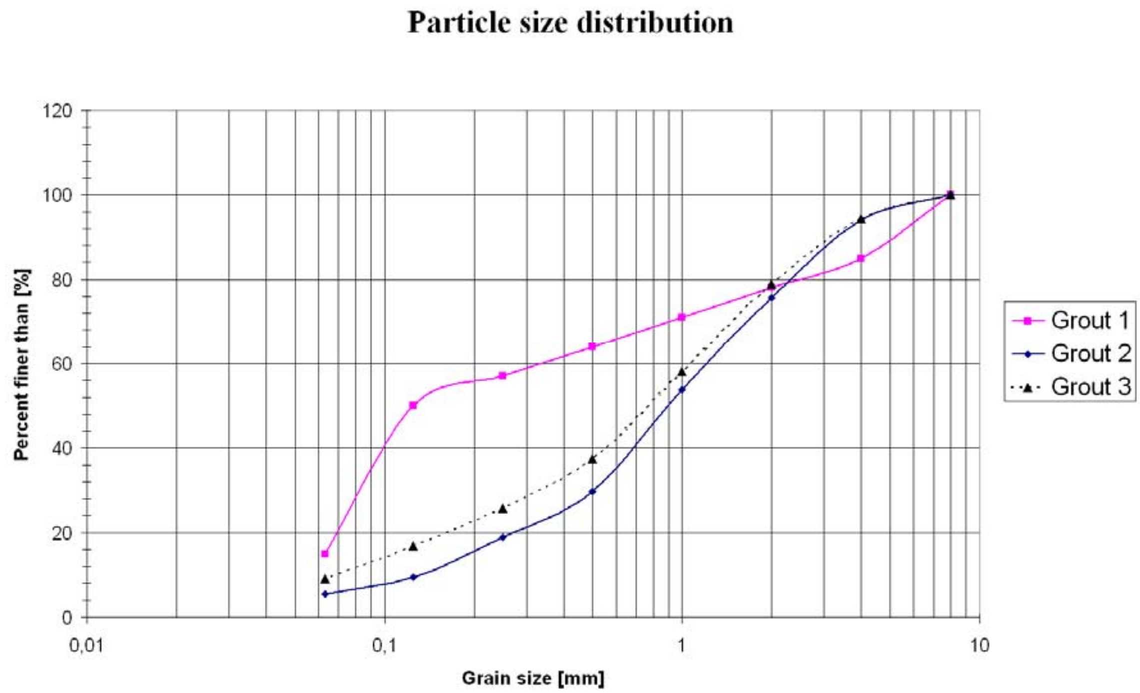


Figure 5-2. Particle distribution curves of the tested flexible grouts.

Grout 1 was designed by using collected data from the core soil at Grundsjöarna hydropower station. Only the d_{15}, d_{50} *and* d_{85} measurement were given. Beside the three values from Grundsjöarna, in order to create a particle size distribution curve, the other fraction sizes were considered evenly distributed. Therefore, the particle size distribution of grout 1 does not represent the actual situation at Grundsjöarna. It represents only an estimation of the real situation

Included soil particles of the flexible grout have various particle sizes and fall in the range of 8 mm to clay.

Grout 2 and finally grout 3 was a further development of grout 1.

5.2.3 Preparation and dismantling of permeability testing

The most important action when preparing the samples is to do it with the same procedure. If not, it is hard to obtain a good comparison between the different tests. The procedure has been the following:

1. Grain sizes with measurements 8-4mm, 4-2mm, 2-1mm, 1-0,5mm, 0,5-0,25mm, 0,25-0,125mm, 0,125-0,063mm and dry clay is dry mixed in a bowl according to desired proportions;
2. A desired amount of water is added. The mixture is blended in a blender for two minutes. After this, additional mixing by hand is performed;
3. The inside of the permeameter is coated with a thin layer of grease;

4. The mixture is poured into the permeameter and sealed with a lid. The top and bottom of the grout sample is protected by filter stones;
5. Additional water is added to fully saturate the sample;
6. The permeameter is mounted and connected with water hoses;
7. The top and bottom valves are opened, water is now surging the sample. A water tank with level control gives the water pressure.

The samples will be left in the permeameter for one – two weeks if surged. Two hours if un-surged. If no surging is desired, only step 1 – 5 is performed during the preparation of the test.

After each test run, the tested soil should also be divided into smaller sections, or slices. This will allow a more detailed view, slice by slice, of how the surge affects the tested flexible grout.

When testing is finished, the permeameter must be dismantled. This is done by the following procedure:

1. The top and bottom valves are closed and the water hoses are removed;
2. The lids of the permeameter and the top filter stone are removed;
3. The sample is carefully pushed out of the permeameter tube at about 20mm each time. The bottom filter stone act as a pressure distributor on the sample;
4. The 20mm of grout is cut of from the original sample and placed into a marked aluminium tin. This is repeated until the permeameter tube is empty;
5. Each slice is weighed and put in an oven to dry for 24h at a temperature of 105°C.

If no surge has been performed, only step 2 – 5 is performed. The work procedure for dismantling can be seen in *figure 5-3*.

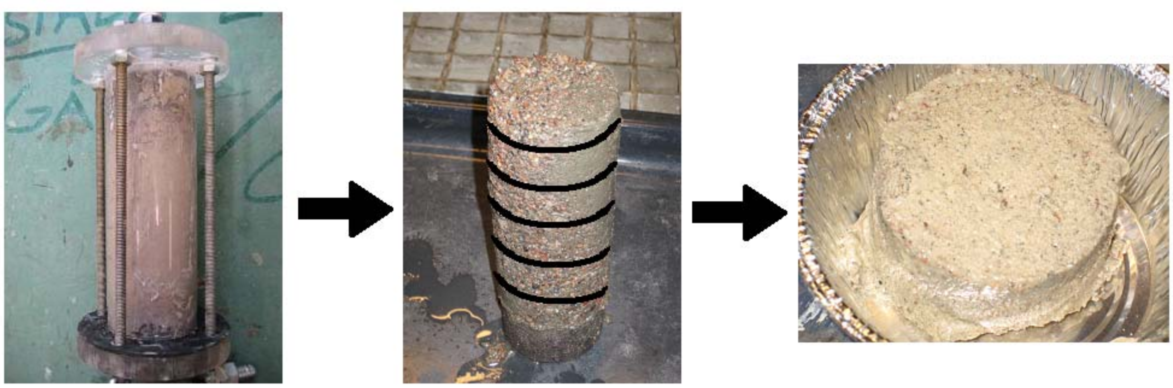


Figure 5-3. Procedure of making slices out of a tested grout.

The total duration of surge for each test through out the permeability testing is one to two weeks for all tests.

5.2.4 Evaluations of the flexible grout after permeability testing

If surging different types of flexible grout via permeability testing, extra evaluations beside the permeability must be done. Measurements and evaluations of the following parameters will be done:

- Permeability
- Water content;
- Compaction, porosity and void ratio;
- Migration of fines;
- Sieve analysis.

An analysis of the obtained values from sieving will also be done in form of histograms and a numerical evaluation of the change in distribution of the different particle size distribution curves.

5.2.4.1 Permeability of the flexible grout

The testing will follow the rules according to Swedish standard SIS 02 71 11 for permeability testing. The type of permeability testing used is the constant-head permeameter. A water tank equipped with a level control guarantees a constant head of water. The water inflow to the tank must be kept slightly higher than the outflow, thus remaining a constant head.

The flexible grout is put in a permeameter. The permeameter is shaped as a cylinder and made from Plexiglas. The permeameter is connected with water hoses on each side and placed vertically. Valves are opened and the water is allowed to surge the permeameter, thus exposing the flexible grout to seepage.

The setup of permeability testing is demonstrated in *figure 5-4*

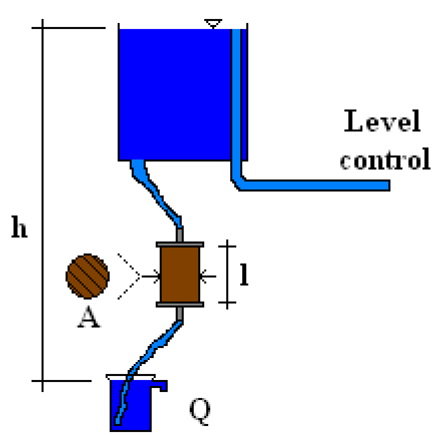


Figure 5-4. Set up of the permeability testing.

The rate of flow through the grout is proportional to the hydraulic gradient, thus Darcy's law may be used:

$$Q = k\left(\frac{\Delta h}{L}\right)A \tag{6}$$

(**6**) may also be expressed as:

$$\frac{q}{A} = \frac{k \times h}{l} \tag{7}$$

or

$$\frac{dq}{dt} = \overline{Q} \Rightarrow k = \frac{\overline{Q} \times l}{A \times t \times h} \tag{8}$$

where in **(6)**, **(7)** and **(8)**:

$$\begin{cases} q = rate\ of\ \ water\ \left[{m^3}/{s}\right] \\ A = area\ of\ \ sample\ \left[m^2\right] \\ k = coefficient\ of\ \ permeability\ \left[{m}/{s}\right] \\ h = hydraulic\ head\ difference\ \left[m\right] \\ l = length\ of\ \ sample\ \left(permeameter\right)\left[m\right] \\ \overline{Q}\ = volume\ of\ \ water\ during\ time\ \left[m^3\right] \\ t = time\ \left[s\right] \end{cases}$$

The procedure of extracting the permeability, *k*, of a tested grout is given by formula **(8)**. The permeability is the only parameter that will be measured during each test run. The permeability may tell indirect if particle transport occurs. If the permeability rise during the test, fines of the grout may be washed out. If the permeability lowers, a concentration of fines may create a low permeable stratum within the tested grout. Both cases refer to particle movements within the tested flexible grout.

The permeability is measured for the whole tested grout, not slice by slice. The water will be given a pressure of 3,27m (Δh).

The hydraulic gradient will be calculated by using formula **(9)**

$$i = \frac{\Delta h}{\Delta l} \tag{9}$$

5.2.4.2 Water content of the flexible grout

The testing will follow the rules according to Swedish standard SIS 02 71 16. Water content is measured in order to find the amount of water in the grout. This is necessary to do if the total density of the mixture grout/water is to be found. It may also be possible to spot if there are stratums with a concentration or a loss of fines within the sample, plus where these stratums are located.

Each slice of the dismantled flexible grout sample is placed in an aluminium tin and placed in an oven. The oven is set to 105°C and the sample is dried during 24 hours. The sample is weighed before and after. The water content is given by the formula:

$$w = \frac{W_{wet\ soil} - W_{dry\ soil}}{W_{dry\ soil}} \Rightarrow \frac{W_w}{W_s} \times 100 \tag{10}$$

The water content is expressed as a percentage and falls in the range of

$$0 < w < \infty$$

5.2.4.3 Degree of compaction, porosity and void ratio of the flexible grout

Measurements of the density are important since the permeability of a flexible grout depends on its total density and void ratio. A higher degree of compaction (total density) of the flexible grout should ensure that it remains at its injected location inside the dam. If a higher compaction of the flexible grout is reached, the void ratio and the permeability will also lower. A low void ratio and permeability in turn offers a grout less sensitive to internal erosion (Shuttle et al, *Prediction and validation of compaction grout effectiveness*)

To obtain the total density of the flexible grout, tested in the permeameter, all the included volumes and weights must be known (voids, water and solids). To do so, a matrix as seen in *table 5-1* can be used.

	Volume $[m^3]$	Density $[t/m^3]$	Weight $[t]$
Air	V_a	0	0
Water	$\dfrac{W_s \times w}{\rho_{H_2O}}$	ρ_{H_2O}	$W_s \times w$
Solids	$\dfrac{W_s}{\rho_s}$	ρ_s	W_s
Total	V	ρ	W

Table 5-1. Matrix in order to facilitate calculations of total density, void ratio and porosity of the flexible grout.

In order to calculate the density of the flexible grout inside the permeameter, the tested grout must be assumed being saturated. This is in reality seldom the case (Cernica 1995) but the degree of saturation after a permeability test, lasting one to two weeks, should be close to 100%, hence the total volume of the grout may be expressed as:

$$V_a = 0 \Rightarrow V_v = V_w \Rightarrow V_{tot} = V_s + V_w \tag{11}$$

The ***density*** of the sample may then be expressed as:

$$\rho = \frac{(W_s \times w) + W_s}{\left(\dfrac{W_s \times w}{\rho_{H_2O}}\right) + \left(\dfrac{W_s}{\rho_s}\right)} \Rightarrow \quad \rho = \frac{W}{V} \tag{12}$$

The density is expressed as t/m^3 and, in this case, falls in range of

$$\rho_{H_2O} < \rho < \rho_s$$

The ***porosity*** of the sample, *n*, equals

$$n = \frac{V_v}{V} \Rightarrow \frac{V_w}{V} \times 100 \tag{13}$$

Porosity is expressed as a percentage and falls in range of

$$0 < n < 100$$

The ***void ratio***, *e*, is described by the formula:

$$e = \frac{V_v}{V_s} \tag{14}$$

Void ratio is expressed as a number and falls in the range of

$$0 < e < \infty$$

Void ratios are also mentioned in *chapter 4.3.3.2*. They should be kept at a low level since there is a strong connection between void ratios and internal erosion, see *figure 4-7*.

5.2.4.4 Migration of fines from the flexible grout

The lower small water container, equipped with a level control, will also function as a trap for fines. If fines are washed out from the permeameter during testing they will be sedimented and trapped in the container. After the testing procedure the water will be poured into a coffee filter, dried and weighed.

5.2.4.5 Sieve analysis of the flexible grout

The sieve analysis will be performed on each slice after the permeability testing. The sieve used is placed in the facilities of Vattenfall Research and Development at the Älvkarleby laboratory and is constituted out of seven different sieves creating a column.

Performing sieve analysis on each slice offers a way to spot particle transportation within the grout sample. By comparing the different particle size distribution curves from the same sample, variations between the curves can be found. These variations are thought to show movement of material within a grout sample after surging. The influence of surging should also be viewable if comparing particle size distribution curves before and after a permeability test.

Figure 5-5. Tap sieve in Älvkarleby ballast laboratory.

Tap sieving is performed and generated by using an electro-magnetic drive. The measurements of the different meshes are, from top to bottom:

- 4mm;
- 2mm;
- 1mm;
- 0,5mm;
- 0,25mm;
- 0,125mm;
- 0,063mm.

The dried sample of flexible grout (slice) is placed on the top mesh. Since dried flexible grout contains clay, it will be more or less homogenous after drying hence it must be separated into smaller pieces. This will be done by hand over the top mesh. Each slice will undergo a total sieve time of 10 minutes. After 10 minutes each slice should be entirely broken down into its original fractions. Finally, the sieve column will be dismantled and the particles trapped in each mesh will be weighed and put into a log diagram. By doing so, particle size distribution curves for each slice will be obtained. This procedure will be done for every slice of flexible grout throughout the test series.

5.2.4.6 Usage of the parameters from the sieve analysis

The obtained parameters will be used to create numerical analyses. The numerical analysis may give a different view of the results, obtained by the sieve analysis. The performed numerical analyses are:

- Histograms;
- Numerical changes of the particle size distribution curve.

5.2.4.6.1 Histogram

Histograms offer a good way to spot how often a certain outcome is obtained. If the sieve analysis is performed on each slice throughout a test run of one type of flexible grout, there will be a certain degree of deviations between the results. The caught material in each mesh (4mm, 2mm, 1mm, 0,5mm, 0,25mm, 0,125mm and 0,063mm) has a certain weight that forms the basis of the particle size distribution curve. By knowing this specific weight, it is possible to tell how much of the total sieved weight that have passed a certain mesh. For example, if the total weight of a sieved grout is 100g and 10g is caught in the top mesh, 90% of the sieved grout has passed the top mesh. When performing a full test series on one type of flexible grout, these values differ. They change between each test. If there is a large difference between the values, the tested grout should be considered more prone to internal erosion. If there is an enlargement of the deviations before and after surging, it can be spotted via the histogram analysis. The desired scenario is that a histogram of a surged flexible grout is more or less similar to the histogram, obtained before the surge.

5.2.4.6.2 Numerical changes of the particle size distribution curve

This analysis is based on the same principal as the construction of histograms. By doing a numerical analysis of the particle size distribution curve it is easier to spot the actual variation (movement) of particles within the tested grout. The numerical analysis treats the maximal variations of the outcome in each mesh before and after surging. It will treat the deviations in means of both standard deviations and the de facto maximum and minimum deviations. The difference in variations should be minimized in order to obtain a stable grout structure.

5.3 Filter box testing

If a grout is internally stable, it must be examined how it cooperates with its surrounding core soil if grouted inside an embankment dam. Excessive damaging of the core if grouting should in all cases be avoided. This can be examined if performing experiments with a grouted zone inside a core soil. The filter should also be included into the model since this is a sensitive location inside a dam. For this purpose a small-scale model of an embankment dam must be constructed. The model should preferably be small in order to allow more than a single experiment to be performed. Since more than one experiment needs to be done, the model must also guarantee that similar conditions exists at the very start of each test. The solution to this is a ***filter box***. The filter box contains both an artificial filter and core. The box can be surged and dismantled without destroying the tested grout.

The grouted zone should be put in place manually before the actual test begins. By doing so, it is possible to test a perfectly grouted void. This should give the best possible view of how a type of grout affects the surrounding core soil without considering the actual injection process.

Grouting will be done with both hardening- and flexible grouts. The hardening grout should resemble of grouts used in actual remedial work. The flexible grout should be chosen according to the outcome of the first laboratory test, permeability testing.

5.3.1 Goal of the filter box testing

The goal of the filter box testing is to visualize problems related to the usage of hardening grouts inside a damaged core.

5.3.2 Hypothesis of the filter box testing

The hypothesis for the filter box is connected with how an injected grout affects the core soil. If any type of grout is injected in an embankment dam it will affect the original soil. Therefore, the hypothesis used is:

"Flexible grouts are better suited than hardening grouts if used in an embankment dam"

In order to get a clearer view of how the original core soil behaves if injected with a plug of hardening material or a flexible grout, it is evident to design and construct a test uniquely for this event. The filter box relates to the fact that both the core soil and filter is modelled. A central part of the core soil is replaced by a sand/cement- or a flexible grout. A water flow is then applied on the core soil that surge the box.

The filter box must be prepared and dismantled the same way when testing the two types of grout.

5.3.3 Used grouts

The testing is done with two different types of grout:

1. Grouting with hardening grout;
2. Grouting with flexible grout.

5.3.3.1 Hardening grout

Hardening grouts are used as a remedial method for reparation injection in embankment dams. The type of hardening grout used in the filter box testing is a sand/cement grout with a weight ratio of $^{50}\!/_{50}$. The w/c ratio of the grout is 0,6. The grout is been put in place at the same time as the core soil and at a fresh state (see *chapter 5.3.6*). The surging of the filter box starts as soon as the preparation is done. The time from application of the hardening grout until surge is 10 minutes.

Figure 5-6. Preparation of a filter box test with hardening grout. The hardening grout plug can be seen as a brighter area in the center of the aluminium tube.

Figure 5-6 shows the appearance of the upstream side of the aluminium tube just before the cone head is fixed. The centre of tube is brighter because of the content of cement in the grout. The grout has at this point a higher viscosity than the surrounding core soil. This is due to the large difference in water content between the grout and the core soil.

The durations for the filter box tests performed with hardening grouts can be found in *table 5-2*

Test number	Duration (days)
4	5
6	7

Table 5-2. Duration of tests performed with hardening grouts. The two filter box tests with hardening grouts are named test 4 and test 6.

5.3.3.2 Flexible grout

Flexible are also used as a remedial method for damaged embankment dams. The term flexible relates to the fact that the grout does not undergo any hydrating processes as the case for cement bound grouts. The flexible grout is designed to resemble of the original core soil. This will allow the grout to unify with the original core soil and restore the damaged part of the dam into its original state. To do so may diminish the risk of damaging the original core soil.

The flexible grout used is grout 3, which has been tested in *chapter 6.1.3*. The particle size distribution of grout 3 and the core soil used for the filter box tests can be found in *figure 5-7* below:

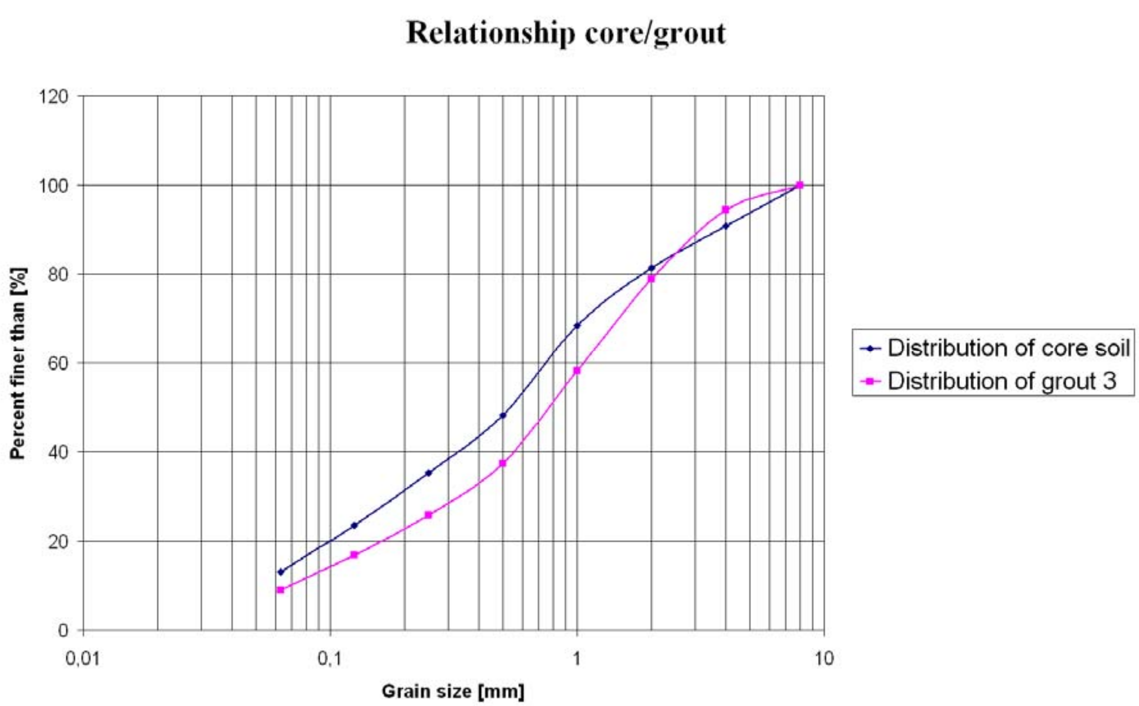


Figure 5-7. Relationship of particle size distribution curves between core soil and flexible grout.

The durations for the filter box tests performed with hardening grouts can be found in *table 5-3*:

Test number	Duration (days)
5	7
7	7

Table 5-3. Duration of tests performed with hardening grouts. The two filter box tests with flexible grouts are named test 5 and test 7.

5.3.4 Design of the filter box

The design of the filter box is simple and constituted out of only four parts. A box made from Plexiglas, a tube made of aluminium, a retaining wall and a cone head of aluminium.

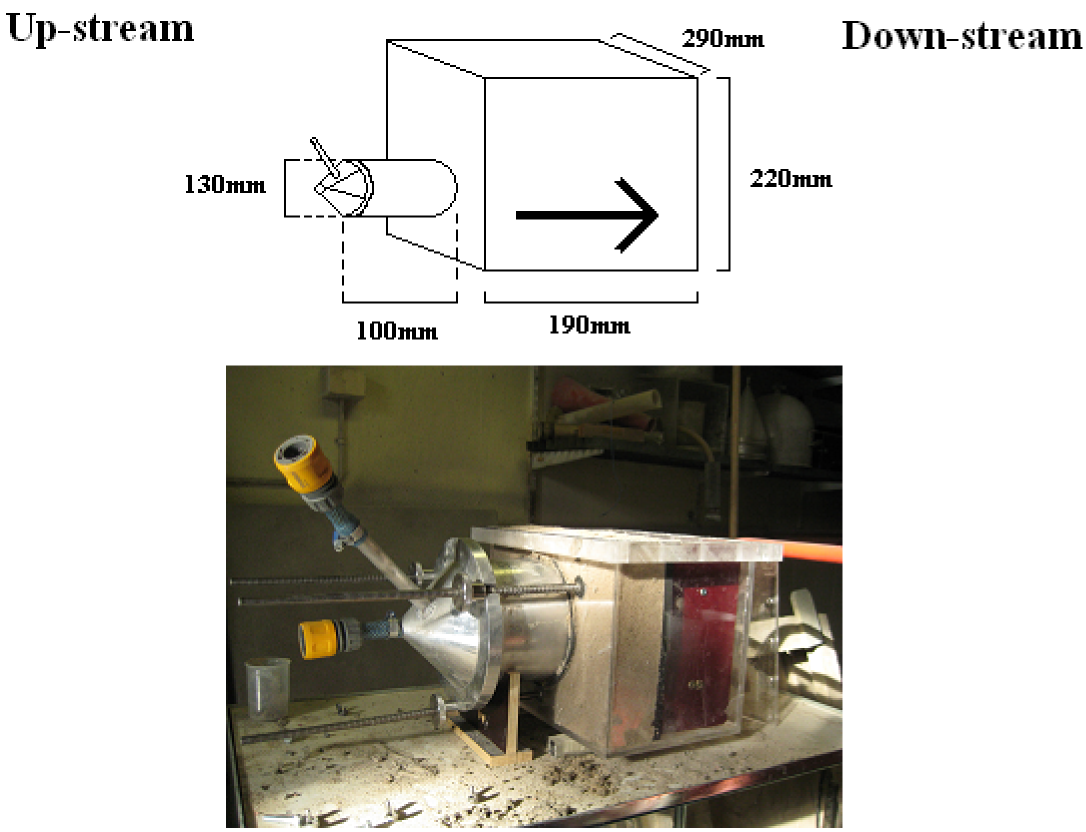


Figure 5-8. Setup and design of the filter box.

The box, filled with sand acts as a filter and the aluminium tube, filled with core soil, acts as a core. When surged, the water enters the core soil via a special aluminium cone equipped with a mesh. The seepage then escapes through the filter and emits via the bottom of the box.

Four main parts constitute the filter box (see *figure 5-9*).

1. **The box**. The box is made from Plexiglas with dimensions of 190x220x290mm. It is reinforced at the top so that a lid may be put in placed and secured with small bolts. All the walls have a thickness of 5mm. At the bottom of the box there is a small squared escape hole where the seepage is allowed to emit. The hole is covered with a mesh with a width of 0,063mm. During testing, an aluminium tin is placed under the hole and the amount of seepage is measured.
2. **A small retaining wall**. This wall raises the effective stress of the filter sand. By doing so, the core soil inside the aluminium tube also gains effective stress. It is made from plywood and has a dimension of 220x290mm. Many larger holes have been cut out from the plywood. These holes have then been covered with a mesh with a width of 0,063mm. The hole ensures the drainage of the filter. The retaining wall is placed inside the box.
3. **An aluminium tube**. The aluminium tube has a length of 100mm and a diameter of 120mm. The thickness of the tube is 3mm. The tube is connected to the box and the cone shaped head of aluminium.
4. **A cone shaped head of aluminium**. The cone is constructed to distribute the incoming water over an area equivalent to the aluminium tube. On one side there is an inlet for water. The inside of the cone is filled with medium coarse gravel (Ø ≈ 10mm). At the opposite end of the water inlet a mesh with a width of 0,063 mm is placed. The mesh keeps the gravel in place, inside the cone. The cone is connected to the aluminium tube and a water hose. The same water tank with level control as used for the permeability testing is used.

The surge through the filter box acts horizontally. Throughout the permeability testing all experiments were performed vertically but the filter box was designed to represent a horizontal scenario.

The blueprint of the filter box can be seen in *appendices*.

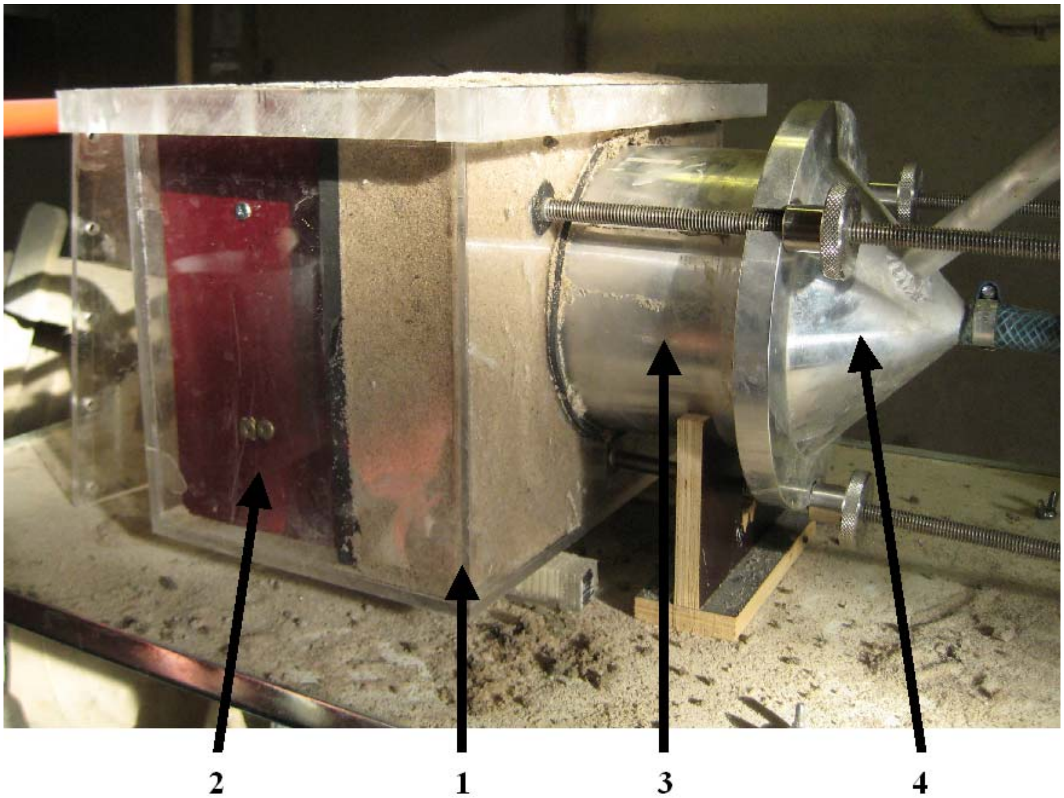


Figure 5-9. The filter box and its four main parts: (1) The box, (2) Retaining wall, (3) Aluminium tube and (4) Cone head.

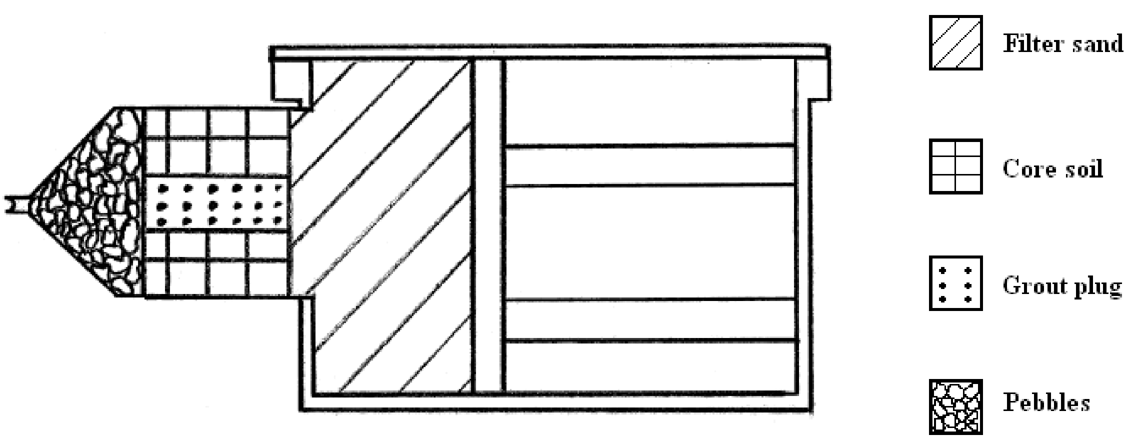


Figure 5-10. See through view of the filter box along its center axis. The water inlet is to the left. The boundary between the pebbles and core soil/grout plug is protected by a fine mesh (c/c = 0,063mm).

5.3.5 Relationship filter/core

The filter has been very fine graded so that no particle transport between the core soil and filter has occurred. However, facing a piping damage in a real dam automatically includes a defect filter, but this experiment will only describe how the two types of grout affect the core soil. A further discussion about defect filters is proposed in *chapter 8.2*.

In order to decide the correct filter criteria it must first be examined whether the core soil is narrowly graded or widely graded. This is done by extracting C_u, which describes the steepness of the particle distribution curve. If $C_u > 3$ the core soil is widely graded. C_u is given in formula **(15)**:

$$C_u = \frac{d_{60}}{d_{10}} \tag{15}$$

Using **(15)** with a d_{60} = 0,72mm and d_{10} = 0,05mm (from *figure 5-11*) gives a C_u = 14,4, which is larger than 3. The used core soil in the filter box tests is widely graded. Since the degree of fines in the core soil is less than 30% the following filter rules are used:

$$4 < {}^{D_{15}}\!/_{d_{15}} < 40 \tag{16}$$

Where in **(16)**:

$$D_{15} = 0{,}3mm, \quad d_{15} = 0{,}06mm$$

Thus:

$$ {}^{D_{15}}\!/_{d_{15}} \Rightarrow \frac{0{,}3}{0{,}06} = 5 > 4 \quad OK$$

D_{15} should be less than 0,7mm since the core soil is widely graded (Sherard et al 1989), thus:

$$D_{15} \leq 0{,}7mm \tag{17}$$

And according to **(17)**:

$$D_{15} \leq 0{,}7mm \Rightarrow D_{15} = 0{,}3mm \quad OK$$

Filter rules have been fulfilled. The relationship filter/core can be seen in *figure 5-11*:

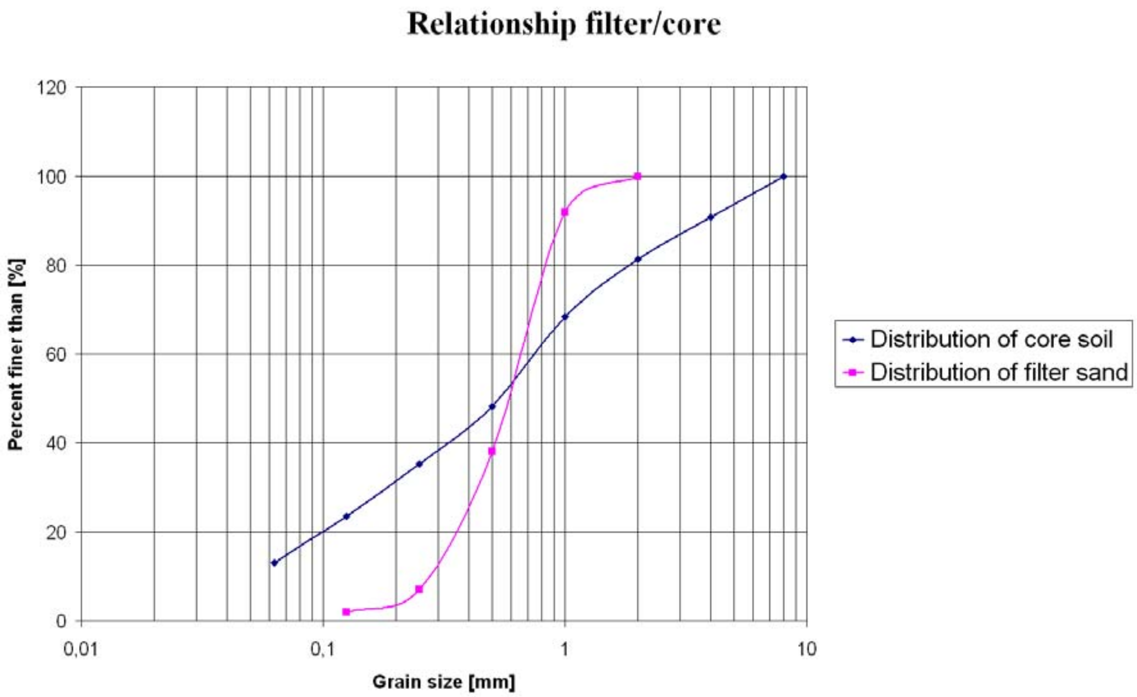


Figure 5-11. Relationship between filter and core soil.

The material used for the filter is ordinary silver sand. The water content in the core soil at start is around 10 % and 3% in the filter sand.

The hydraulic gradient throughout the testing with the filter box is given by formula **(9)**:

$$i = \frac{\Delta h}{\Delta l}$$

5.3.6 Preparation and dismantling of filter box

On the right side of *figure 5-9* a hose is connected. This hose is connected to the same tank used for the permeability testing. At the bottom of the box there is a hole, covered with a mesh (0,063mm) where the water is allowed to escape. The water enters the core soil on the up-stream side, surge the core soil and filter and escape at the very bottom of the box.. To obtain a better-compacted filter the whole box is not used. In stead a wooden mesh is placed to cut off the main part of the total volume of the box.

The preparation for the testing is done in the following steps:

1. The wooden meshed wall is placed into the box and the lid is fixed with bolts;
2. The box is placed with the circular hole upwards;
3. Filter sand is poured into the circular hole and distributed over the volume of the box/mesh. It is also compacted;

4. The aluminium tubes inside is coated with grease and put in place;
5. In the centre of the aluminium tube a smaller plastic empty tube (∅ = 45mm);
6. Core soil is poured and compacted in sections inside the aluminium tube all the way up to the rim. Note that the plastic tube is not filled;
7. The plastic tube is carefully removed;
8. A sand/cement grout (50/50 mixture, W/C = 0,6) or grout 3 (from permeability testing, *chapter 6*) is placed and compacted inside the hole created by the plastic tube;
9. More core soil is applied on the top. The extra core soil is not placed on top of the sand/cement plug;
10. Further compaction of the soil inside the aluminium tube;
11. The cone head is put in place and fixed with bolts;
12. The box is then rotated and placed horizontally;
13. The lid of the box is removed and extra filter sand is added, which is thoroughly compacted;
14. The lid is once more put in place and fixated with bolts;
15. A water hose from a drop tank is connected and water pressure is applied.

The water content of the filter and core soil before testing procedure is inaugurated is 0% and 13% respectively.

After each test run, the following procedure is performed in order to dismantle the filter box:

1. The water hose is removed;
2. The lid of the box is removed and samples of the filter sand is taken out at three locations as the filter sand is excavated;
3. The bolts, securing the cone head is removed;
4. The aluminium tube is removed together with the cone head and put horizontally in a support;
5. The downstream side (facing the filter) is carefully removed of its remaining filter sand;
6. Photos are taken as the filter sand is removed with a fine brush;
7. Undrained shear strength is measured on the downstream side;
8. When the core soil/grout is fully visible, the cone head is removed and the upstream side is visible. Photos are taken and the undrained shear strength is measured;
9. The core soil is taken out of the aluminium tube and samples for water content is removed at various points.

The core soil used for each new test will not be changed. It will be kept wet with a water content of 13% between each test. This will assure that the clay in the original core soil doesn’t swell. If a grouting procedure is performed inside a real dam, the original core soil has most likely attained its final swelling caused by the clay. Swelling of the modelled core soil may prevent damages from being detected after performing a filter box test run. By keeping the modelled core soil wet between each test, a more real grout scenario should be achieved.

5.3.7 Evaluations of the performed filter box test

In order to perform an evaluation of the filter box tests, the following parameters will be monitored

- Seepage measurements;
- Water content;
- Uniaxial compressive strength tests (hardening grouts only);
- Undrained shear strength;
- Photographs.

5.3.7.1 Seepage measurements of the filter box

Water has been collected from a square hole under the filter box during a time of 10 minutes. Seepage measurements were done every 24h:s except during weekends. After water enter the cone head it surge the core soil, filter sand and emits through bottom of the box. At this location an aluminium tin is placed and the amount of water is weighed. The water is given a pressure of 2,77m (Δh).

The permeability can't be found since the filter box won't be entirely saturated. The rate of seepage through the filter box will however offer a similar view of how the permeability change through time as obtained during permeability testing, *chapter 6*.

The hydraulic gradient will be calculated by using formula **(9)** and becomes:

$$i = \frac{2{,}77}{0{,}165} = 16{,}8$$

A hydraulic gradient of 17 is about ten times higher than an expected value inside an embankment dam. This high value will speed up natural damaging processes in the filter box thus, shorter test duration will be needed.

5.3.7.2 Water contents in the filter box

After each test procedure the filter box is broken down into its original pieces. Filter sand and core soil is collected from various point and weighed before and after drying. The formula for calculating the water content is given in *chapter 5.2.4.2* with formula **(10)**.

5.3.7.3 Uniaxial compressive strength of hardening grout

Uniaxial compressive strength will be measured for hardening grouts only. The tests where performed in Älvkarleby. By its basic definition, compressive strength is given by the formula

$$\sigma = \frac{F}{A} \tag{18}$$

Where in **(18)**:

$$\begin{cases} F = \quad Applied\ load \quad [N] \\ A = \quad Area \quad [m^2] \end{cases}$$

The sand/cement plug is put under a load, with a rate of $2{,}5\, kN/s$ and the test is aborted when the plug ruptures. At this point of rupture, *F* is given.

5.3.7.4 Shear strength difference of core soil and flexible grout

The undrained shear strength, τ_{fu}, is examined by using the cone penetration test. The method is simple and allows many tests to be performed without being forced to remove any soil from the aluminium tube. The undrained shear force is given by the formula:

$$\tau_{fu} = k \times g \times \frac{m}{i^2}\left(1 + \frac{a}{i}\right) \tag{19}$$

Where in **(19)**:

$$\begin{cases} \tau_{fu} = \quad Undrained\ shear\ strength \quad [kPa] \\ k = \quad Shape\ factor\ of\ the\ cone \quad [-] \\ g = \quad Gravitational\ pull \quad [N] \\ m = \quad Mass\ of\ cone \quad [g] \\ i = \quad Cone\ penetration \quad [mm] \\ a = \quad Height\ difference\ between\ cone\ tip\ and\ soil\ surface \quad [mm] \end{cases}$$

Since the height difference, *a*, is zero, g = 10 and the cone angle = 30º thus giving k = 1,0, **(19)** can be simplified into:

$$\tau_{fu} = \frac{10m}{i^2} \tag{20}$$

Two different cones have been used. The $100g/30°$ and $400g/30°$ cone.
Since the water content is below 45% there is no need to reduce the shear strength.

The cone penetration test is however created to test the undrained shear strength of clays but will in this case act as a creator for an index. Using the same method on both the core soil and the flexible grout, this method can be useful since it will tell the difference of shear strength between them. If this index is around one, the strengths of the core soil and flexible grout are similar. The index will be calculated as:

$$I = \frac{\tau_{fu,\,grout}}{\tau_{fu,\,core}} \tag{21}$$

If a resemblance between the core soil and grout is wanted, this method may be useful to spot this resemblance. The index from **(21)** will not be used in the same way when comparing the difference in shear strength between the core soil and hardening grout. For this case the dividend in **(21)** will be the uniaxial compressive strength for the hardening grout plug. The denominator will be chosen according to a normal value for this type of soil. Regarding the particle size distribution of the artificial core soil it resembles of sandy till. Normal values of shear strength for this soil is 50 – 100kPa (Stål et al 1984).

5.3.7.5 Photographs

Photos are taken on both the upstream- and downstream side of the core soil. Only the aluminium tube is of interest since this is the location of the grout/core. The filter box tests are designed to evaluate how a grout may affect the surrounding core soil. Taking photographs offers a good way to spot presumptive damaging of the core soil caused by the usage of different types of grout.

6 Results From the Permeability Testing

6.1 Introduction

This chapter demonstrates the results from permeability testing of grout 1, grout 2 and grout 3.

Throughout this chapter different levels are mentioned. The levels 135mm, 115mm, 95mm, 75mm, 55mm and 35mm represent the orientation of each slice within the permeameter, see *figure 6-1*. It is essential to remember what these values represent to fully understand this chapter.

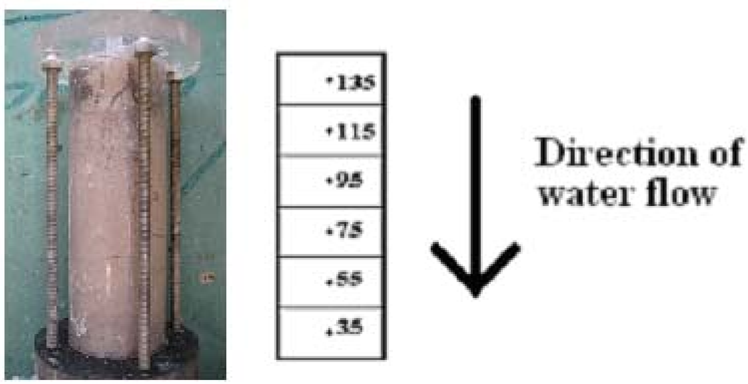


Figure 6-1. How the tested grout is dissected. Six slices are made from each test. Their orientation are described by their height inside the permeameter.

6.1.1 First series, grout 1

Grout 1 was quickly abandoned due to difficulties with handling and high water content. When undergoing permeability testing the sample did not let through any water during a test time of 48 hours. Because of this the testing of this specific grout was abandoned. Even though failing to perform a proper permeability testing some values could be found such as water content, total density, void ratio and porosity.

A description of grout 1 is found in *table 6-1* and *figure 6-2*.

	Mesh [mm]	Weight [g]	% finer than
Grout 1	8		100
	4	120	85
	2	56	78
	1	56	71
	0,5	56	64
	0,25	56	57
	0,125	56	50
	0,063	280	15
	>0,063	120	0
Total		800	

Table 6-1. Grain sizes and weight ratios constituting grout 1.

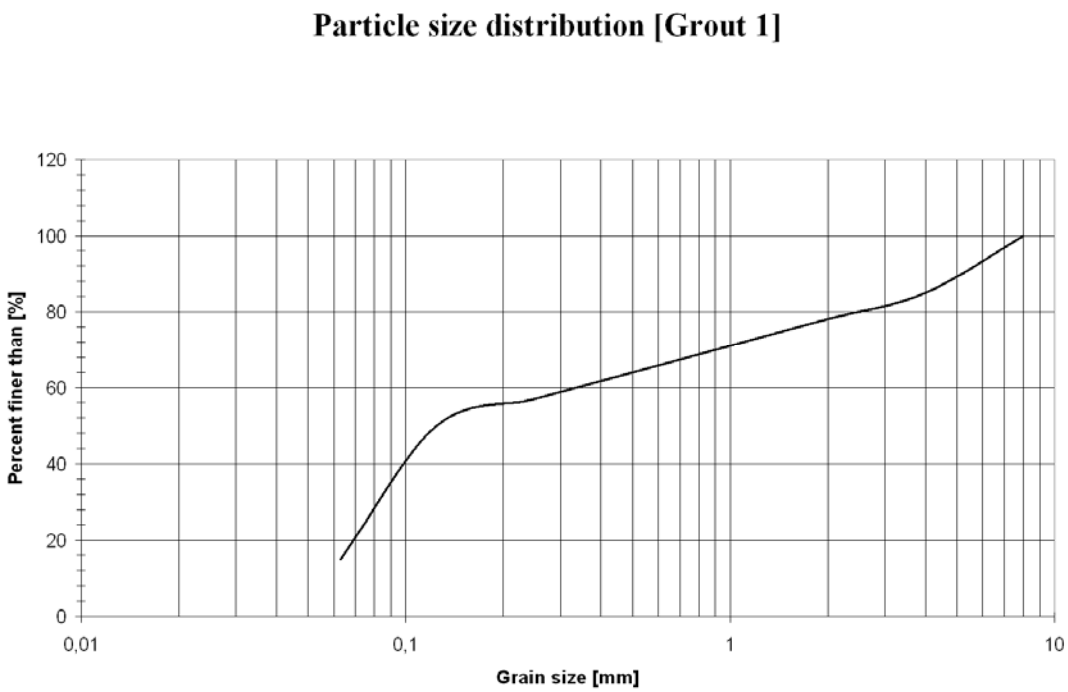


Figure 6-2. Particle size distribution curve of the first tested grout, grout 1.

The particle size distribution curve of grout 1 is gap graded. d_{15}, d_{50} and d_{85} are known from earlier sieve analysis. Values beside d_{15}, d_{50} and d_{85} are assumed evenly distributed, see *table 6-1*.

6.1.1.1 Water content

The water content for grout 1 after the test is shown in *figure 6-3*.

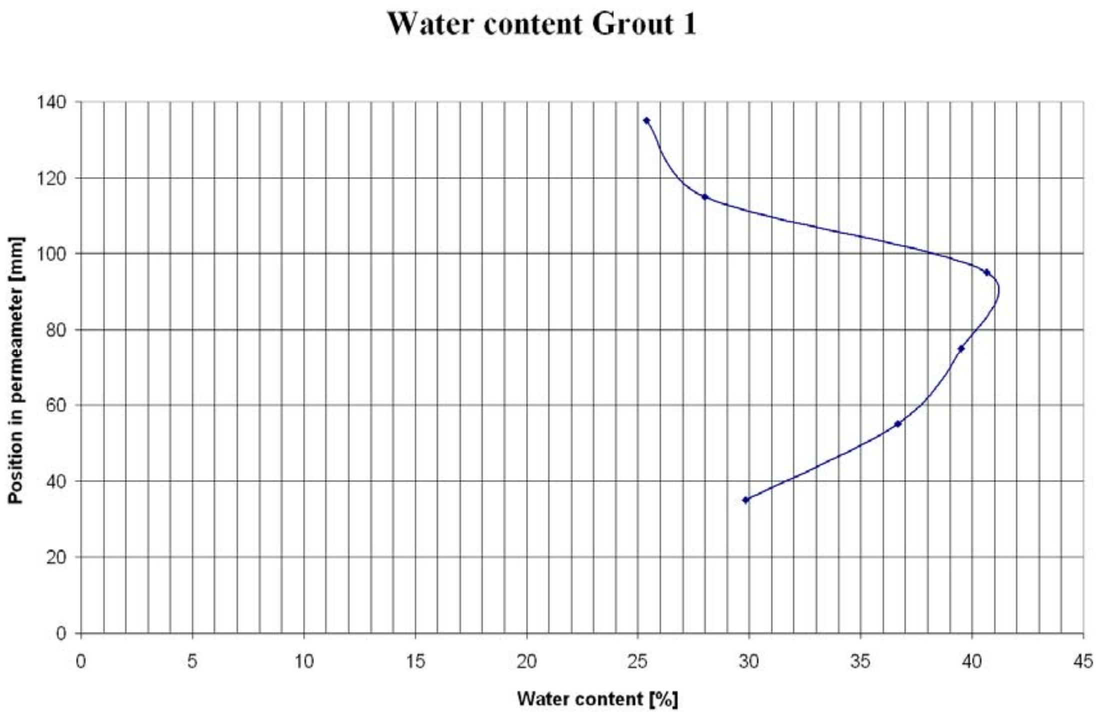


Figure 6-3. Water content of the first tested grout, grout 1.

The water content varies greatly through the sample. The centre slices have a water content around 40%, which is similar to clays, while the top and bottom slices have a much lower water content around 25-30%.

6.1.1.2 Degree of compaction, void ratio and porosity

If the assumption is made that the sample is fully saturated the following data can be extracted from grout 1:

	Grout 1
ρ	1,88
e	0,88
n	47

Table 6-2. Obtained degree of compaction, void ratios and porosities for grout 1.

Where in *table 6-2*:

$$\begin{cases} \rho = \left[t/_{m^3} \right] \\ e = [-] \\ n = [\%] \end{cases}$$

The density of grout 1 is similar to clay and the void ratio is higher than the suggested critical void ratio of around 0,4 (Shuttle et al. *Prediction and validation of compaction grout effectiveness*).

6.1.2 Second series, grout 2

The second series was a further development of grout 1. Grout 1 had high content of clay and provided an impermeable body of grout (no surge attained after 48h).

A description of grout 2 is found in *table 6-3* and *figure 6-4.*

	Mesh [mm]	Weight [g]	% finer than
Grout 2	8		100
	4	44	94
	2	137	76
	1	161	54
	0,5	178	30
	0,25	81	19
	0,125	70	9
	0,063	29	6
	>0,063	41	0
Total		741	

Table 6-3. Grain sizes and weight ratios for the second tested grout, grout 2.

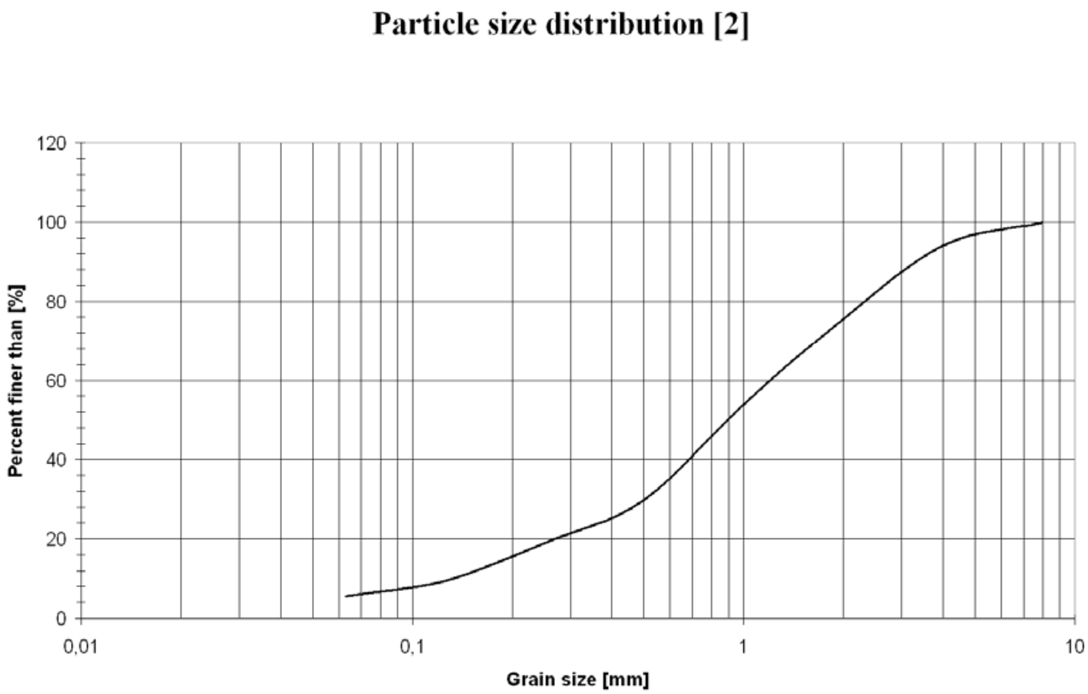


Figure 6-4. Particle size distribution curve of the second tested grout, grout 2.

The amount of clay in grout 2 was decreased in comparison to grout 1. The particle size distribution curve was also designed more smoothly. Surge of the permeameter was achieved when testing grout 2, which was not the case with grout 1.

The testing time for each sample was one week.

Test number	Duration (weeks)
A	1
B	1
C	1

Table 6-4. Duration of test for grout 2.

6.1.2.1 Permeability

The attained permeability for sample 2a, 2b and 2c is shown in *figure 6-5*.

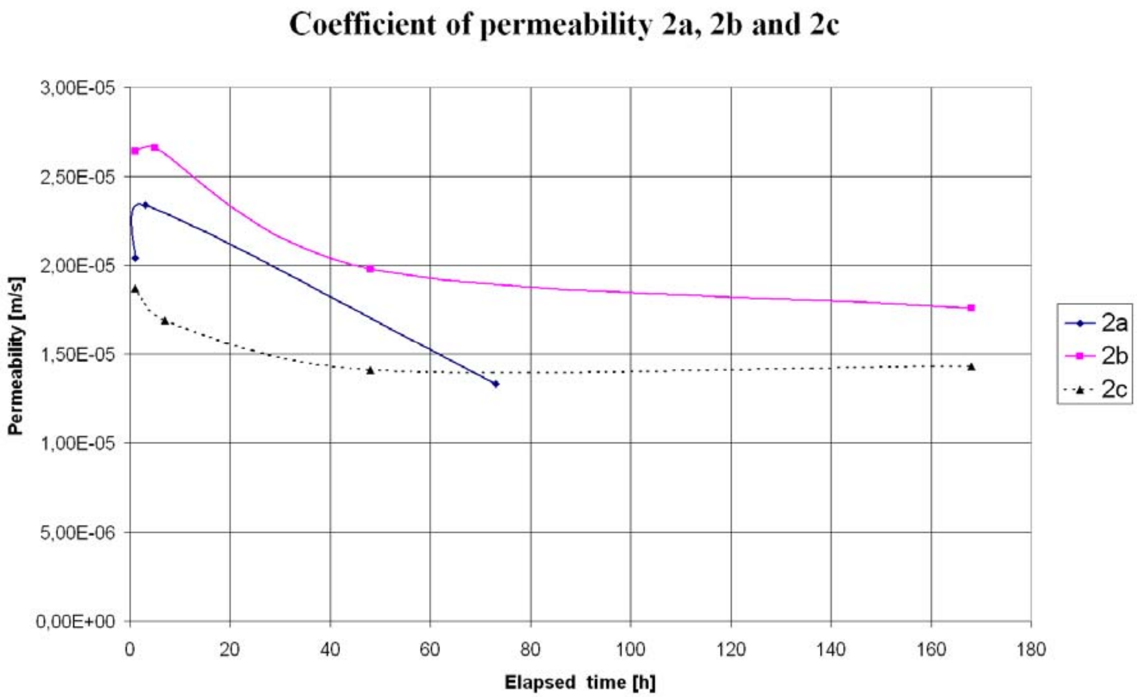


Figure 6-5. Change of the coefficient of permeability through time for grout 2a, 2b and 2c.

The average permeability of grout 2 is around $1{,}6 \times 10^{-5}\, {}^{m}\!/_{s}$ which is above the desired permeability of $1 \times 10^{-6}\, {}^{m}\!/_{s}$.

The hydraulic gradient of grout 2 throughout testing is given by formula **(9)**:

$$i = \frac{3{,}27}{0{,}14} = 23{,}4$$

6.1.2.2 Water content

The water content is measured for both un-surged samples and surged.

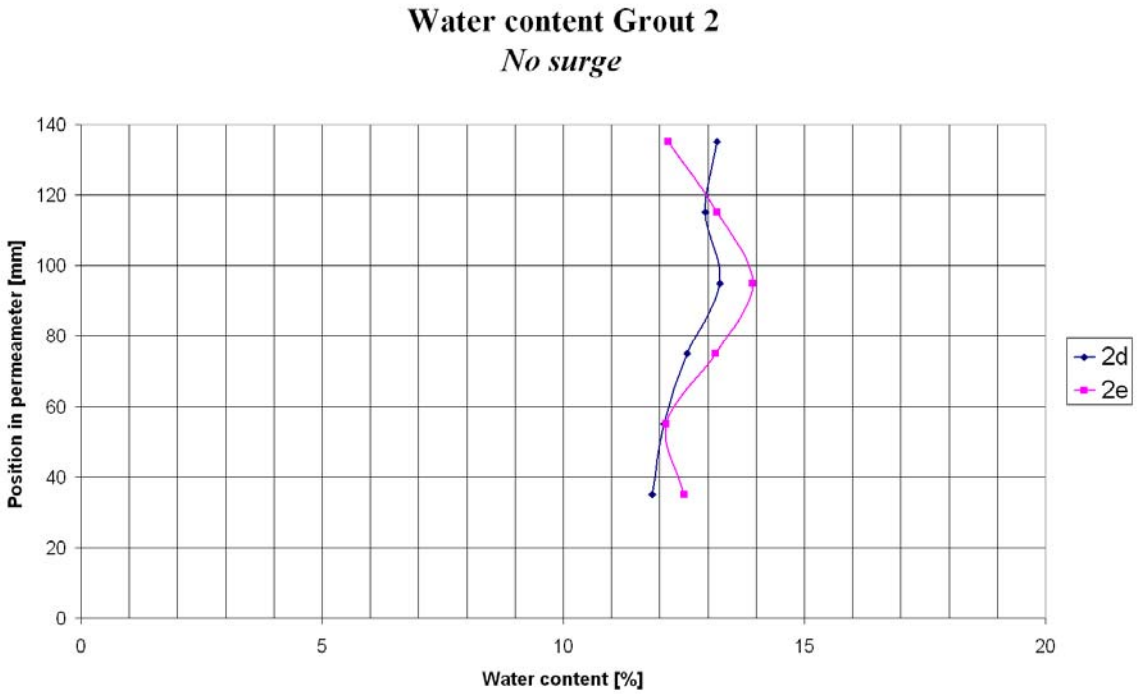


Figure 6-6. Variation of water content for un-surged grout 2d and 2e.

The water content if un-surged roughly falls in the range of 12% to 14%

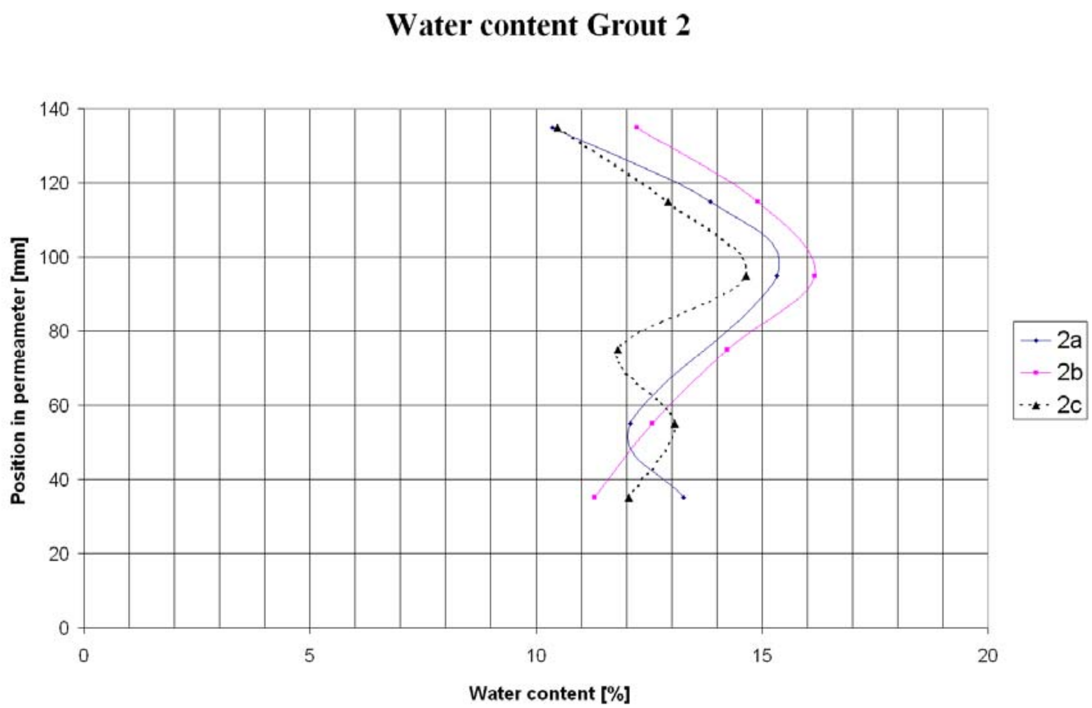


Figure 6-7. Variation of water content for surged grout 2a, 2b and 2c.

If surged, the water content of grout 2 varies within the range of 10 – 15% with virtually the same distribution throughout each sample. It can be seen that the maximum water content is found at the 95mm level in each sample.

6.1.2.3 Degree of compaction, void ratio and porosity

The obtained values for the surged grout samples 2a, 2b and 2c are:

	2a	**2b**	**2c**
ρ	2,22	2,21	2,24
e	0,35	0,36	0,33
n	26	26	25

Table 6-5. Obtained degree of compaction, void ratios and porosities for grout 2.

Where in *table 6-5*:

$$\begin{cases} \rho = \left[t/m^3 \right] \\ e = \left[- \right] \\ n = \left[\% \right] \end{cases}$$

The densities of the samples are around $2{,}22\ {}^{t}\!/_{m^3}$. This value corresponds to till but also sand and gravel. The void ratios are well below 0,4. From this point of view grout 2 has similar values as original till used in a dam core.

6.1.2.4 Migration of fines

No fines were found in the lower water trap at any time throughout the test series.

6.1.2.5 Particle size distribution curves for grout 2

Obtained particle size distribution curves for grout 2 if no surge is applied is found in *figure 6-8* and *figure 6-9*.

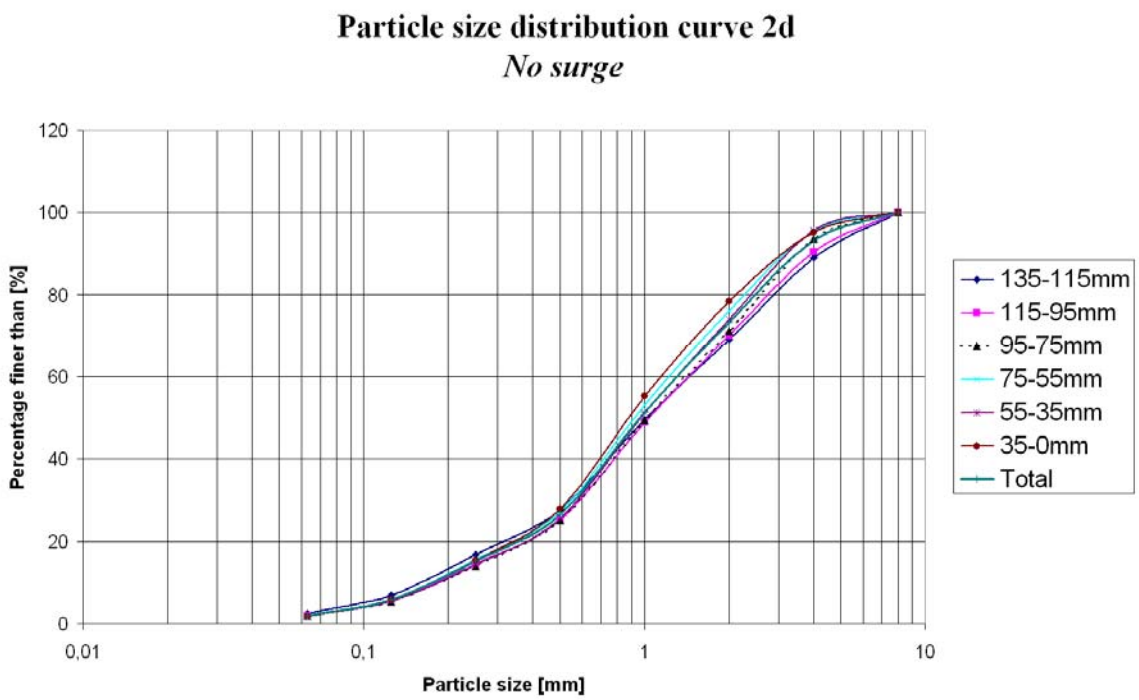


Figure 6-8. Particle size distribution curves for grout 2d. This grout has not been surged.

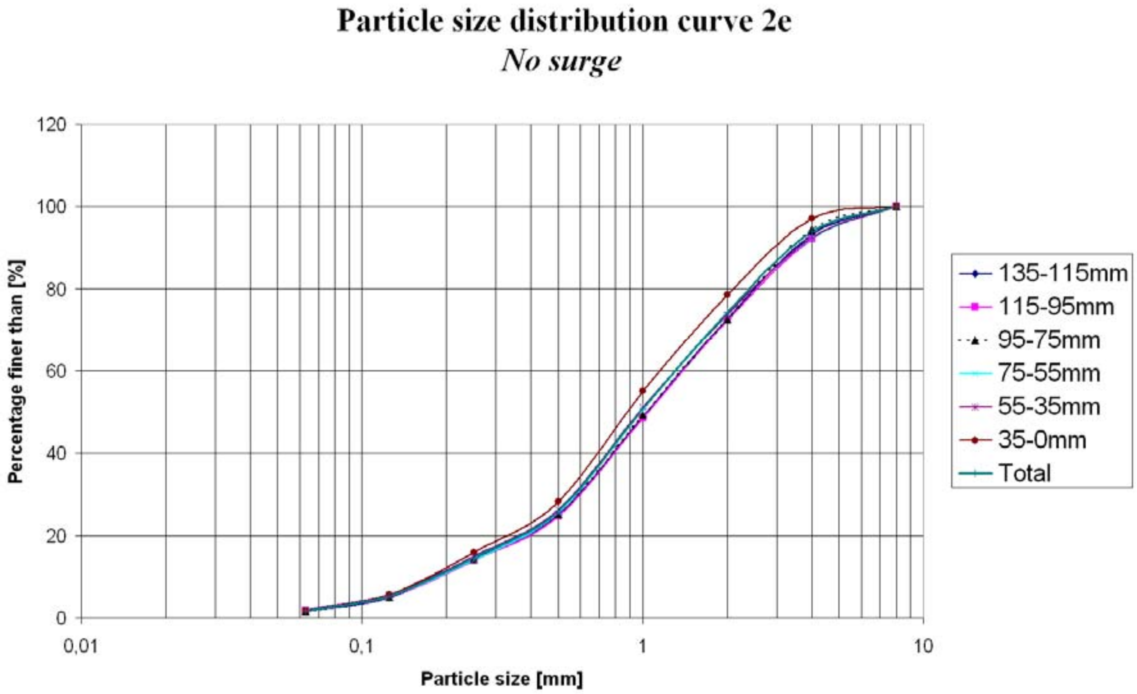


Figure 6-9. Particle size distribution curves for grout 2e. This grout has not been surged.

The particle size distribution curves in *figure 6-8* and *6-9* show how the particle size distribution curves differ for each level (each slice) in the un-surged samples of grout 2. These curves are used as a reference to make a comparison with the grout samples that have been surged. If the particle size distribution curves change due to surging, the surge action should be considered to cause particles within the grout structure to move.

The particle size distribution curves for grout 2 when surged follow in *figure 6-10* to *figure 6-12*:

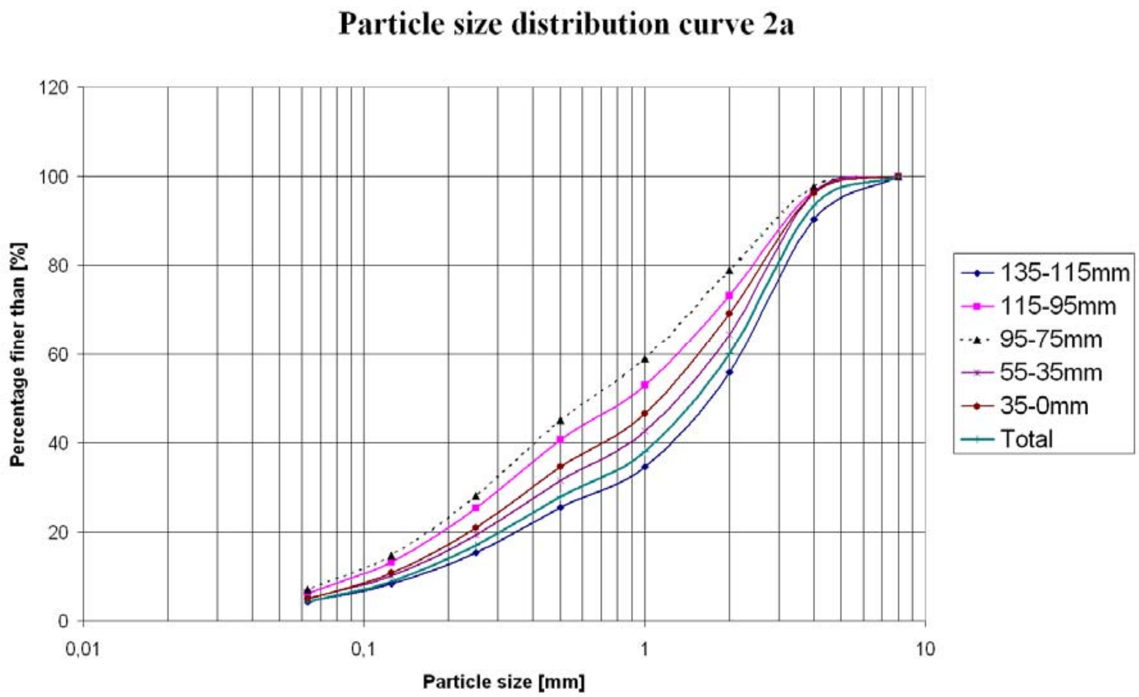


Figure 6-10. Particle size distribution curves for grout 2a. This grout has been surged.

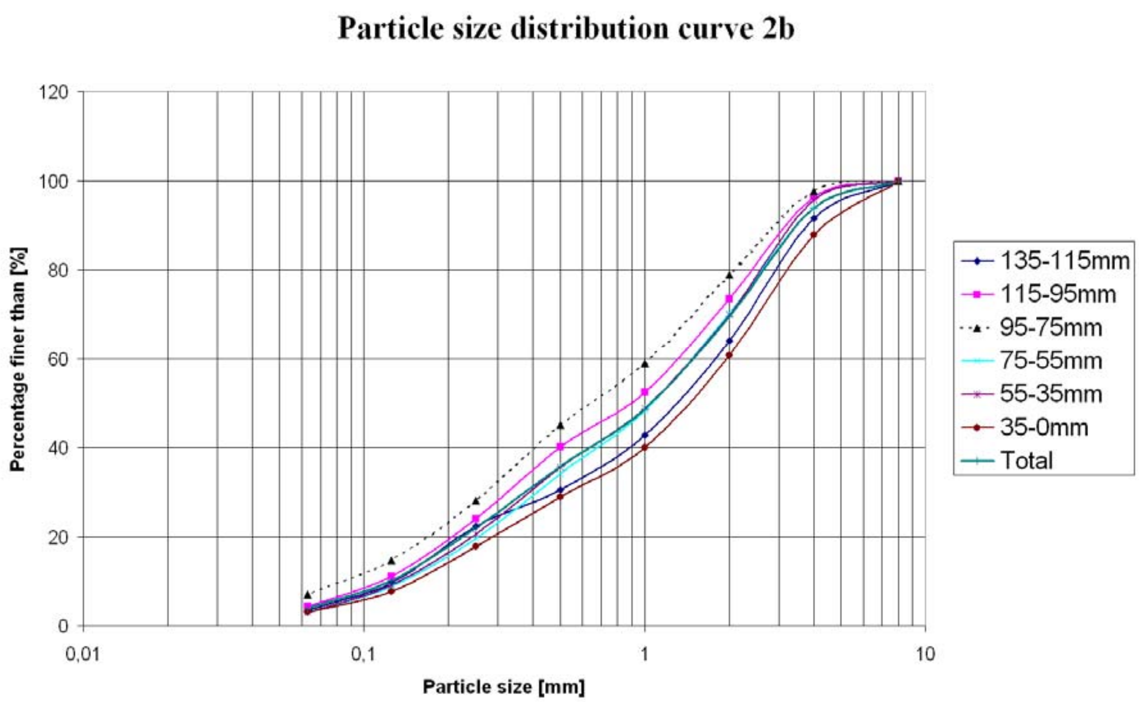


Figure 6-11. Particle size distribution curves for grout 2b. This grout has been surged.

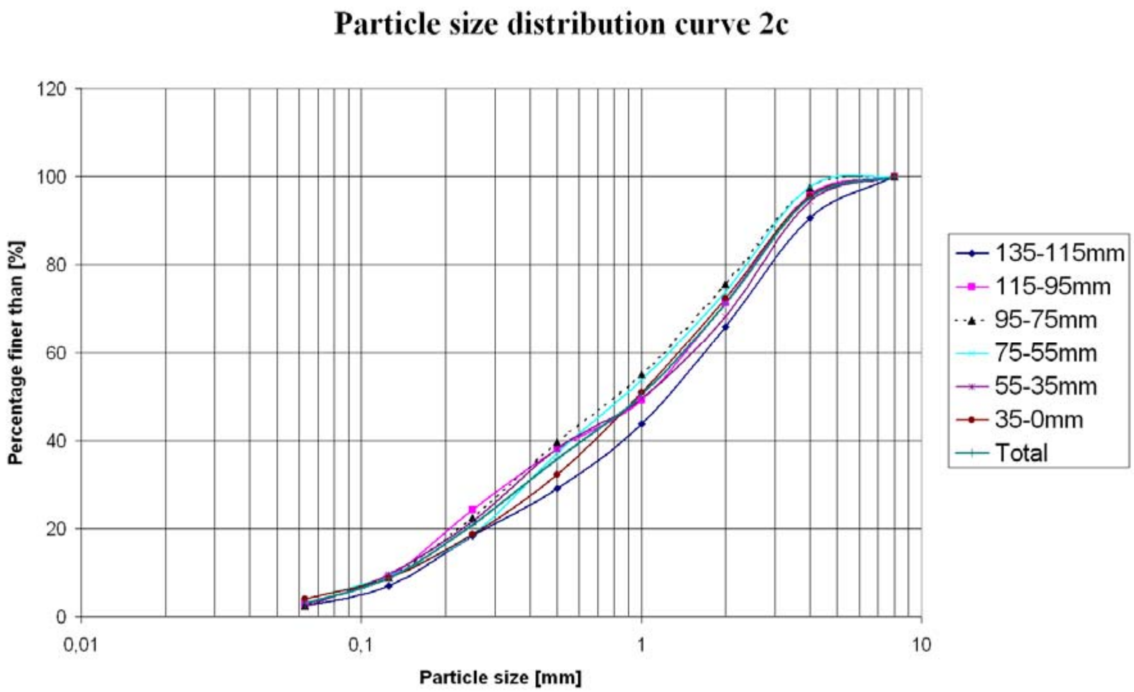


Figure 6-12. Particle size distribution curves for grout 2c. This grout has been surged.

Figure 6-10 to *figure 6-12* shows how the particle distribution curves differ in comparison with the un-surged grout. The variation of the distribution of the particle size distribution curves for each slice is greatly increased.

6.1.2.6 Histogram

Figure 6-13 and *6-14* shows the frequencies of the outcome at each sieve analysis of grout 2. These outcomes constitute the histogram of grout 2. The 4mm line, for example, represents the amount of soil particles that pass the 4mm mesh during sieving (in percentage) and how often a specific value is achieved.

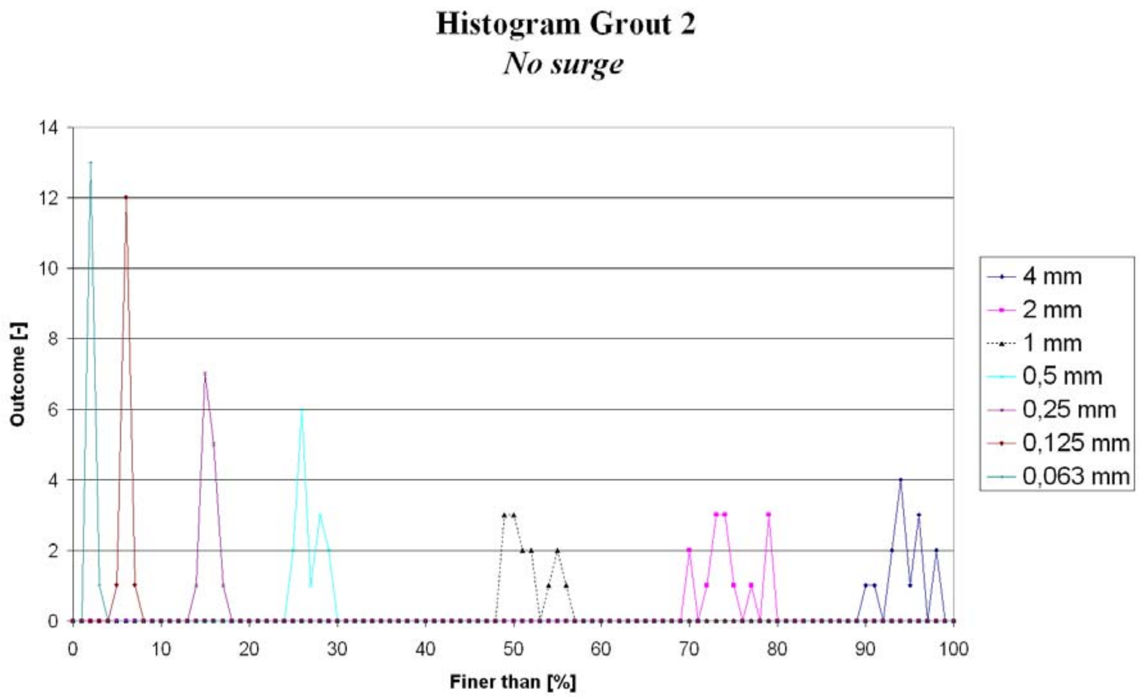


Figure 6-13. Histogram from tested grout 2 if un-surged.

Figure 6-13 indicates that the sieved grout is evenly distributed since the values obtained at every mesh are fairly concentrated.

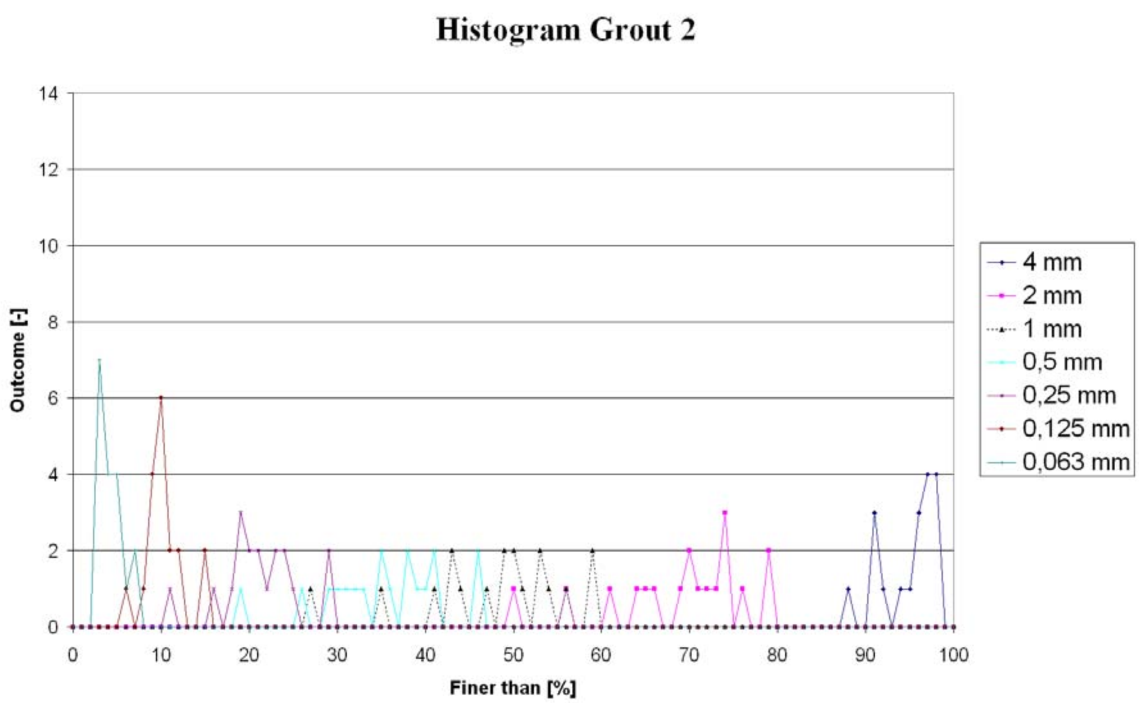


Figure 6-14. Histogram from tested grout 2 if surged

Figure 6-14 exhibits a wider variety of outcomes. Grout 2 is clearly affected by the surge.

6.1.2.7 Numerical variations of the particle distribution

To obtain a different view of the changes of the particle size distribution curve, numerical values need to be extracted from the particle distribution curves. The values are extracted from the outcome of particles caught in each mesh (4mm, 2mm, 1mm, 0,5mm, 0,25mm, 0,125mm and 0,063mm). By comparing the distribution before and after surge, it will give a good view of the enhanced spread of distribution due to surge.

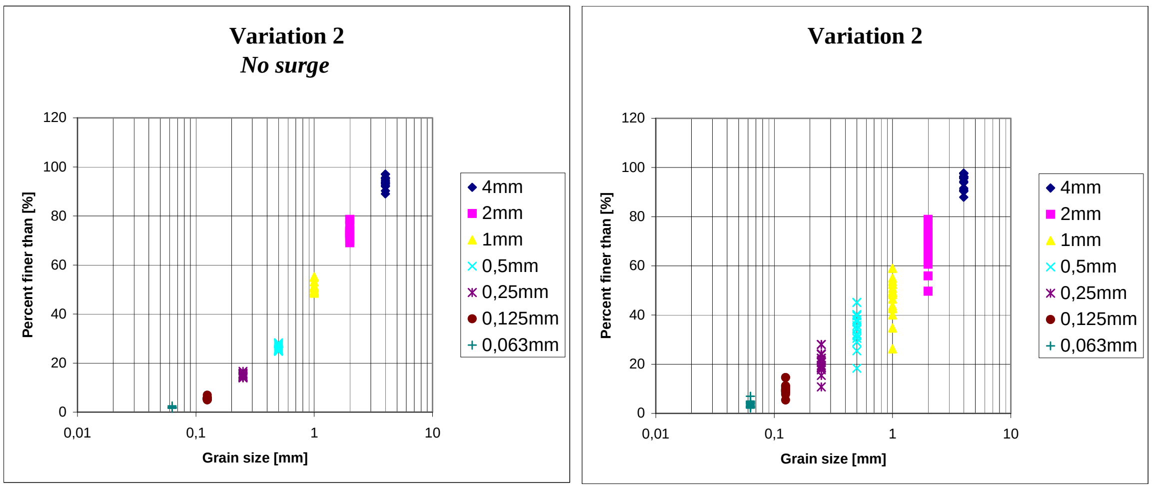


Figure 6-15. A plot of the points constituting the particle size distribution curves for tested grout 2. This allows to calculate the numerical values of the distribution at each point of measurement. To the left the un-surged grout and to the right the surged grout.

Numerical values from *figure 6-15* are found below in *table 6-6*. The maximal and minimal outcome from each mesh is used to describe the influence of surging.

Grout 2	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
No surge	MAX [%]	97	79	55	28	17	7	3
	MIN [%]	89	69	48	25	14	5	2
	DIFF. [%]	**8**	**10**	**7**	**3**	**3**	**2**	**1**
Surge	MAX [%]	98	79	59	45	28	15	7
	MIN [%]	88	50	26	18	11	5	2
	DIFF. [%]	**10**	**29**	**33**	**27**	**17**	**10**	**5**
	DIFF. Tot [%]	**2**	**19**	**26**	**24**	**14**	**8**	**4**

Table 6-6. The max/min differences of the variations at each mesh. The difference between un-surged and surged tests is found at the bottom row.

Table 6-6 shows the maximal and minimal values of the percentage of material passing each mesh. It also shows the difference between them. For example, the mesh with a width of 0,5mm exhibits an increased distribution of the outcome of 24% if grout 2 is surged.

If the standard deviations (STDEV) from *figure 6-15* are included, the differences become smaller, see *table 6-7*.

Grout 2 NOsurge			
mesh	cov	STDEV	mean
4	0,02	**2,33**	93,73
2	0,04	**3,06**	73,89
1	0,05	**2,43**	51,20
0,5	0,05	**1,29**	26,29
0,25	0,06	**0,83**	14,91
0,125	0,09	**0,49**	5,55
0,063	0,13	**0,24**	1,86

Grout 2 surge			
mesh	cov	STDEV	mean
4	0,03	**3,01**	94,57
2	0,11	**7,62**	68,61
1	0,17	**8,26**	47,49
0,5	0,20	**6,79**	34,67
0,25	0,20	**4,20**	20,81
0,125	0,22	**2,19**	9,82
0,063	0,37	**1,43**	3,87

Table 6-7. Standard deviations of distributions, grout 2.

If using *table 6-7* the following differences can be obtained, see *table 6-8*.

Grout 2	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
No surge	MAX [%]	96	77	54	28	16	6	2
	MIN [%]	91	71	49	25	14	5	2
	DIFF. [%]	5	6	5	3	2	1	0
Surge	MAX [%]	98	76	56	41	25	12	5
	MIN [%]	92	61	39	28	17	7	2
	DIFF. [%]	6	15	17	13	8	5	3
	DIFF. Tot [%]	1	9	12	10	6	4	3

Table 6-8. The max/min differences of the variations at each mesh. Standard deviation is included. The difference between un-surged and surged tests is found at the bottom row.

6.1.3 Third series, grout 3

The third series, grout 3 had a slight increase in fines in comparison with the second series, grout 2. A description of grout 1 found in *table 6-9* and *figure 6-16.*

	Mesh [mm]	Weight [g]	% finer than
Grout 3	8		100
	4	44	94
	2	120	79
	1	161	58
	0,5	160	37
	0,25	90	26
	0,125	70	17
	0,063	60	9
	>0,063	70	0
Total		775	

Table 6-9. Grain sizes and weight ratios constituting grout 3.

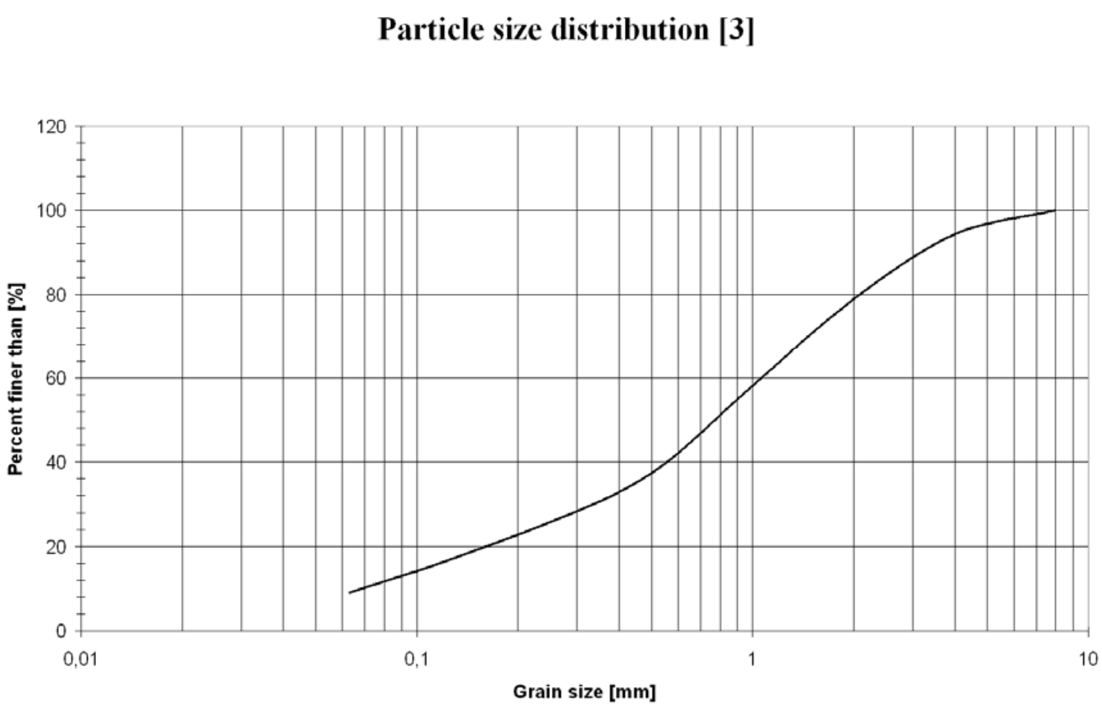


Figure 6-16. Particle size distribution curve of the third tested grout, grout 3.

The finer part of the particle size distribution curve for grout 3 is steeper than the same part of the particle size distribution curve for grout 2.

The test time was between one – two weeks.

Test number	Duration (weeks)
A	1
B	2
C	1
D	1

Table 6-10. Duration of tests for grout 3.

6.1.3.1 Permeability

Like the case from permeability testing of mixture 2 the permeability lowered throughout the tests. In general, the final permeability ranged between $8 \times 10^{-7}\ m/s$ and $3 \times 10^{-7}\ m/s$. This is almost equal or below the desired permeability of at most $1 \times 10^{-6}\ m/s$.

The hydraulic gradient of grout 3 throughout testing is given by formula **(9)**:

$$i = \frac{3{,}27}{0{,}14} = 23{,}4$$

The obtained permeability of grout 3 is found in *figure 6-17*.

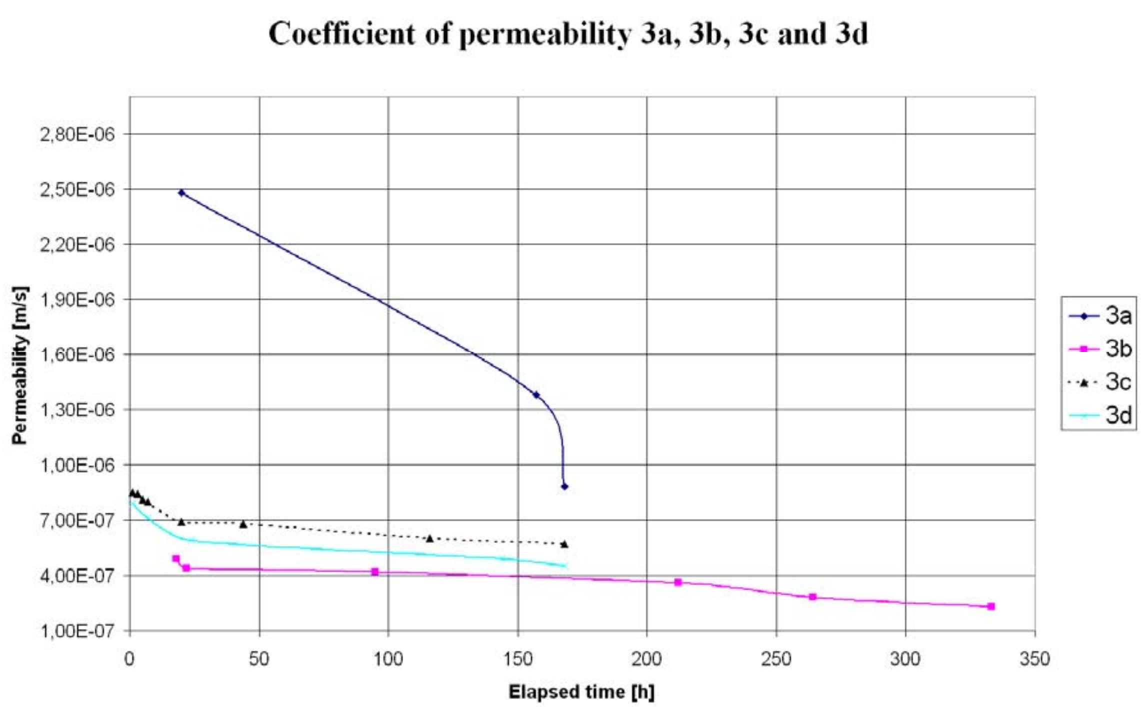


Figure 6-17. Change of the coefficient of permeability through time for grout 3a, 3b, 3c and 3d.

The permeability for sample 3a is higher than the others since a different method of preparation was used. Grout 3a was put inside the permeameter at dry state and back-surged, while 3b, 3c and 3d were given a water content of 17% during the mixing process.

6.1.3.2 Water content

The water content was measured for each separate slice. The values from this series are in the range of 14-16% for 3b, 3c and 3d, which is a bit higher than a normal core of an embankment. The water content for 3a is in the range of 15-18%. The water content in samples 3e and 3f (un-surged) where slightly higher than 3a, 3b, 3c and 3d.

The variation of water content does not vary as much as grout 2, which might indicate that the grout is more evenly distributed.

The water contents for the un-surged grout 3 are shown in *figure 6-18*.

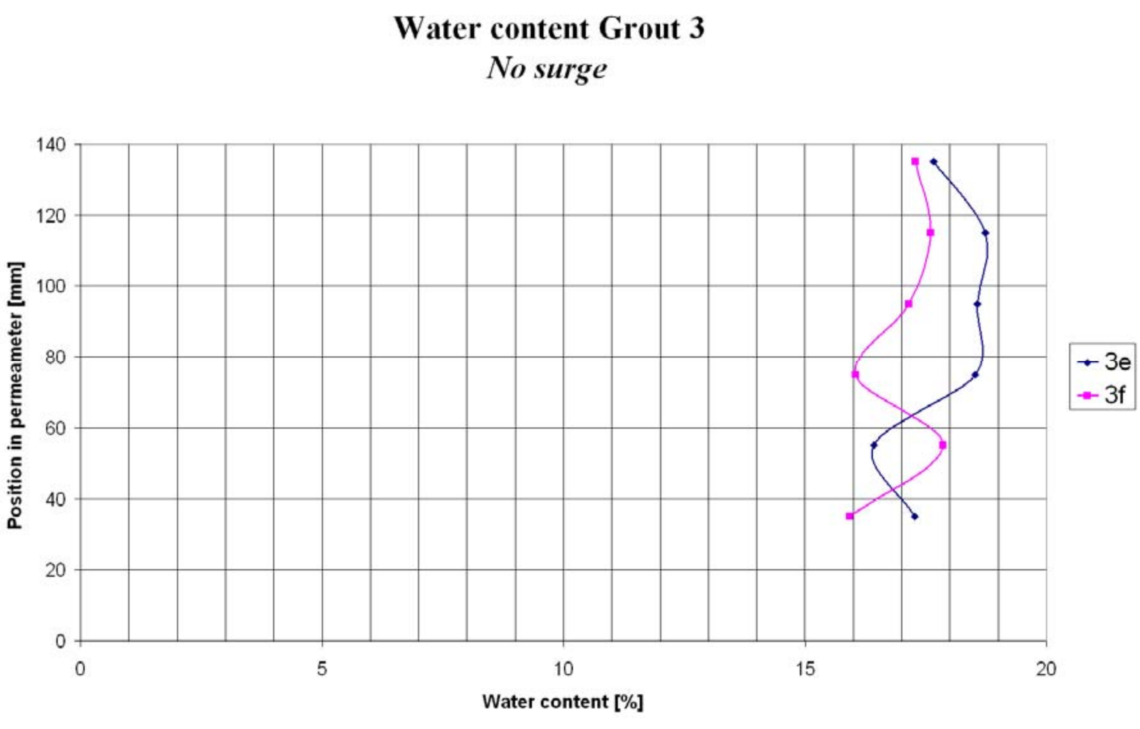


Figure 6-18. Variation of water content for un-surged grout 3e and 3f.

The water content if un-surged roughly ranges from 16% to 18%. Water contents of the surged grout 3 are shown in *figure 6-19*.

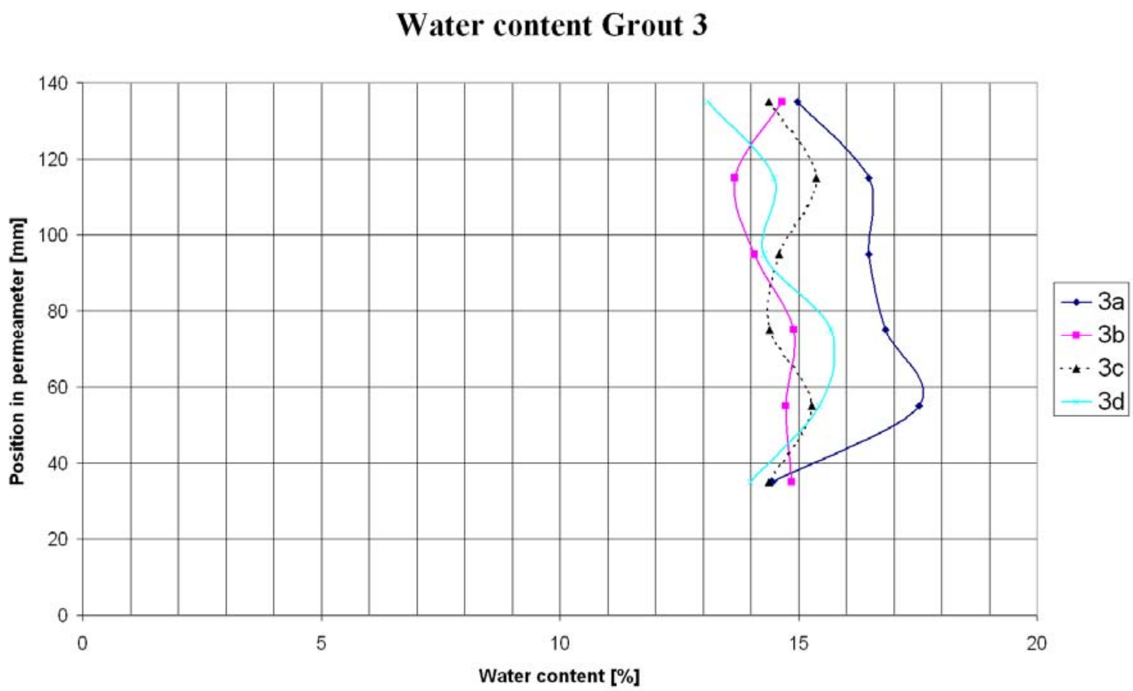


Figure 6-19. Variation of water content for surged grout 3a, 3b, 3c and 3d.

The water content of grout 3 when surged is more uniformly distributed and slightly higher if compared with the water content of surged grout 2.

6.1.3.3 Degree of compaction, void ratio and porosity

The obtained values for the surged grout samples 3a, 3b, 3c and 3d are:

	3a	**3b**	**3c**	**3d**
ρ	2,15	2,19	2,18	2,19
e	0,43	0,38	0,39	0,38
n	30	28	28	28

Table 6-11. Obtained degree of compaction, void ratios and porosities for grout 3.

Where in *table 6-11*:

$$\begin{cases} \rho = \left[t/m^3 \right] \\ e = [-] \\ n = [\%] \end{cases}$$

Grout 3a was compacted at a dry state. This resulted in a lower degree of compaction and higher void ratio and porosity. The density of grout 3 is around 2,18 t/m^3, which is similar to till but also sand and gravel. The void ratio is below 0,4 for 3b, 3c and 3d and similar to a normal dam core.

6.1.3.4 Migration of fines

No fines were found in the lower water trap at any time throughout the test series.

6.1.3.5 Particle size distribution curves for grout 3

Obtained particle size distribution curves for grout 3 if no surge is applied is found in *figure 6-20* and *figure 6-21*

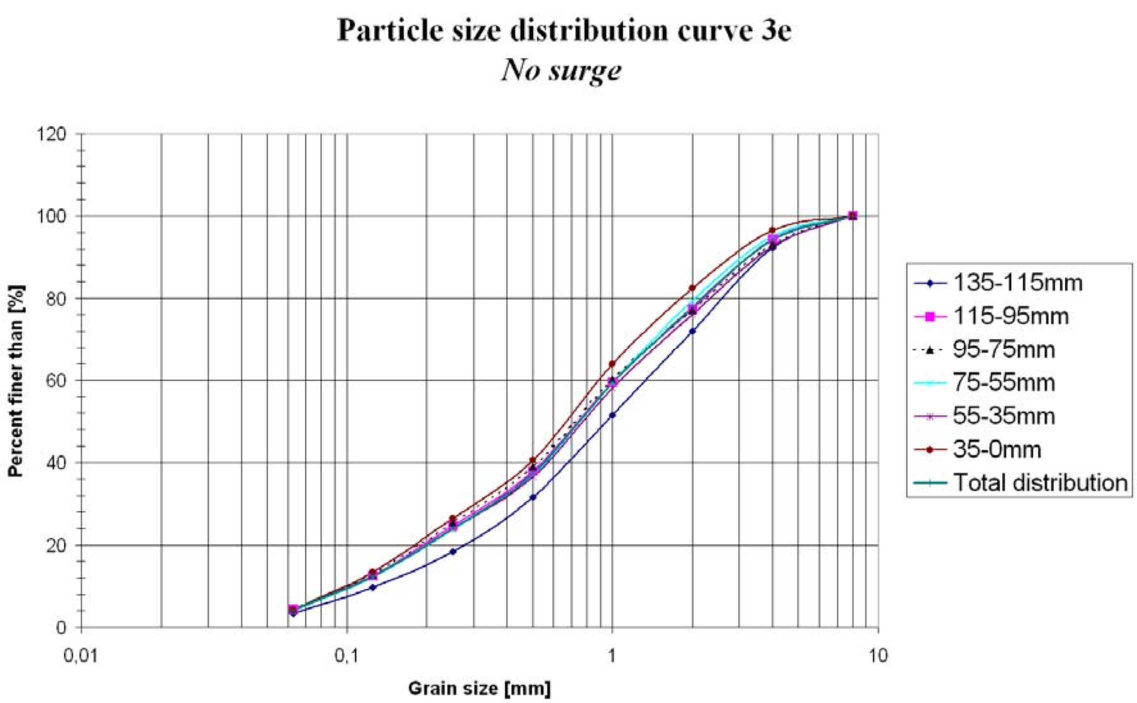


Figure 6-20. Particle size distribution curves for grout 3e. This grout has not been surged.

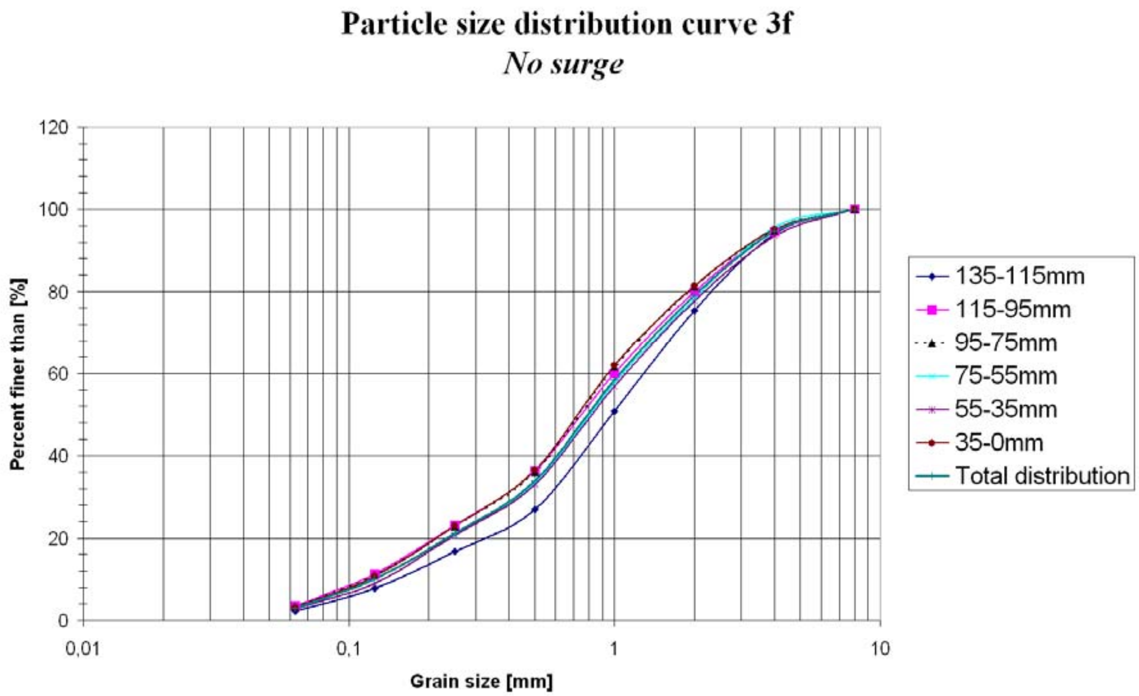


Figure 6-21. Particle size distribution curves for grout 3f. This grout has not been surged.

The particle size distribution curves from grout 3 when un-surged behave similar to grout 2. The very top slice has a coarser distribution and the bottom slice has a slight higher degree of fines. The protected slices underneath and above the top and bottom slice virtually posses similar distributions.

The sieve analysis for surged grout 3 can be seen in *figure 6-22* to *figure 6-25*.

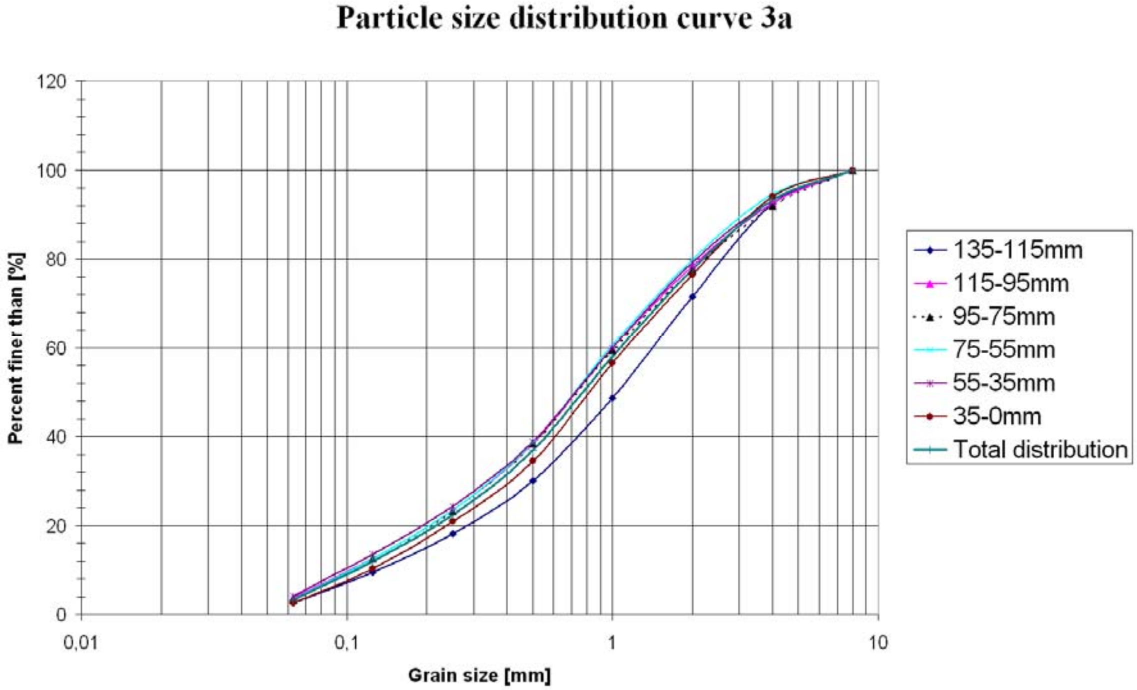


Figure 6-22. Particle size distribution curves for grout 3a. This grout has been surged.

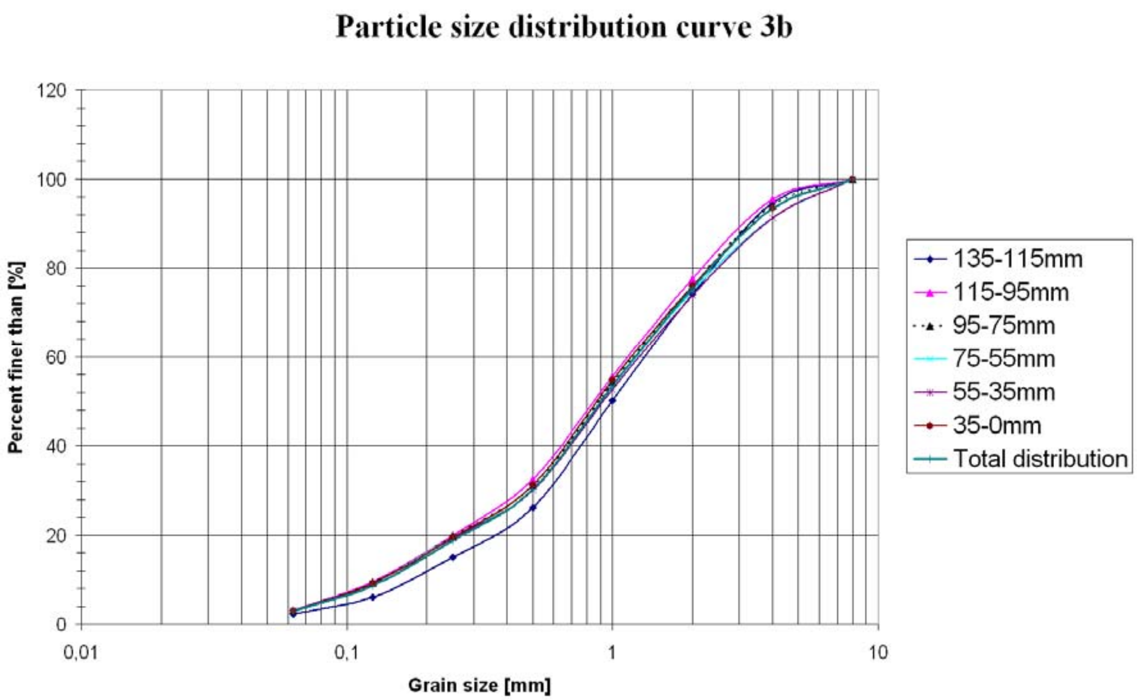


Figure 6-23. Particle size distribution curves for grout 3b. This grout has been surged.

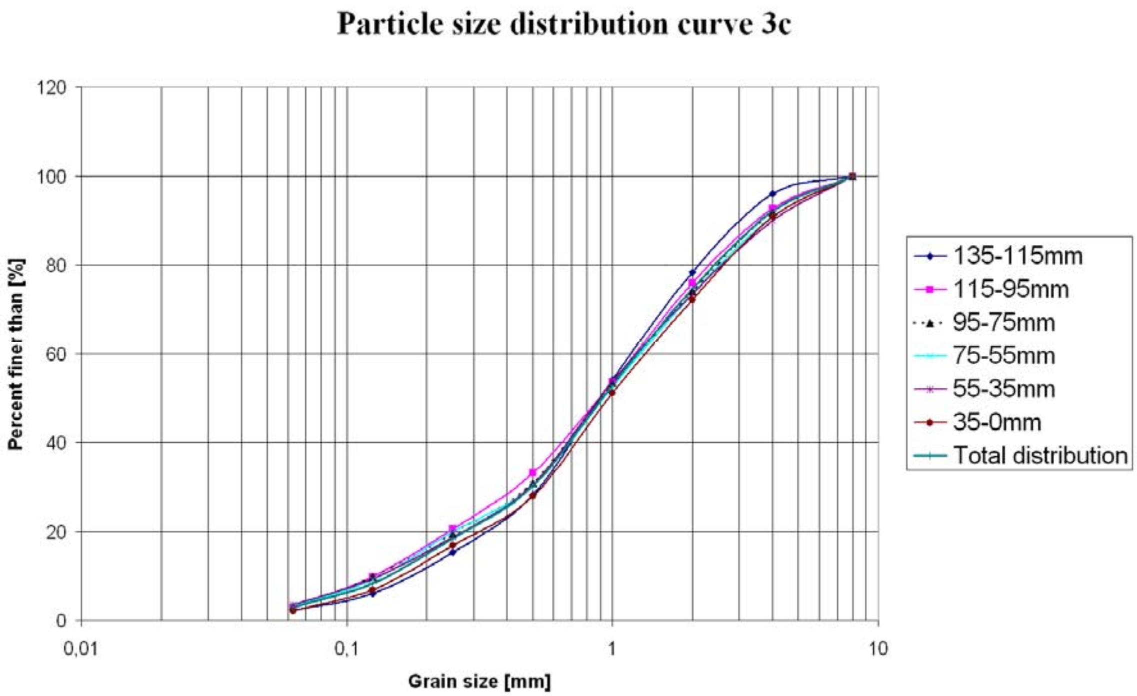


Figure 6-24. Particle size distribution curves for grout 3c. This grout has been surged.

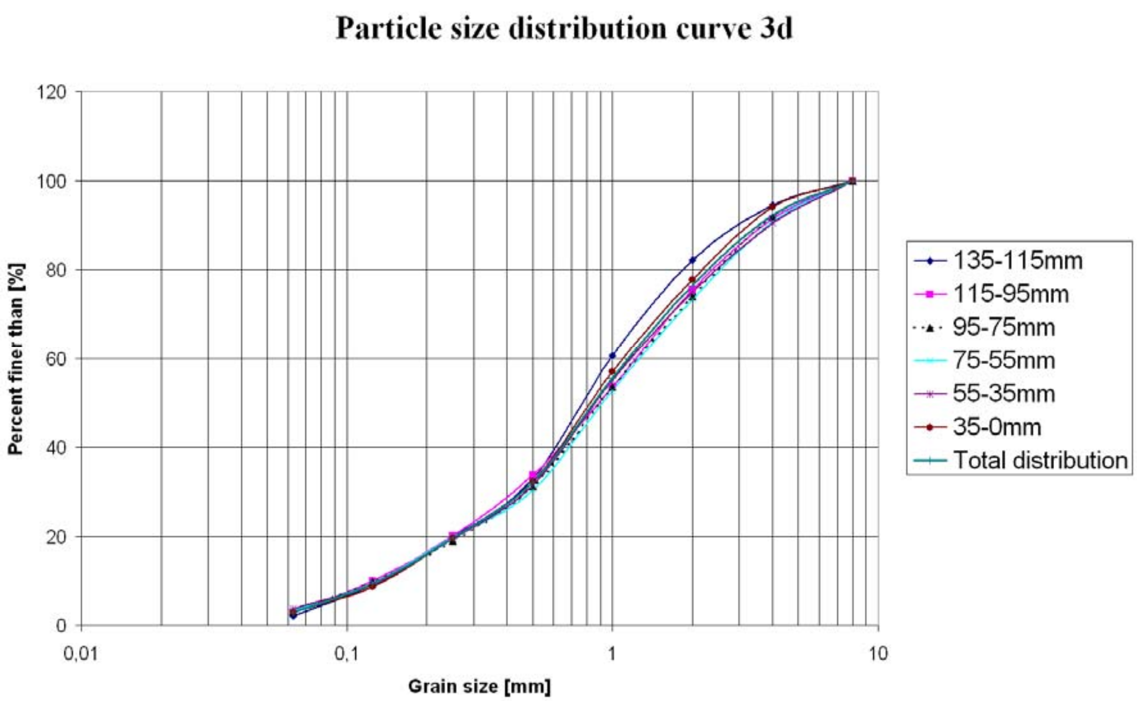


Figure 6-25. Particle size distribution curves for grout 3d. This grout has been surged.

The particle size distribution curves for grout 3 when surged greatly varies from the results obtained when testing grout 2. The differences between un-surged and surged samples are considerably smaller.

6.1.3.6 Histogram

Histograms have also been made for grout 3. The histograms are shown in *figure 6-26* and *figure 6-27*:

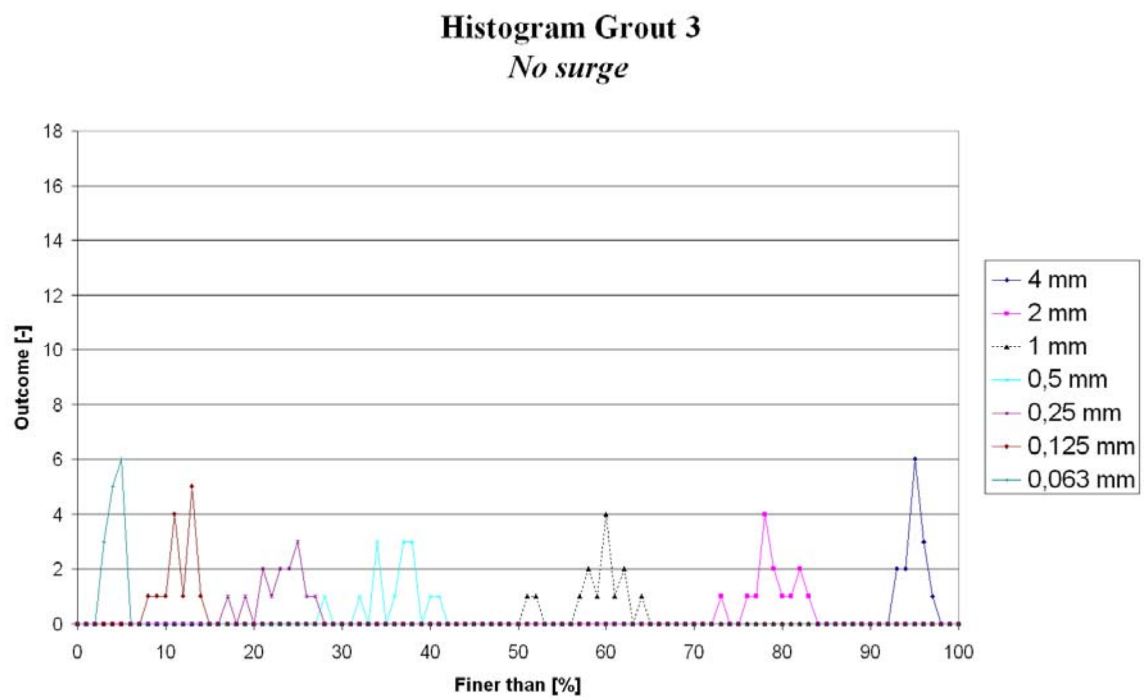


Figure 6-26. Histogram from tested grout 3 if un-surged.

Figure 6-26 indicates that the sieved grout 3, when un-surged is evenly distributed since the values obtained at every mesh are fairly concentrated.

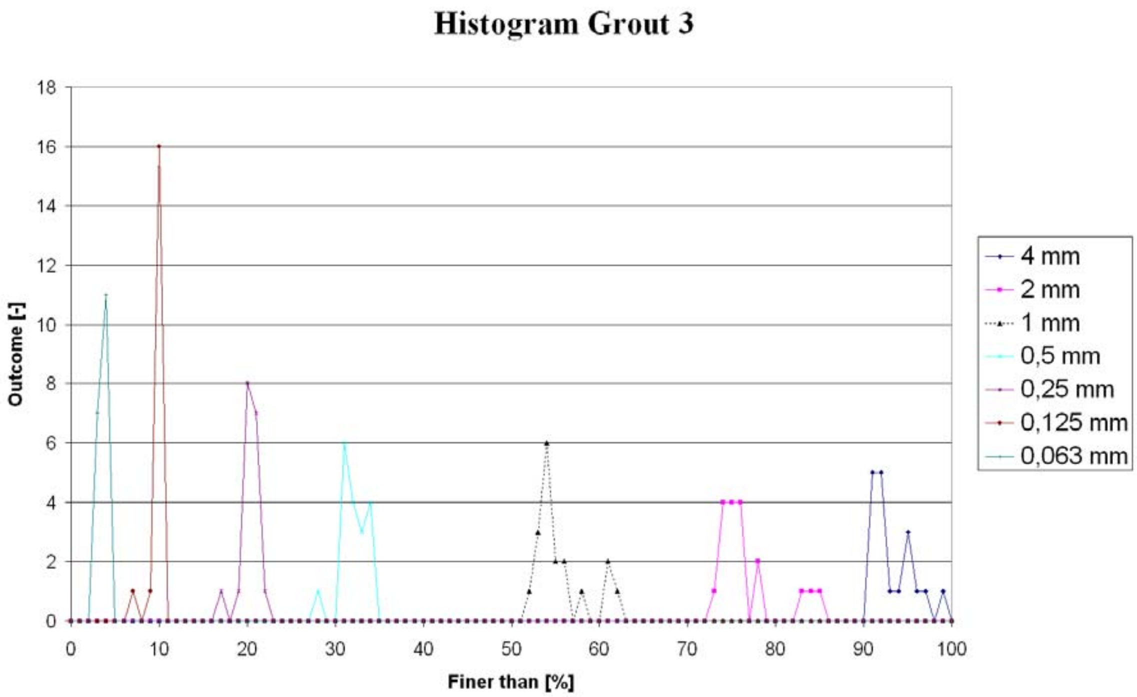


Figure 6-27. Histogram from tested grout 3 if surged.

Figure 6-27, a slight more concentrated variation of outcomes compared with the un-surged grout 3. Grout 3 is clearly less affected by the surge compared to grout 2.

It is noted that surging grout 3 results in less variations of the particle size distribution curves than the un-surged samples of grout 3. This should however also be seen as a change from the normal state (un-surged samples of grout 3) and it does not matter if the variations diminish or rise.

6.1.3.7 Numerical variations of the particle distribution

As the case for grout 2, a similar evaluation of the numerical values, constituting the particle size distribution curve is done.

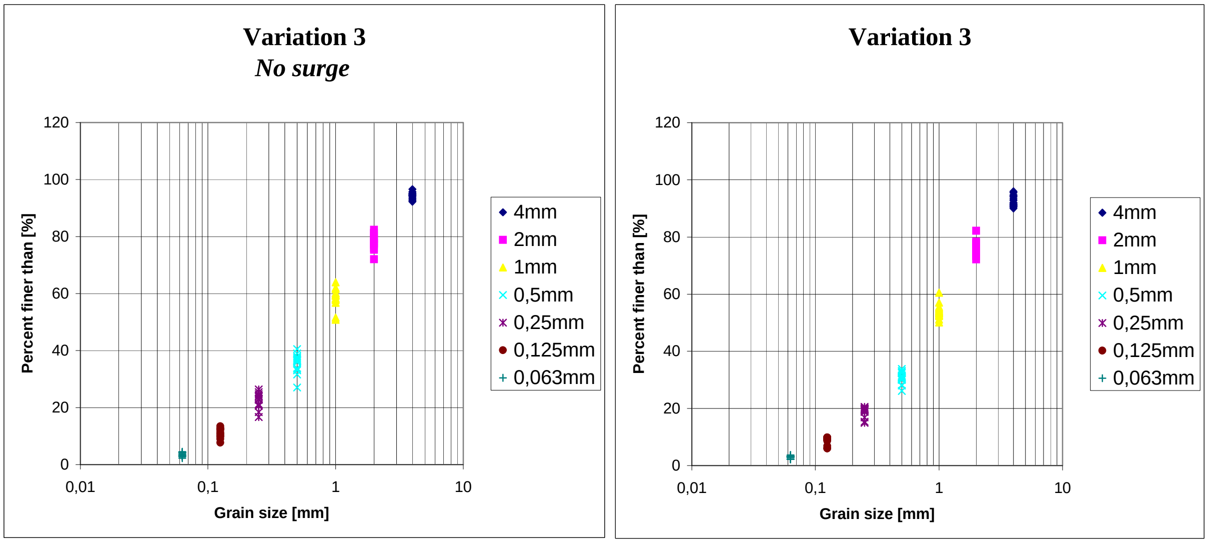


Figure 6-28. A plot of the points constituting the particle size distribution curves for tested grout 2. This allows to calculate the numerical values of the distribution at each point of measurement. To the left the un-surged grout and to the right the surged grout.

Numerical values from *figure 6-28* are found below in *table 6-12*. The maximal and minimal outcome from each mesh is used to describe the influence of surging.

Grout 3	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
No surge	MAX [%]	97	82	64	41	26	13	4
	MIN [%]	92	72	51	27	17	8	2
	DIFF. [%]	**5**	**10**	**13**	**14**	**9**	**5**	**2**
Surge	MAX [%]	96	82	61	34	21	10	4
	MIN [%]	90	72	50	26	15	6	2
	DIFF. [%]	**6**	**10**	**11**	**8**	**6**	**4**	**2**
	DIFF. Tot [%]	**1**	**0**	**2**	**6**	**3**	**1**	**0**

Table 6-12. The max/min differences of the variations at each point of measurement. The difference between un-surged and surged tests is found at the bottom row.

Table 6-12 shows the maximal and minimal values of the percentage of material passing each mesh. It also shows the difference between them. For example, the mesh with a width of 0,5mm exhibits an increased distribution of 8% if surged.

If the standard deviations (STDEV) from *figure 6-28* are included, the differences become smaller, see *table 6-13*.

Grout 3 NOsurge			
mesh	cov	STDEV	mean
4	0,01	**1,18**	94,35
2	0,03	**2,69**	78,21
1	0,06	**3,60**	58,49
0,5	0,10	**3,43**	35,46
0,25	0,12	**2,69**	22,50
0,125	0,15	**1,66**	11,10
0,063	0,20	**0,70**	3,57

Grout 3 surge			
mesh	cov	STDEV	mean
4	0,02	**1,87**	92,56
2	0,03	**2,39**	75,40
1	0,04	**2,27**	53,94
0,5	0,06	**1,93**	30,78
0,25	0,08	**1,57**	18,96
0,125	0,14	**1,24**	8,84
0,063	0,15	**0,45**	2,97

Table 6-13. Standard deviations of distributions, grout 2.

If using *table 6-13* the following differences can be obtained, see *table 6-14*.

Grout 3	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
No surge	MAX [%]	96	81	62	39	25	13	4
	MIN [%]	93	76	55	32	20	9	3
	DIFF. [%]	3	5	7	7	5	4	1
Surge	MAX [%]	94	78	56	33	21	10	3
	MIN [%]	91	73	52	29	17	8	3
	DIFF. [%]	3	5	4	4	4	2	0
	DIFF. Tot [%]	0	0	3	3	1	2	1

Table 6-14. The max/min differences of the variations at each mesh. Standard deviation is included. The difference between un-surged and surged tests is found at the bottom row.

6.2 Discussion about the results

The three different tested grouts behave differently. Grout 1 was to fine grained and provided no surging. Because of this, only the water content, degree of compaction, porosity and void ratio was examined. Grout 2 and grout 3 were both surged but differed in characteristics. All the obtained values from *chapter 6* will be discussed in this part of the thesis.

6.2.1 Discussion about the permeability

Since grout 1 never underwent surge, its permeability could not be measured. This was however possible for grout 2 and grout 3. The permeability varied for all tested grouts in a similar way; it diminished with time. This fact can be interpreted in the following ways:

- The clay inside the soil swells throughout the test time. This seals the possible leakage paths along the sides of the permeameter;
- The pores of the filter-stones were clogged by escaping fines;
- A concentration of fines inside the samples was created due to particle transport. This may eventually lead to a stratum of lower permeability.

The first two points (swelling and clogging) are most likely to occur. The swelling of the clay could be seen through the Plexiglas as the leakage paths between the grout and the permeameter diminished. When the tested grout was taken out from the permeameter it also expanded slightly, proving the swelling ability of the clay.

When dismantling the permeameter, a very small amount of fines were always found in and around both the top and bottom filter stone.

The obtained permeability seems to be connected to the total density of the grout. When performing test 3a, the grout had a lower density, resulting in a higher permeability.

6.2.2 Discussion about the water content

The variation of water content varied in a similar for all three grouts. The top and bottom slice were in general lower than the center slices. The variation of this kind suggests three possible reasons:

- The degree of fines in the top and bottom slices are considerably lower than in the centre slices;
- The top slice, which neighbors a filter stone is more affected by the initial water flow, causing fines to move downwards in the permeameter. The bottom slice has no protection underneath, only a filter stone with a considerably higher permeability;
- The bottom slice is more easily drained when dismantled.

Explanation one and two suggest that viewing the water content diagram may give an indirect evaluation of the amount of fines in each slice. If the amount of fines is higher, than the grout should be able to maintain a larger content of water. This is most probably the reason for the water content to vary through each sample. The water content varies most for grout 1 and least for grout 3.

The previous reasoning of variations of the water content can however be compared with the particle size distribution curve for grout 2. For grout 2, the measured water content was in all three cases at a maximum at the 95mm level. This does correspond to the particle size distribution curves of the same grout at the same level. The particle size distribution curve for surged grout 2 (*figure 6-10* to *figure 6-12*) shows that the sieve analysis performed at the 95mm slice has the highest concentration of fines. The same analysis can also be done for grout 3. This is however more difficult since the variations between the particle size distribution curves are too small.

From this a possible conclusion may be drawn:

> ***"If the water content is evenly distributed throughout the grout sample, the different fractions of the grout should also be evenly distributed"***.

It may therefore be possible to tell by viewing the water content only if the tested grout has suffered from particle transport.

6.2.3 Discussion about the degree of compaction, porosity and void ratio

Even if grout 2 had a higher total density and lower void ratio than grout 3 it suffered from larger variations due to surging. This fact suggests that having a critical void ratio of 0,4 should not be considered applicable for all soils. A critical void ratio should be unique for every soil with varying particle size distributions.

It is also interesting to view the result from grout 3a and compare it with grout 3b, 3c and 3d. Since it had a lesser degree of compaction it should be more prone to suffer from changes. However this was not the case, which may indicate that the distribution of the grout is more important than the density and void ratio of the grout.

6.2.4 Discussion about the migration of fines

No fines were caught in the lower water trap at any time throughout the test procedure for all three grouts.

6.2.5 Discussion about the sieve analysis

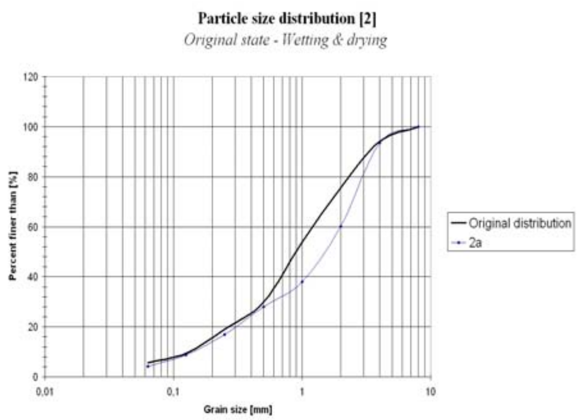
There is a clear difference between grout 2 and grout 3 after surging. The sieve analysis shows that surging can cause the particle size distribution curve to differ from its normal state. The normal state is given by sieving of the un-surged grout. This is in particular the case for grout 2. The particle size distribution curves for grout 3 when surged greatly varies from the results obtained when testing grout 2. Since the distribution is more uniform for grout 3, the fines of this grout are considered less prone to move. This fact alone should make grout 3 more suitable as a grout material inside a dam. Grout 2 separates if surged and if separation occurs, there will be an enlarged risk for internal erosion to occur within the dam structure.

The correlation between the water content and the particle size distribution as described in *chapter 6.2.2*, tells that this method of evaluation works.

It can be noted via the particle size distribution curves that the bottom slice has a higher degree of fine material if un-surged. This is explained due to the following fact:

- During the mixing process of the sample the coarsest grain sizes tend to have a slight concentration of the bottom of the mixing bowl. When the mixture is poured into the permeameter these coarser grains enters the permeameter at a later stage. This has been noted both in an empirical way and by viewing the particle size distribution curves.

If the original particle size distribution curve is compared with the outcome after a permeability test it can be noted that there is a loss of fines, see *figure 6-29*.



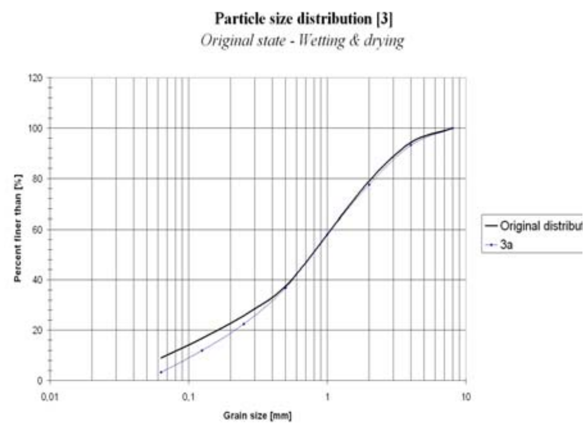


Figure 6-29. "Loss" of fines detected by comparing the original particle size distribution curve (before wetting) with the same grout but surged. Grout2 is on the left and grout 3 is on the right.

However this is not true. The fines stay inside the grout structure since no particles have been found in the sediment trap. This could be explained by the method of evaluation. Since sieving is used after a process of wetting and drying it is virtually impossible to separate all the clay particles from coarser grains. During drying of the slices, finer particles stick to coarser grains, giving the impression of a loss of clay particles. The clay will in this case work similar to cement causing both clay and finer particles to undergo an adhesive process. This can furthermore be visualized (see *figure 6-30*).

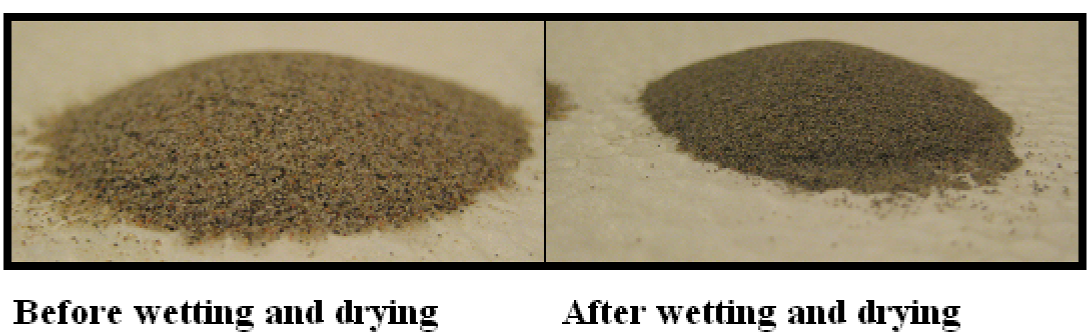


Figure 6-30. Soil particles trapped at the 0,125mm mesh colored by the grayish clay.

As seen in *figure 6-30* the two photos represent the same grain size. After wetting and drying, the soil has been "contaminated" by the grayish clay.

The sieve analysis could be complemented with sedimentation but to do so, according to standard (ISO/TS 17892-4), the degree of fines must exceed 10%. The degree of fines for grout 2 and grout 3 is 6% and 9% respectively.

If complementing the sieve analysis with a sedimentation analysis, this would give a definite result of the final particle size distribution curve of the surged grout samples. This would also allow a better comparison in terms of extracting different values of d such as d_{15}, d_{50} and d_{85}.

6.2.6 Discussion about the histograms

If surged, grout 2 exhibits a wide enlargement of distribution of the outcomes. By comparing the histograms before and after surging, this can be seen. The concentration of each unique outcome has a wider distribution when grout 2 has been surged. This suggests that grout 2 is prone to internal erosion. Material is moving within the grout, thus causing the outcome to differ after surging.

The histograms for grout 3 show that the distribution of the particle size distribution curves is actually larger if un-surged than surged. This should however be considered of lower importance since the goal is to spot enlargements of variations between un-surged and surged samples. An enlargement of the variations may be either a more "narrow" distribution or a wider. If the histograms of grout 2 are compared with the histograms of grout 3, the latter exhibit a much lesser change of variations due to surge.

6.2.7 Discussion about the numerical variations of the particle distribution

If the numerical variations of the particle size distribution curve for grout 3 is compared with the same for grout 2 to the difference between them is evident. Grout 3 suffers from a lower degree of variations of its particle size distribution curve than grout 2 if surged. This can be seen in *table 6-15*.

	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
Grout 2	DIFF. [%]	2	19	26	24	14	8	4
Grout 3	DIFF. [%]	1	0	2	6	3	1	0

Table 6-15. The differences between grout 2 and grout 3 when un-surged/surged are compared.

A similar view can also be obtained by viewing the standard deviations (STDEV) of the results, see *table 6-16*. This describes the change in variations, obtained from each mesh during sieve analysis. The frequencies of outcomes are included in the standard deviation where in *table 6-15*, they are not. Therefore, the differences of deviations are not as large.

STDEV	Mesh [mm]	4	2	1	0,5	0,25	0,125	0,063
Grout 2	DIFF. [%]	1	9	12	10	6	4	3
Grout 3	DIFF. [%]	0	0	3	3	1	2	1

Table 6-16. The differences between grout 2 and grout 3 when un-surged/surged are compared. Standard deviation is included.

The different results from the sieve analysis of grout 2 and grout 3, which indicates a lesser degree of particle separation for the latter indicates that the hypothesis:

> ***"Designing the particle size distribution curve will reduce the risk of internal erosion"***

is verified. Since the only difference between grout 2 and grout 3 is their particle size distribution curves, it should be concluded that by designing the particle size distribution curve it is possible to diminish the risk of internal erosion of a presumptive grout. A different particle size distribution curve in turn results in variations of all the recorded and measured characteristics.

7 Results From the Filter Box Testing

This chapter will present the results from the filter box testing. The results are divided into two main parts:

- Results from the usage of hardening grouts (test 4 and test 6);
- Results from the usage of flexible grouts (test 5 and test 7).

7.1 Results from the usage of hardening grouts

Two successful repetitive tests were performed with a hardening grout. The tests are called test 4 and test 6.

7.1.1 Rate of seepage

The final rate of seepage through the filter box setup when using hardening grouts is around $8{,}5\,{}^{ml}\!/_{10\text{ min}}$. The variations through time can be seen in *figure 7-1*.

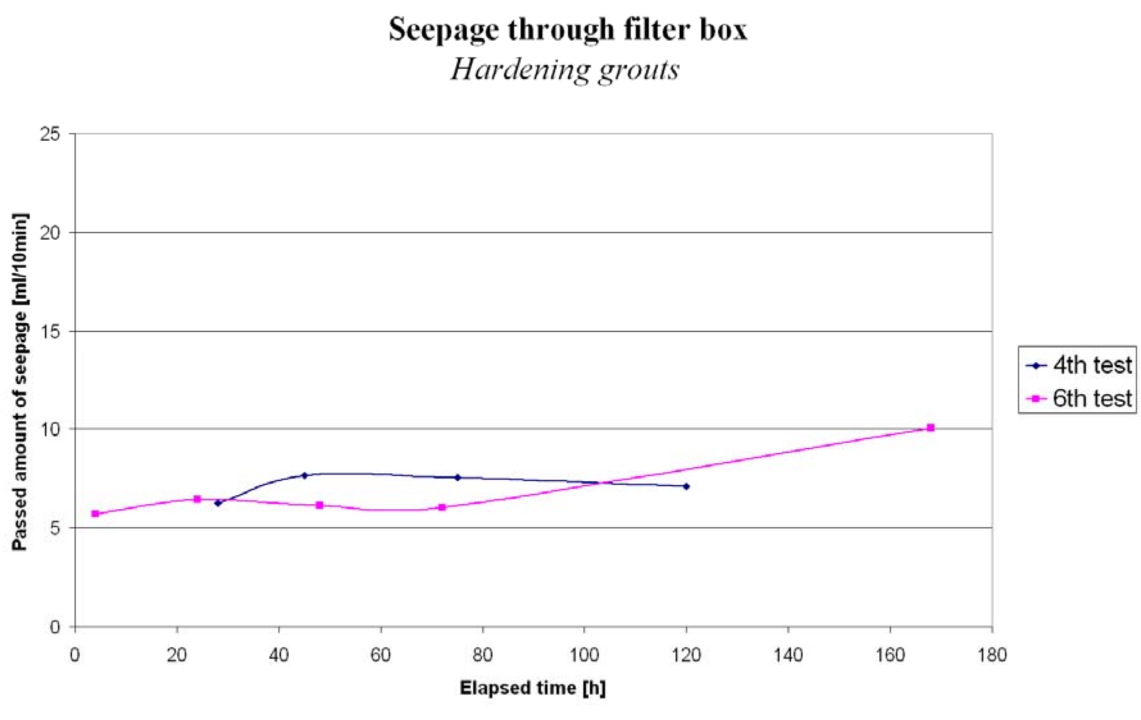


Figure 7-1. Measured rate of seepage for test 4 and test 6, hardening grouts.

The rate of seepage is rather uniform for test 4 but increasing for test 6.

7.1.2 Water content

The water content demonstrated in *figure 7-2* is the water content attained after the testing with the hardening grout.

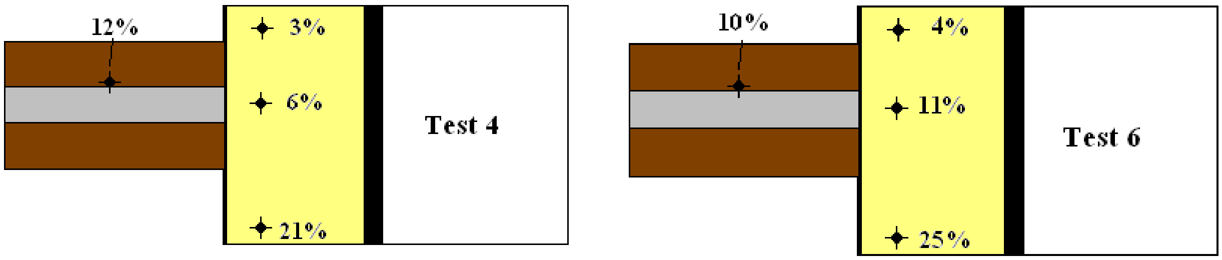


Figure 7-2. Measured water content in different areas of test 4 and test 6, hardening grouts.

7.1.3 Uniaxial compressive strength

The attained compression strength of test 4 and test 6 where respectively:

Test 4

Height [mm]	**Diameter** [mm]	**Weight** [g]	**Load** [N]	**Density** [kg/m3]	**Compressive strength** [Mpa]
58	45	178	33,4	1930	21,00

Table 7-1. Uniaxial compressive strength test of the sand/cement plug after test 4.

Test 6

Height [mm]	**Diameter** [mm]	**Weight** [g]	**Load** [N]	**Density** [kg/m3]	**Compressive strength** [Mpa]
54	45	165	19,35	1921	12,17

Table 7-2. Uniaxial compressive strength test of the sand/cement plug after test 6.

7.1.4 Undrained shear strength.

The undrained shear strength is normally around 50 – 100 kPa (Stål et al 1984) for this type of core soil. Since the uniaxial compressive strength is measured it cannot be compared with the undrained shear strength of the core soil, extracted with the cone penetration test.

Index for test 4, given by formula **(21)**:

$$I_{Mean} = 0{,}004$$

Where the dividend of I_{Mean} is 75 kPa.

Index for test 6, given by formula **(21)**:

$I_{Mean} = 0{,}006$

Where the dividend of I_{Mean} is 75 kPa.

7.1.5 Dissection photos

Photos taken after dismantling the filter box. Photos of both test 4 (*figure 7-3* to *figure 7-5*) and test 6 (*figure 7-6* to *figure 7-7*) can be viewed

7.1.5.1 Photos from filter box test 4, hardening grout

Figure 7-3. View of downstream side after test 4, hardening grout.

In *figure 7-3* the downstream side of the aluminium tube is viewed. The downstream side is facing the filter box, thus particles from the filter (filter sand) can be seen as stuck against the cohesive core soil. The main focus of *figure 7-3* should be put on the circular shape in the centre. The circular shape is in fact a crack that has developed between the hardening grout and the core soil. The circular shape origins from the shape of the mould used to create the damaged part of the core soil inside the tube. Cracks of this kind were not found at the other end of the tube (upstream side, facing the cone head).

Figure 7-4. Close-up view of crack, found at the downstream side after test 4, hardening grout.

Figure 7-4 shows a close-up of the same crack as shown in the previous photo. It can be seen that the crack has not just developed in the thin layer of sand but also between the core soil and grout. The crack at this location has a width of about 0,4 - 0,5mm.

Figure 7-5. Close-up view of crack, found at the downstream side after test 4, hardening grout.

Figure 7-5 shows a further slight excavation of downstream surface of the core soil. The excavation of the tube is done by carefully brushing the surface with a brush. The crack at this very point has a width of about 1-2mm. The cracks were in particular found at the upper side of the sand/cement grout plug.

7.1.5.2 Photos from filter box test 6, hardening grout

The second test of hardening grout was a repetition of test 4. The similarities between the two tests are obvious. The width and extension of the cracks found were however considerably smaller than the cracks observed after test 4.

Figure 7-6. View of downstream side after test 6, hardening grout.

Figure 7-6 confirms the photos taken from test 4. Cracks develop on the downstream side of the surged core soil/grout.

Figure 7-7. Close-up view of crack, found at the downstream side after test 6, hardening grout.

Figure 7-7 shows a close up of the crack when further excavations have been done. To the right of the photo is the core soil and to the left the hardening grout. At the interface, the crack can clearly be seen. The cracks were in particular found at the upper side of the sand/cement grout plug. Cracks of this kind were not found at the other end of the tube (upstream side, facing the cone head).

The orientation of the found cracks can be seen in *figure 7-8*.

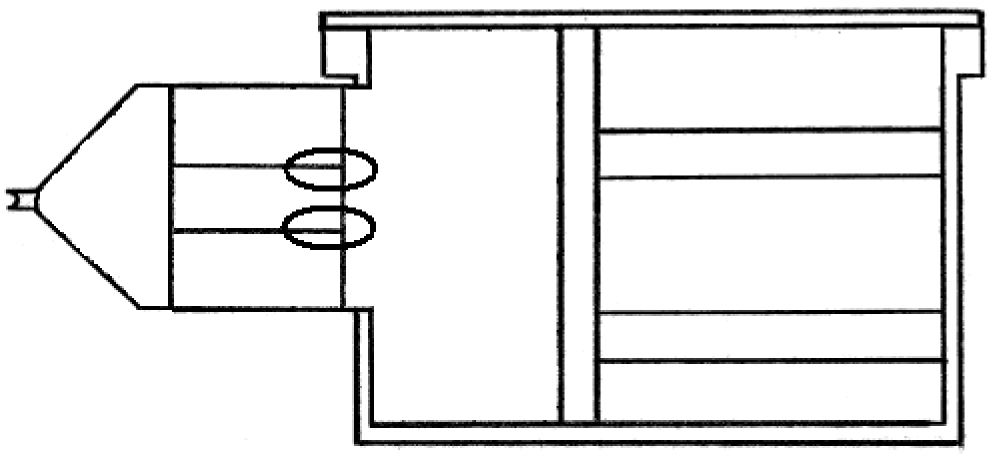


Figure 7-8. Location of the found cracks when using hardening grout. The locations are circled and orientated at the downstream interface between the core soil and grout plug.

7.2 Results from the usage of flexible grouts

Two successful repetitive tests were performed with a flexible grout. The tests are called test 5 and test 7.

7.2.1 Rate of seepage

The final rate of seepage through the filter box during when using flexible grouts is $18\,{}^{ml}\!/_{10\ \min}$. The variations of the seepage during testing can be seen in *figure 7-9*.

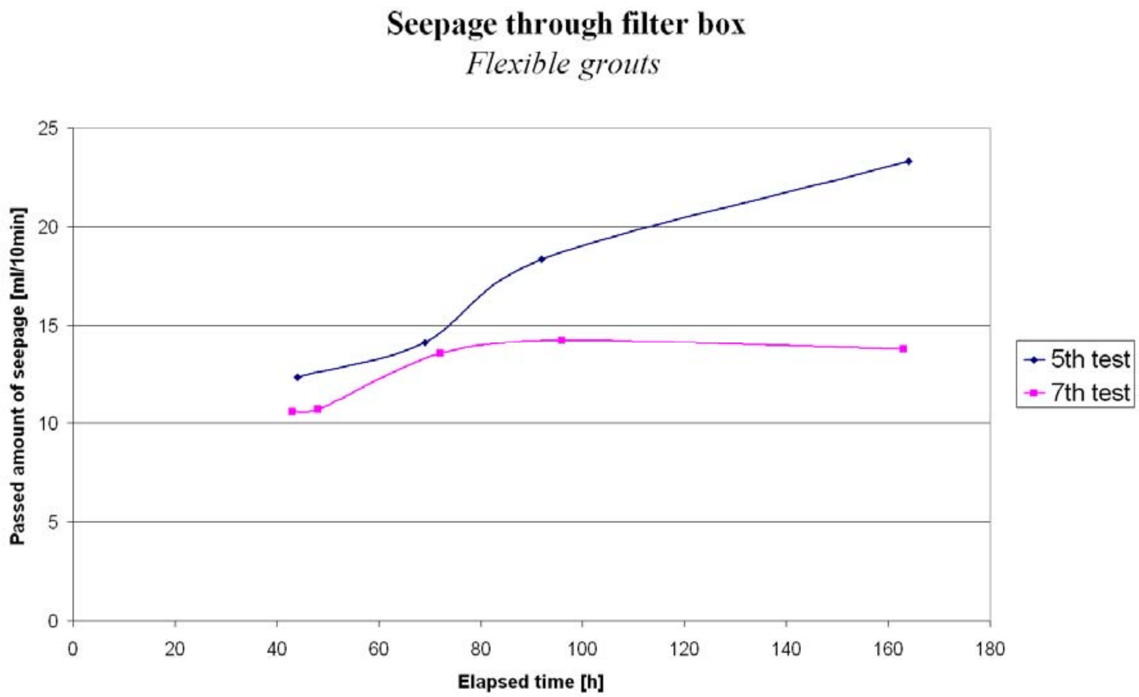


Figure 7-9. Measured permeability for test 5 and test 7, flexible grouts.

The rate of seepage during test 5 is constantly increasing while the rate of seepage for test 7 is uniform.

7.2.2 Water content

The water content demonstrated in *figure 7-10* is the water content attained after the testing.

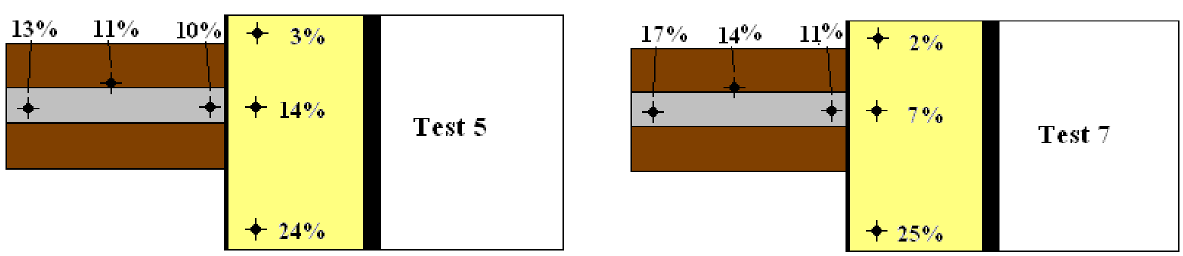


Figure 7-10. Measured water content in different areas of test 5 and test 7, flexible grouts.

7.2.3 Undrained shear strength

The results from the undrained shear strength test are found in *table 7-3* and *table 7-4*.

Downstream core soil			Downstream grout			Upstream core soil			Upstream grout		
m [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]
400	8,5	55,4	100	6,3	25,2	100	8,5	13,8	100	10,6	8,9
100	5,9	28,7	100	5,2	37,0	100	9,5	11,1	100	9,6	10,9
100	5,1	38,4	100	5,0	40,0	100	10,1	9,8	100	11,3	7,8
400	9,3	46,2	100	5,8	29,7	100	8,3	14,5	100	10,2	9,6
100	5,4	34,3	100	5,1	38,4	100	8,8	12,9	100	8,9	12,6
		40,6			**34,1**			**12,4**			**10,0**

Table 7-3. Undrained shear strengths after test 5, flexible grout.

Index for test 5, given by formula **(21)**:

$$\begin{cases} I_{downstream} = 0{,}84 \\ I_{upstream} = 0{,}81 \end{cases}$$

Downstream core soil			Downstream grout			Upstream core soil			Upstream grout		
m [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]	***m*** [g]	***i*** [mm]	***T*** *fu* [kPa]
100	5,0	40,0	100	5,9	28,7	100	9,1	12,1	100	13,0	5,9
400	8,0	62,5	400	9,4	45,3	100	10,2	9,6	100	13,3	5,7
400	7,2	77,2	400	9,2	47,3	100	9,8	10,4	100	14,2	5,0
100	5,5	33,1	100	7,0	20,4	100	9,0	12,3	100	13,9	5,2
400	8,2	59,5	100	5,5	33,1	100	9,9	10,2	100	13,3	5,7
		54,4			**34,9**			**10,9**			**5,5**

Table 7-4. Undrained shear strengths after test 7, flexible grout.

Index for test 7, given by formula **(21)**:

$$\begin{cases} I_{downstream} = 0{,}54 \\ I_{upstream} = 0{,}61 \end{cases}$$

7.2.4 Dissection photos

Photos taken after dismantling the filter box. Photos of both test 5 (*figure 7-11* to *figure 7-12*) and test 7 (*figure 7-13* to *figure 7-14*) can be viewed.

7.2.4.1 Photos from filter box test 5, flexible grout

Figure 7-11. View of downstream side after test 5, flexible grout.

In *figure 7-11* the location of the flexible grout can be seen. A small circle was discovered while excavating the downstream surface of the tube. However, no cracks where found at the interface core/grout. A further close-up of the photo can be seen in *figure 7-12*. The created circle is an imprint of the flexible grout against the downstream filter.


Figure 7-12. Close-up view of core soil/grout after test 5, flexible grout.

Figure 7-12 shows a close-up of the grouted part. The grout is found in the centre of the photo. The imprint of the flexible grout plug can be seen but no cracks are found.

7.2.4.2 Photos from filter box test 7, flexible grout

The second test of hardening grout was a repetition of test 5. The similarities between the two tests are clear.


Figure 7-13. View of downstream side after test 7, flexible grout.

Figure 7-13 shows the downstream side of the tube. The grout can be seen if the location of it is known (in the centre of the photo). No cracks where found.

Figure 7-14. Close-up view of core soil/grout. Test 7, flexible grout.

Figure 7-14 shows a close-up of the grout. No cracks where found and the core and grout is unified.

7.3 Discussion about the results

The filter box has been testing how the original core soil cooperates with an intrusion of a new material (grouts). Two types of grouts have been tested, hardening (test 4 and test 6) and flexible (test 5 and test 7). The results of the usage of these grouts will be discussed in the next segment of this chapter.

7.3.1 Discussion about the rate of seepage

The final rate of seepage after each test run with the filter box is lower if using hardening grouts than flexible. The attained rates of seepage are:

- $8{,}5 \, {}^{ml}\!/_{10\ \mathrm{min}}$ if using a hardening grout and;
- $18 \, {}^{ml}\!/_{10\ \mathrm{min}}$ if using a flexible grout.

The change of the rate of seepage through time is either increasing or uniform no matter the grout used. When performing permeability testing in *chapter 6*, the permeability was constantly diminished. A comparison between the permeability from the permeability testing and the rate of seepage from the filter box testing can be done since the permeability origins from the rate of seepage. The fact that the rate of seepage was not diminishing when performing filter box testing can be explained by the following factors:

- Since the testing was performed horizontally there might have been a consolidation of the core soil in the aluminium tube that created a void at the top of the tube, allowing a faster route for the seepage;
- The plywood in the retaining wall, supporting the filter sand inside the box might bulge backwards, diminishing the degree of compaction slightly;
- There was no particle movement between core and filter, clogging the latter.

The differences in the rate of seepage between filter box testing and the permeability from permeability testing is most likely explained by the usage of different setups (filter box vs. permeameter).

7.3.2 Discussion about water content

The water content shows a similar distribution inside the filter box. The used grout doesn't seem to affect how the water content is distributed. The water content is in general higher on the upstream side than at the downstream side. Even though the water content varies inside the core soil, it represents a real scenario. The water content inside a dam core might vary from location to location.

Hardening grout - Test 6 indicates a wetter filter and it is probable that there is a larger rate of seepage through the filter box. This is confirmed by the measured rate of seepage (*figure 7-1*). Another possible reason for the higher water content in the filter in test 6 is the extra test duration of two days.

Flexible grout - During test 5 the water content was higher in centre of the filter, which may indicate that the seepage was concentrated via the grout. This possible concentration may also be an explanation to why the permeability rises throughout the testing since grout 3 have a higher permeability than the core soil.

7.3.3 Discussion about uniaxial compressive strength

Uniaxial compressive strength tests were only performed on the final product of the hardening grout (plug of grout). The results from this provided a varying result of 21,00 MPa and 12,17 MPa. Yet another two plugs where created during the calibration process of the filter box. These grout plugs attained a compressive strength of 9.94 MPa and 11,99 MPa. This may indicate that the environment, where the grout hardens, disturbs the hardening process. The final attained strength is in any case much higher than the undrained shear strength of the core soil.

7.3.4 Discussion about undrained shear strength

The undrained shear strength seems to be linked to the water content. There is a difference between the undrained shear strength between the downstream and the upstream side. The water content is higher on the upstream while the undrained shear strength is lower. The water content on the downstream side is lower while the undrained shear is higher. This is however not surprising since a high water content in a soil weakens its strength.

The average undrained shear strength of the core soil should be between 50 – 100 kPa (Stål et al 1984). The average uniaxial compressive strength of the sand/cement plug is around 15 MPa. Thus, the shear strength of the plug is roughly 170 times greater than the original core soil with an index of 0,004 – 0,006. This index is supposed to be as close to one as possible if a resemblance is desired. When using flexible grouts, the index is between 0,6 and 0,8. These latter values are indicating a grout with basically the same shear strength as the original core

7.3.5 Discussion about photos

There is one main difference between the usage of hardening- and flexible grouts. If hardening grouts are used, cracking occurs at the downstream side of the tested core soil/grout. If flexible grouts are not used, cracks do not appear. The two types of grouts were tested with the exact same procedure. From this point of view the hypothesis:

"Flexible grouts are better suited than hardening if used in an embankment dam"

is verified.

7.3.5.1 Discussion about photos of hardening grouts

The main possible explanation why cracks do occur when using hardening grout is the shrinkage of the sand/cement grout. When cement hydrates it undergoes shrinkage since the volume of water is decreased when reacting with the cement. If considering the final density of the sand/cement of about 1,92 t/m^3 this suggest a final water content of the sand/cement grout, expressed in kilograms of water/m^3 of grout, of about $400\ kg_{H_2O}/m^3$.
If this value is used in combination with the expression from (Ljungkrantz et al 1994):

$$\varepsilon_{s0} = \left(\frac{W}{215}\right)^3 \times 10^{-3} \tag{22}$$

Where in **(22)**:

$$\begin{cases} \varepsilon_{s0} = \textit{reference shrinkage} \\ W = \textit{water content of the sand / cement} \quad \left[{}^{kg}\!/\!{}_{m^3} \right] \end{cases}$$

The reference shrinkage, ε_{s0}, = 0,64%. If this value is combined with the diameter of the grout plug of about 50mm, the final reference shrinkage of the sand/cement plug could be expected to reach a distance of 0,3mm. As seen in *figure 7-4*, the observed cracks had a width of about 0,5mm.

However, the shrinkage of the concrete alone does not explain the appearance of the cracks. Since no cracks were found on the upstream side, only on the downstream side there must be another factor involved. If viewing the water content of the core soil it is found that it is higher on the upstream side than the downstream side for all the test runs. The lower water content on the downstream side may cause the core soil to exhibit false cohesion since it is not saturated. There is both water and air inside the soil structure. Since air bubbles are created, their surface tension adhere the core soil inside the aluminium tube together and thus creating a stiffer soil. If the core soil in other hand is saturated, it will become looser.

If the influence false cohesions raise, the soil will become less prone to consolidate. Since the water content is lower at the downstream side of the filter box, where the cracks were found, the false cohesion will become higher than at the upstream side. The false cohesion may cause the core soil not to heal the opened boundary between the sand/cement plug and core soil as the plug shrink. On the upstream side of the core soil the water content is higher, resulting in a lowered influence of false cohesion. The lower false cohesion permits the core soil to heal the boundary between the sand/cement plug and original core soil.

When performing test 6 with a hardening grout, the observed cracks were smaller than the cracks after test 4 (compare *figure 7-5* with *figure 7-7*). The densities of the sand/cement grout plugs were basically the same so it is possible to say that they suffered equal shrinkage. The water content of the filter, facing the sand/cement grout plug was however doubled (11% for test 6 and 6% for test 4). If the water content in the filter is doubled it is likely to believe that the water content of the neighboring core soil also is higher. Higher water content make the core soil less cohesive thus, the magnitude of the cracks should be smaller.

The cracks were in particular found at the upper side of the sand/cement grout plug (above the plug vertically) on the downstream side of the aluminium tube. This does also speak in favor of the presented theory. Due to possible consolidation of the core soil, the sand/cement grout plug joins the downward movement because of gravitational forces. The presence of false cohesion in the overlaying soil cause this part of the core soil not to follow the movement of the plug.

The cracks may also have been caused by a concentration of the seepage flow through the filter box. Since the hardening grout plug is virtually impervious, no seepage will surge it. This should instead redirect the seepage around it and there is a possibility that the seepage will flow along its surface. If such event occurs the core soil is at risk at being subjected to a concentrated leak.

The final possibility for the cracks to occur is backward erosion. If there is a higher rate of seepage along the surface of the hardening plug, the shear forces provided by the water acting on the finer particles of the core soil will become larger. Fines may start to move and escape the aluminium tube out in the filter. This should however be considered less likely since the filter was designed not to allow the escape of particles from the core soil.

7.3.5.2 Discussion about photos of flexible grouts

No cracks were visible at any location and the grout seemed to be a natural part of the core soil. The lack of cracks and the slight higher rate of seepage were the two main characteristics separating the usage of flexible grout from the usage of hardening.

The first possible reason for cracks to not occur is the swelling ability of the clay, constituting 9% of the flexible grout. Since the clay was wetted within 15 minutes of application inside the original core soil, its swelling ability may have caused the entire flexible grout plug to swell. This is a desirable characteristic of a grout. If the grout swells it heals eventual openings between the flexible grout plug and the core soil. The clay mineral used is unknown but swelling was observed during the permeability testing (see *chapter 6.2.1*).

The second possible reason for cracks to not occur is the resemblance in strength and permeability characteristics between the flexible grout and core soil. Since the core soil and the flexible grout possess basically the same strength and permeability characteristics, they will react similar if subjected to variations. This variation can have the form of shifting water content thus; the flexible grout should be able to follow the movements such as consolidation of the core soil. This is an opposite reaction compared with the usage of flexible grouts.

The third possible reason for cracks not to occur is that surging of the flexible grout plug was allowed. The seepage could surge any part of the core soil inside the tube without any kind of forced redirection because of the plug. The similarities in permeability and shear strength should furthermore make it more difficult for boundary flow to appear along the interface of flexible grout plug and original core soil.

The fourth possible conclusion, internal erosion, should also be excluded. The primary possible reasons for this are two:

1. The filter has been designed not to allow any form of particle transport;
2. No damaging such as formation of cracks where found at any location after each test run with a flexible grout.

8 Conclusions and Future Work

8.1 Introduction

Remedial injection grouting has been studied in this thesis. The main focus has been to bring the theoretical knowledge of embankment dams, internal erosion and grouting techniques into the laboratory. In the laboratory different criterions of the grout have been tested.

Remedial methods of today often include a grout that has a certain amount of cement. The cement bound grout, or hardening grout, is assumed to have a similar impact on the dam as subjected to an embankment dam at the interface dam/concrete structures. These interfaces give a higher risk for development of cracks and are notoriously more sensitive against internal erosion. This has lead to test whether it is possible or not to use grouts inside a damaged embankment dam core, where the grout is constituted solely of soil particles and water. These non-hardening grouts have been named flexible grouts.

Flexible grouts have been successfully used in real embankment dams. At Bennet dam in Canada, a flexible grout was successfully used with a compaction grouting technique. Bennet dam suffered from internal erosion in its central core and exhibited a sinkhole on its crest. The used grout was designed to resemble of the original core soil and contained soil and water only.

Two hypotheses have been set up for the laboratory work:

> ***“Designing the particle size distribution curve of a flexible grout will reduce the risk of internal erosion”***.

The first hypothesis has formed the basis of the ***permeability testing***. The literature study reveals that the particle size distribution curve of a soil influence its proneness to internal erosion. If a flexible grout is used it must be verified that it remain inside the dam at its injected location over a longer period of time. Its particle size distribution curve may be designed so the grout is more stable in terms of resistance to internal erosion, without any interference of cement.

The second hypothesis is:

> ***“Flexible grouts are better suited than hardening if used in an embankment dam”***

The second hypothesis has been useful when designing and evaluating the tests performed with the ***filter box***. The filter box will allow testing of how either a hardening grout or a non-hardening grout affects a core soil if injected.

All laboratory work has been limited to the fact that the dam core has been damaged by internal erosion and a pipe has formed. It has also been limited to the fact that the tested grout has been assumed to be in its intended place, inside the core soil. Studies of how to inject have to this point not been performed.

8.1.1 Conclusions of the permeability testing

Three types of flexible grout where tested in the permeability testing. Permeability testing was chosen in order to surge the grout since if this type of grout is used in a dam, it will be subjected to erosive forces similar to the original core soil. If the particle size distribution curve of the flexible grout is designed, the influence of the erosive forces by the surge may be diminished. Evaluations after the permeability testing series validated that: Design of the particle size distribution curve of the grout may decrease the risk of material transport within the grout.

The sieve analyses performed after each test run revealed a decrease of material transport between the tested grouts, grout 2 and grout 3. Grout 2 exhibited a change in its soil structure when surged while the surge on grout 3 had a lesser impact. Even though similar conditions where met, there was a large difference between the two series. Grout 1 proved to be impervious and had characteristics similar to clay. Testing of grout 1 was therefore abandoned.

Grout 2 $\left(k = 1{,}6 \times 10^{-5}\, {}^{m}\!/\!{}_{s}\right)$ had a higher permeability than grout 3 $\left(k = 5 \times 10^{-7}\, {}^{m}\!/\!{}_{s}\right)$. When performing permeability tests of grout 2 its permeability was above $1 \times 10^{-6}\, {}^{m}\!/\!{}_{s}$. This value represents the theoretical value where clay particles start to erode. The permeability of grout 3 was however lower than $1 \times 10^{-6}\, {}^{m}\!/\!{}_{s}$. The only difference between the two soils was their original particle size distribution curve. Therefore, the appearance of the particle size distribution curve of the grout had a large influence on the attained permeability.

The water content of the tested grouts seems to correspond to the distribution of material within the tested grouts. This was in particular noticeable for grout 2. Since the tested grout was divided into slices, six different water contents could be extracted from each test run. Grout 2 exhibited higher water content at its centre part inside the permeameter, compared with its end slices. If the same centre part was compared with the result from the sieve analysis, a higher degree of fines could be measured. Hence, if the water content throughout the tested grout sample is evenly distributed, it can be said that its particle size distribution is also evenly distributed. Grout 3 had a more even distributed water content than grout 2.

The setup of the permeability didn’t allow any fines to be washed out from the permeameter. The fines in the grouts where either re-arranged or caught in the lower filter stone.

The usage of histogram and numerical analyses shows clearly that the distribution of the tested grouts before- and after surge changes depending on the original particle size distribution curve. The difference of the original particle size distribution curves between grout 2 and grout 3 was minor but it seems to have had a large impact on the expected stability of the grout structure. Grout 3 had a slight higher content of clay compared to grout 2.

It seems possible to obtain a grout that possesses no hardening characteristics to remain durable if grouted in a dam core. It also seems possible to obtain geotechnical characteristics of a flexible grout that are closely related to the characteristics of the original core soil. The final tested grout, grout 3, exhibited small variations in its structure before/after surging. If these variations are small the grout is less likely to separate, hence more able to withstand internal erosion over a longer period of time.

8.1.2 Conclusion of the filter box testing

The filter box was constructed to model a finished grout situation in a dam. The filter box allowed testing to be performed horizontally. It was also possible to test both hardening and flexible grouts. The filter box consists of three main parts: A box for filter sand, an aluminium tube fore core soil/grout plug. The final part is a cone head that distributes the incoming water over the entire cross section area of the core soil inside the aluminium tube. The filter box allows studies of a how a grouted core is affected by seepage. The experiments with both hardening and flexible grouts where performed with the exact same procedure. This allowed a direct comparison of how these two types of grout affected the original dam core.

The water content differs between the upstream- and downstream side of the filter box. This difference was noted when using both types of grouts as a remedial method inside the aluminium tube. The water content in the core soil at start of each test run was 10%. After each test run the water content of the core soil rose but to a higher degree at the upstream side. This resulted in a saturated zone at the upstream side of the core soil inside the aluminium tube. The lower water content of the core soil at the downstream side caused the soil to undergo false cohesion.

The uniaxial strength of the hardening grout is far greater than the strength of the core soil. If similarities between the used grout and original core soil are desired, this will be difficult to attain with a hardening grout. When using flexible grouts the difference was almost eliminated. This should allow the flexible grout to cooperate with an embankment dam when, and if it is subjected to various forms of movements.

The photos taken after each test run with the filter box revealed that cracks occurred between the core soil and grout if hardening cracks where used. The formation of cracks did not occur when using flexible grout. The observed cracks after the test with hardening grout only appeared at the downstream side where the water content of the core soil where lower. Since the hardening grout shrink when hydrating, the un-saturated core soil with lower water content exhibit false cohesion and was not able to heal the cracks between the core soil and hardening grout plug. When using flexible grout the reaction was inversed. The used clay in the flexible grout swelled when used thus; no cracks between the flexible grout and core soil where created.

If considering the second hypothesis, flexible grouts are better suited than hardening if used in an embankment dam, is verified according to the filter box testing. Cracks were observed when using hardening grouts while the usage of flexible grouts provided no detectable damages.

8.2 Further work

To this date, no studies regarding the actual injection of the grout have been examined. This will therefore constitute the future work.

The demands put upon a grouting procedure can be summarized into five points:

1. Reaching the intended location of damage;
2. Initial injection where the flexible grout stops at the start of the pipe, at the interface downstream filter/core soil;
3. Fill the pipe with the flexible grout;
4. Obtain durability of the injected flexible grout;
5. Avoid putting excessive damages upon the dam.

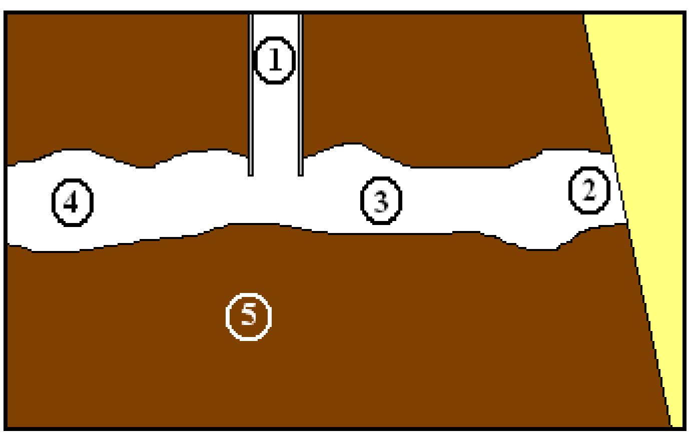


Figure 8-1. Conceptual plan of grouting implementation.

1 – This point will not be examined. When performing future research the localization and drill work to the damaged part will be assumed done. Experiments will therefore not include this.

2 – Initial injection of a pipe is important to examine. Inside a pipe there is a higher rate of seepage. When injecting a flexible grout into a pipe, its water content will rise and therefore undergo separation. It will be examined how the velocity of the seepage influence the separation of the grout. It will also be examined how the appearance of the downstream filter affects the durability of the injected grout. If the downstream filter has been washed out from its fines it will become coarser. If its coarseness is too high, grout 3 will not be sufficient in order to fulfil the filter criterion. If these criterions are not fulfilled, the grout will not remain durable inside the dam. Therefore, test will be done with coarse filters and injection of a flexible grout in two steps. The first step with a coarser flexible grout and a second step with a finer grout, supposedly grout 3.

3 – When the initial grouting is done, the pipe must be filled with a flexible grout. In this part it will become important to study the grout pressures. If grout 3 is used in order to fill the pipe it must be examined how far from the end of casing the grout may travel. A higher grout pressure will ensure that the grout travels farther from the end of casing but too high pressures can rupture the core soil. Therefore the needed distance between each grout outlet (end of casing) will be examined, and how the applied grout pressure affects these distances.

4 – When the pipe is filled it must be examined whether the injected grout will remain durable inside the pipe. This will be done by examine the particle size distribution and permeability of the grouted pipe.

5 – This point will not be given to much attention separately. It is closely linked to the work performed in point 3.

8.2.1 Future laboratory setup

These experiments are planned to be performed with plastic tubes. At the end of the tube, an artificial filter will be created. On the other end of the tube, a desired water pressure (velocity of seepage) will be applied. Injection of a flexible grout will then be done at the top of the plastic tube. The grout will seal the tube from the downstream side and backwards and it will be examined how the flexible grouts initial particle size distribution curve is affected by the velocity of the seepage. The initial grouting will be found at the downstream side of the tube, neighbouring the filter.

After the initial grouting, a secondary grouting will be done to fill the plastic tube. At this point the influence of grout pressures and distance between grout holes are examined.

The final step is to evaluate the attained durability of the finished grout situation. The plastic tube will be cut into slices and the flexible grout from each slice will be evaluated by extracting their particle size distribution curves. The interface grout/filter will be examined if it fulfils the filter criterion. The final permeability of the grouted tube should furthermore drop below $1\times10^{-6}\,{}^{m}\!/_{s}$. This will lower the flexible grouts proneness to suffer from internal erosion.

References

Alexanderson, Helena. (2004). *fil.dr. The institute of Physical geography and quaternary geology. University of Stockholm.* Written correspondence.

Bonazzi, Darius. (1965) *Alluvium grouting proved effective on alpine dam.* Journal of the soil mechanics and foundations division. Proceedings of an American Society of Civil Engineers

Bradley, Jeff. Bruce, Donald. Ferguson, Keith. Poulos, Steve. Talbot, James. Vrymoed, John. *Center Hill Dam consensus report.* August 30 2007.

Brantberger, Martin. *"Metodik för förinjektering i hårt berg".* Licentiate thesis, division of soil and rock mechanics KTH, Stockholm 2000

Cernica , John N., (1995) *Soil Mechanics.*

Charles, J. A., Tedd, P., Hughes, A. K. *and* **Lovenbury, H T.** (1996) *Investigating embankment dams, A guide to the identification and repair of defects*

Chen, Y.J. (1982) *Grouting for sealing the cohesive fills of embankment dams.* 14:th ICOLD meeting in Rio de Janeiro 1982.

Dahlin, Olle *and* **Fagerström, Hans.** (1987) *Injektering av sättningsskada i Hällbydammen.* Seminar about dams 20-21 November 1986, KTH.

Eklund, Daniel. (2005) *Introduktion till injektering av fyllningsdammar*

Eklund, Daniel. (2005) *Penetrability due to filtration tendency of cement based grouts.* Doctoral thesis, soil- and rock mechanics, Stockholm 2005

Ekström, Tomas *and* **Pusch, Roland.** (2001) *Bentonit i fyllningsdammar*. ELFORSK report 01:15.

Ericsson, Joacim *and* **Jender, Mattias.** (1998) *Dokumentation och undersökning av skador i Lövödammen.* Master thesis, Luleå university of technology, Department of Civil, Mining and Environmental Engineering.

Fagerström, Hans. Wiesel, Carl-Eric. (1972) *Permeabilitet och kapillaritet. Förslag till geotekniska laboratorieanvisningar, del 8.*

Fell, Robin, Macgregor, Patrick, Stapleton, David *and* **Graeme Bell** (2005) *Geotechnical engineering of dams.*

Garner, Steve. 09-11-2006. ICOLD Workshop in Stockholm.

Garner, S.J. Warner, J. Jefferies, M.G. Morrison, N.A. *A controlled approach to deep compaction grouting at WAC Bennet dam.* 2000.

Gullberg, Ingvar E. (2000). *Swedish – English specialized technical dictionary.*

Göthlin, Caroline. (2005) *Alternative reparation material based on limestone filler for grouting in embankment dams – Evaluation of mechanical- and early leaching problems.* Master thesis at the Royal institute of technology, Stockholm 2005.

Hansson, Pär. (1995) *Injektering av jorddammar – Problemformulering och litteraturinventering.* Vattenfall Research and Development.

Hansson, Pär. (1999) *Travel report, Bennet dam, BC Hydro, Canada.* Teknikbevakning av injekteringsarbeten. ELFORSK report 99:37.

Hansson, Pär. (2005) *Injektering av fyllningsdammar – Storskaliga modellförsök.* Vattenfall Research and Development

ICOLD 1994

ISO/TS 17892-4. (2004) *Geotechnical investigation and testing – Laboratory testing of soil – Part 4: Determination of particle size distribution.*

Jefferies, M.G. *Nor-sand: A simple critical state model for sand.* Géotechnique 43, p91 – 103. 1993.

Johansson, Sam. Bartsch Maria. Landin, Ola. Barmen, Gerhard. Dahlin, Torleif. *and* **Ulriksen, Peter.** (1995). *Utrednings- och utvecklingsprojekt med anledning av Flödeskommitténs riktlinjer. Nyare metoder för tillståndskontroll av fyllningsdammar.* VASO dam committee report nr 21

Johansson, Sam. (1997) *Seepage monitoring in embankment dams.* Doctoral thesis Royal Institute of Technology, Stockholm 1997.

Johansson, Sam *and* **Farhadiroushan, Mahmoud.** (1999) *Fiber-optic measurements at the Lövön dam.* ELFORSK report 99:36.

Johansson, Sam. Farhadiroushan, Mahmoud, Parker. T. *and* **Watley, D.** (2003) *Distributed strain measurements for embankment dams – Laboratory Tests, Installation, and Initial Monitoring Experiences,* ELFORSK report 03:19.

Johansson, Sam. Friborg, Johan *and* **Dahlin, Torleif.**.(2005) *Seepage monitoring by resistivity and streaming potential measurements at Hällby embankment dam 1996 – 1999.* ELFORSK report 00:15.

Johansson, Sam *and* **Nilsson, Åke.** (2005) *Internal Erosion Detection at the Rösvatn Test Site – Experiences from blind tests using Resistivity, Self Potential, Temperature and Visual inspection.* ELFORSK report 05:42.

Kakuturu, Sai *and* **Reddi, Lakshmi N.** (2006) *Mechanistic models for self-healing of core cracks in earth dams.* Journal of Geotechnical and environmental engineering ASCE July 2006 p 890-901.

Karlsson, Rudolf. (1981) *Consistency limits. Performance and interpretations of laboratory investigations, part 6.*

Karlsson, Rudolf. Hansbo, Sven. (1989) *Soil classification and identification. Performance and interpretations of laboratory investigations, part 2.*

Kenney, T. C. and **Lau, D.** (1985) *Internal stability of granular filters.* Canadian Geotechnical Journal 22, p 215 – 225.

Lagerlund, Johan. (2007) *Omfattande dammreparation vid Lilla Edet*. ELFORSK nyhetsblad februari 2007.

Lambe, T. William *and* **Whitman, Robert T.** (1979) *Soil mechanics – SI version.*

Ljungkrantz, Christer. Möller, Göran. *and* **Petersons, Nils.** (1994) *Betonghandbok – Material, utgåva 2.*

Nilsson, Åke. (1995) *Filter – Inventering och funktionsanalys. Utrednings- och utvecklingsprojekt med anledning av flödeskommitténs riktlinjer.* VASO dam committee report number 13.

Nilsson, Åke. Ekström, Ingvar *and* **Söder, Carl-Olof.** (1999) *Inre erosion i svenska dammar. Beskrivning och utvärdering av sjunkgropar.* ELFORSK report 99:34.

Pusch, Roland. (1973) *Densitet, vattenhalt och portal. Förslag till geotekniska laboratorieanvisningar.*

RIDAS (1994)

RIDAS (2004) *Presentation of RIDAS by* **Nilsson**, **Åke.** 2004-08-25 at NNCOLD:s Fagseminar 2004.

Rönnkvist, Hans. Presentation 12/9 – 2006 ICOLD workshop in Stockholm.

Rönnkvist, Hans. (2002) *An evaluation of the internal stability of embankment dams owned by Vattenfall AB.* Master thesis at the Royal Institute Of Technology, Stockholm.

Sherard, J. L. and **Dunnigan, L. P.** (1989). *Critical filters for impervious cores.* Journal for geotechnical engineering, ASCE. 115 (7) pp 927-947.

Sherard, James L., Woodward, Richard J. *and* **Clevenger, Stanley A.** (1965) *Earth and earth-rock dams. Engineering problems of design and construction.n*

Shuttle, Dawn. Jefferies, Michael. *Prediction and validation of compaction grout effectiveness.*

Shuttle, Dawn, Jefferies, Michael and **Members ASCE**. *Prediction and validation of compaction grout effectiveness*

Sjöström ,Örjan A. (1999) *Reparation av jorddammar genom injektering.* ELFORSK report 99:48.

Sobkowicz, John and **Garner, Steve.** *Anomalous pore pressures in earth dams.*

St-Arnaud, Guy. (1995) *The high pore pressures within embankment dams: an unsaturated soil approach. Canadian Geotechnical journal* Vol. 32, No. 5

Stål, Torbjörn. Wedel, Per. Avén, Sigurd (1984) *Geoteknik, handboken bygg.*

Svensson, Conny. (1996) *Conny Svenssons ingenjörsgeologiska exkursion*.

Tam, Joe G. (1996) *Grout expectations. Description of and applications for grouts used in the civil engineering field.*

Terzaghi, Karl. Peck, Ralph B. and **Mesri, Gholamreza.** (1996) *Soil mechanics in engineering practice – Third edition.*

Vattenfall (1988).

Warner, James P.E.. (2004) *Practical handbook of grouting. Soil, rock and structures.*

Warner, James. Jefferies, Michael and **Garner, Steve.** *Compaction grouting for sinkhole repair at Bennet dam.* Geotechnical special publication No. 120. Grouting and ground treatment – Proceedings of the third international conference

Warner, James. Jefferies, Michael and **Garner, Steve.** *Compaction grouting for sinkhole repair at WAC Bennet Dam.*

Watts, B. D., Gaffran, P. C., Stewart, R. A., Sobkowicz J. C. and **Küpper, A. G.** (2000) *WAC Bennet Dam – Characterization of sinkhole NO. 1.* Montreal 2000: 53d annual conference of the Canadian geotechnical society. Special session.

Watts, B. D., Gaffran, P. C., Stewart, R. A., Sobkowicz J. C. and **Küpper, A. G**. Montreal 2000. 53:e Annual conference of the Canadian geotechnical society

Widing, Svante. (1987) *Några allmänna synpunkter på damminjektering*. Seminar about dams. 20-21 of November, 1986, Royal Institute of Technology Stockholm.

Wiesel, Carl-Eric. Hansbo, Sven. Broms, Bengt. (1985) *Skjuvhållfasthet. Geotekniska laboratorieanvisningar, del 9.*

Windelhed, Kjell. (2001) *Jetinjektering. En intressant reparationsmetod för jorddammar.* ELFORSK report 99:48.

Wolski, W., Lipinski, M. J., Fürstenberg, A. and **Baranski, T.**. *Influence of internal erosion on safety of old dams.* International conference on geofilters, 3, Warsaw, 5-7 June 2000. Proceedings.

Websites used

http://connywww.tg.lth.se/exkursionstartmenus.html#anchor380315 - *Soils*

http://webmineral.com/ - *Mineralogy*

http://www.glossary.oilfield.slb.com/ - *Bentonite*

http://www.haywardbarker.com - *Injection grouting*

http://www.heidelbergcement.com/se/sv/cementa/ - *Cementa AB. Report Ultra fine 16 (UF 16)* August 2003

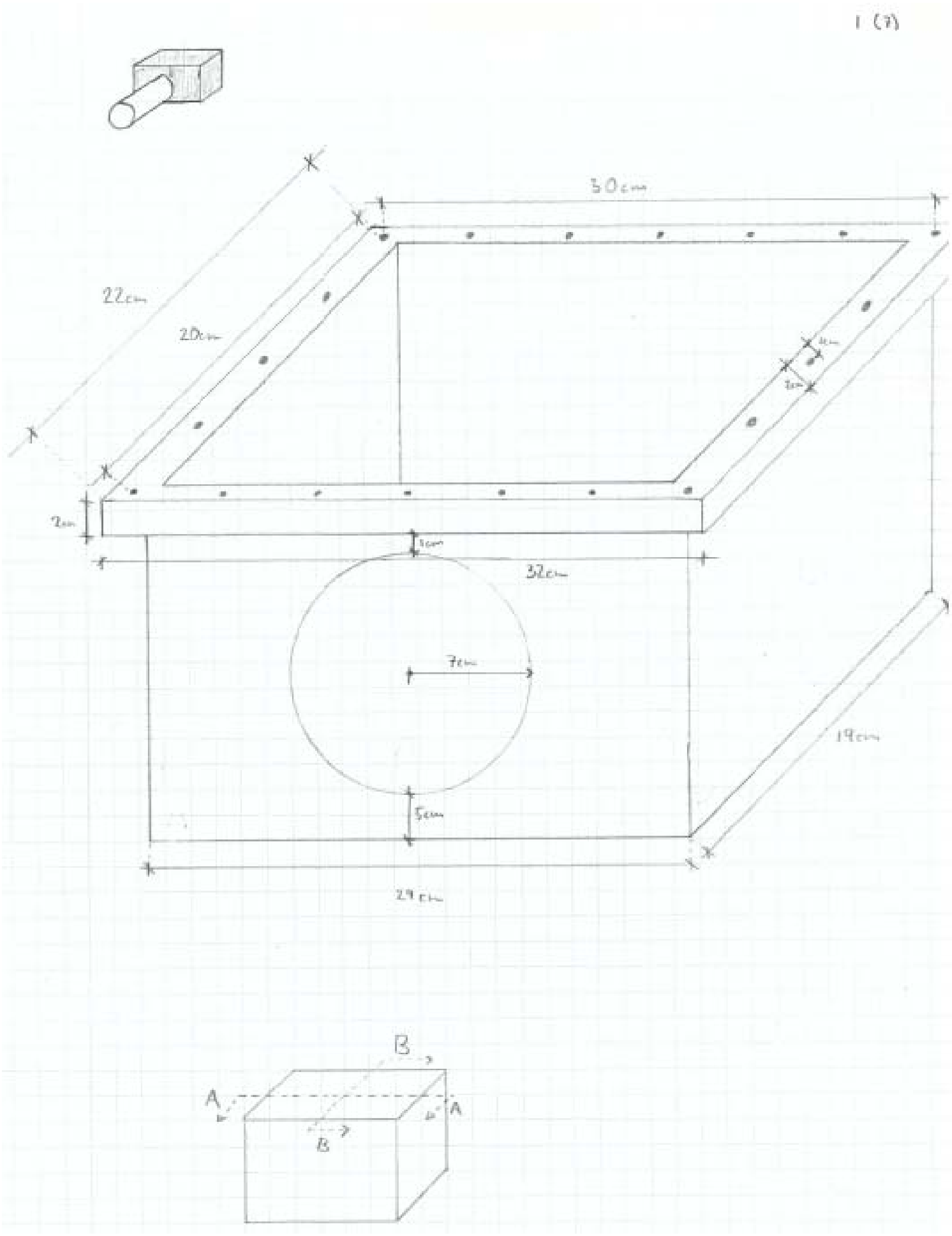
http://www.mooreandtaber.com/permeation_grouting.htm - *Permeation grouting*

http://www.portlandonline.com/CSO/index.cfm?a=dfdca&c=dajfg - *Tremie pipe*

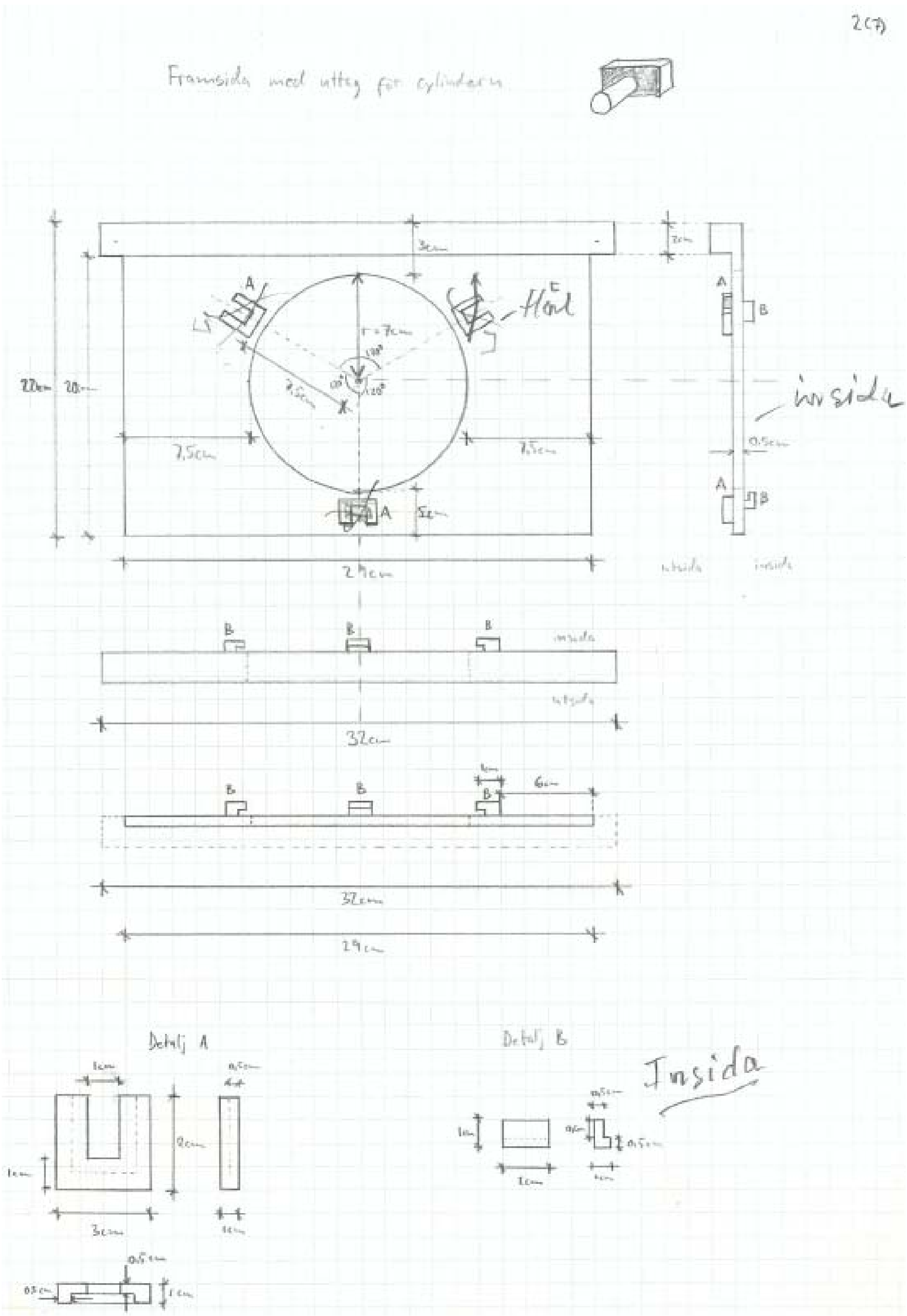
http://www.webelements.com/

Appendices

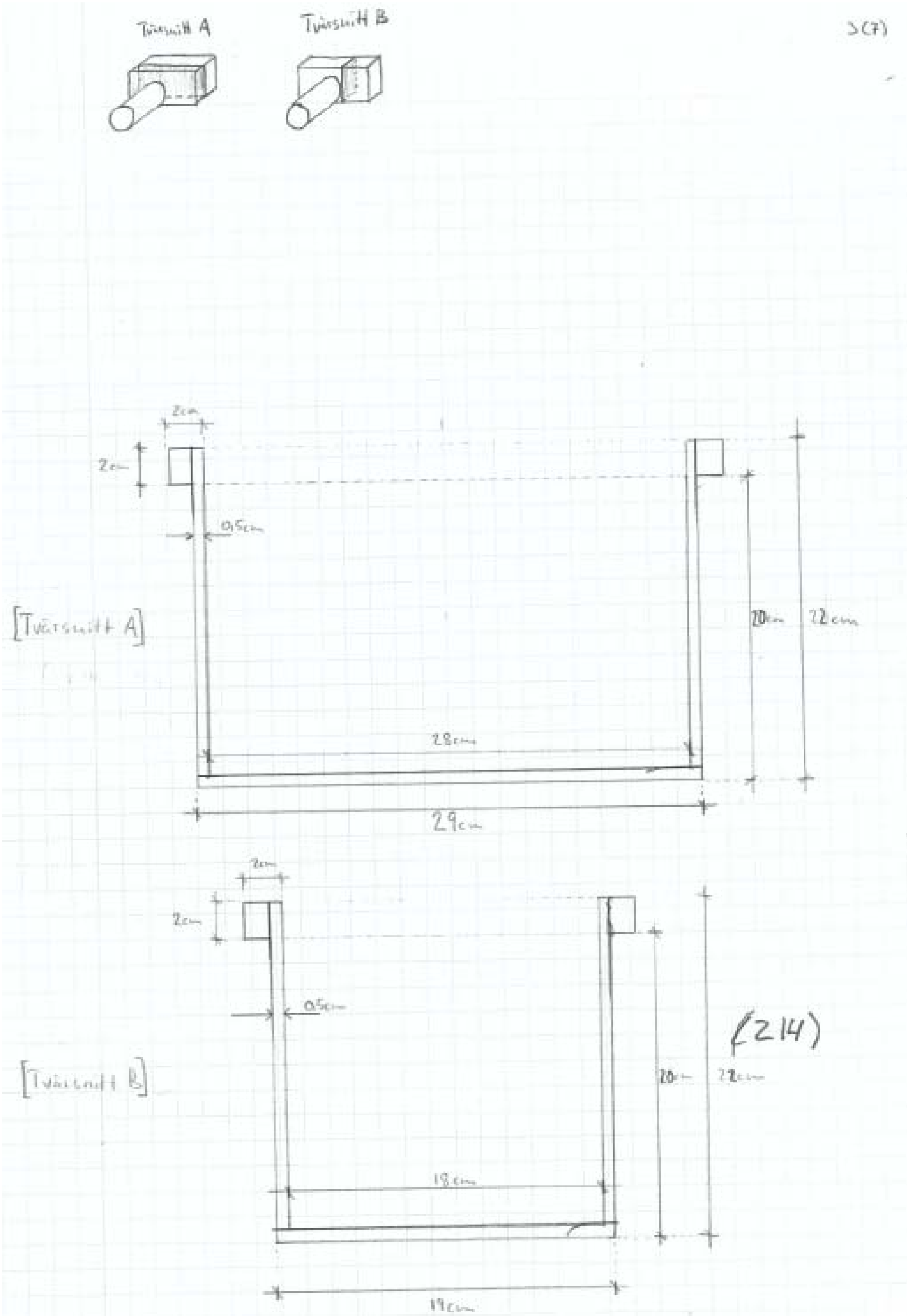
Appendix I - Blueprint of filter box



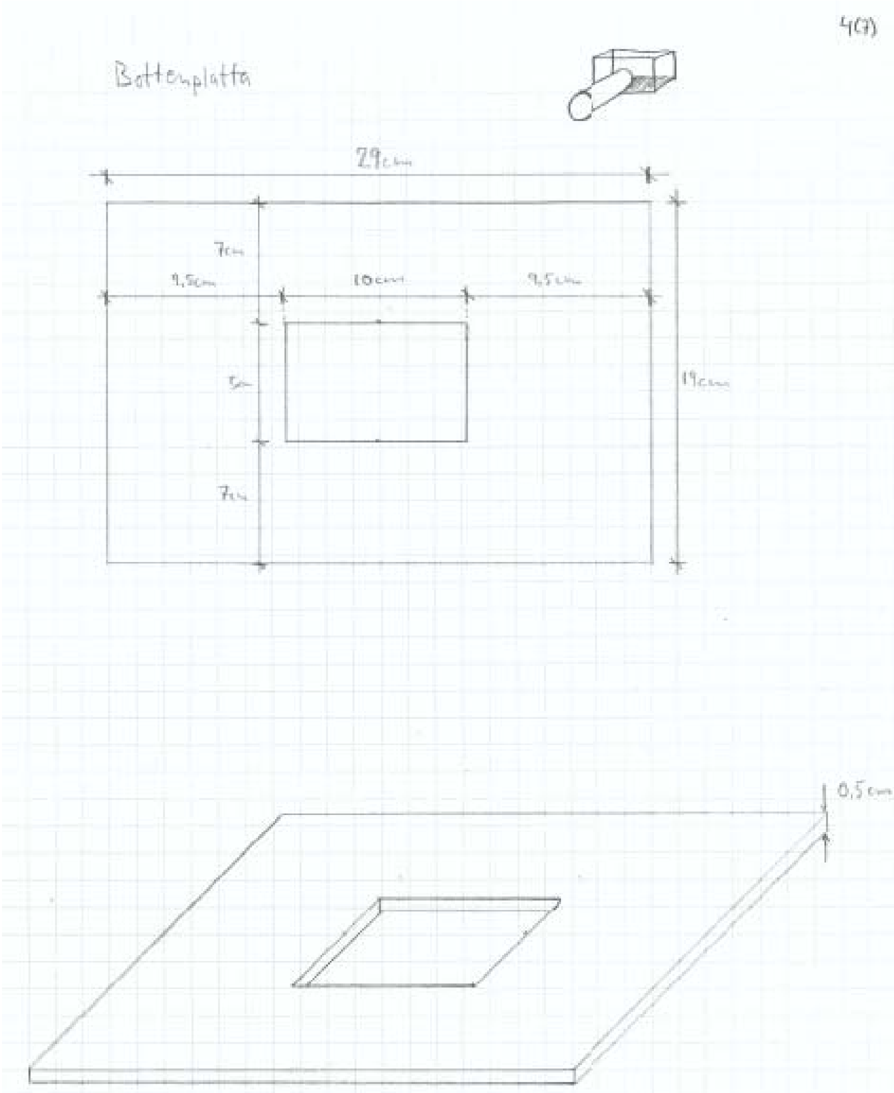
Appendix 1. View of the filter box.



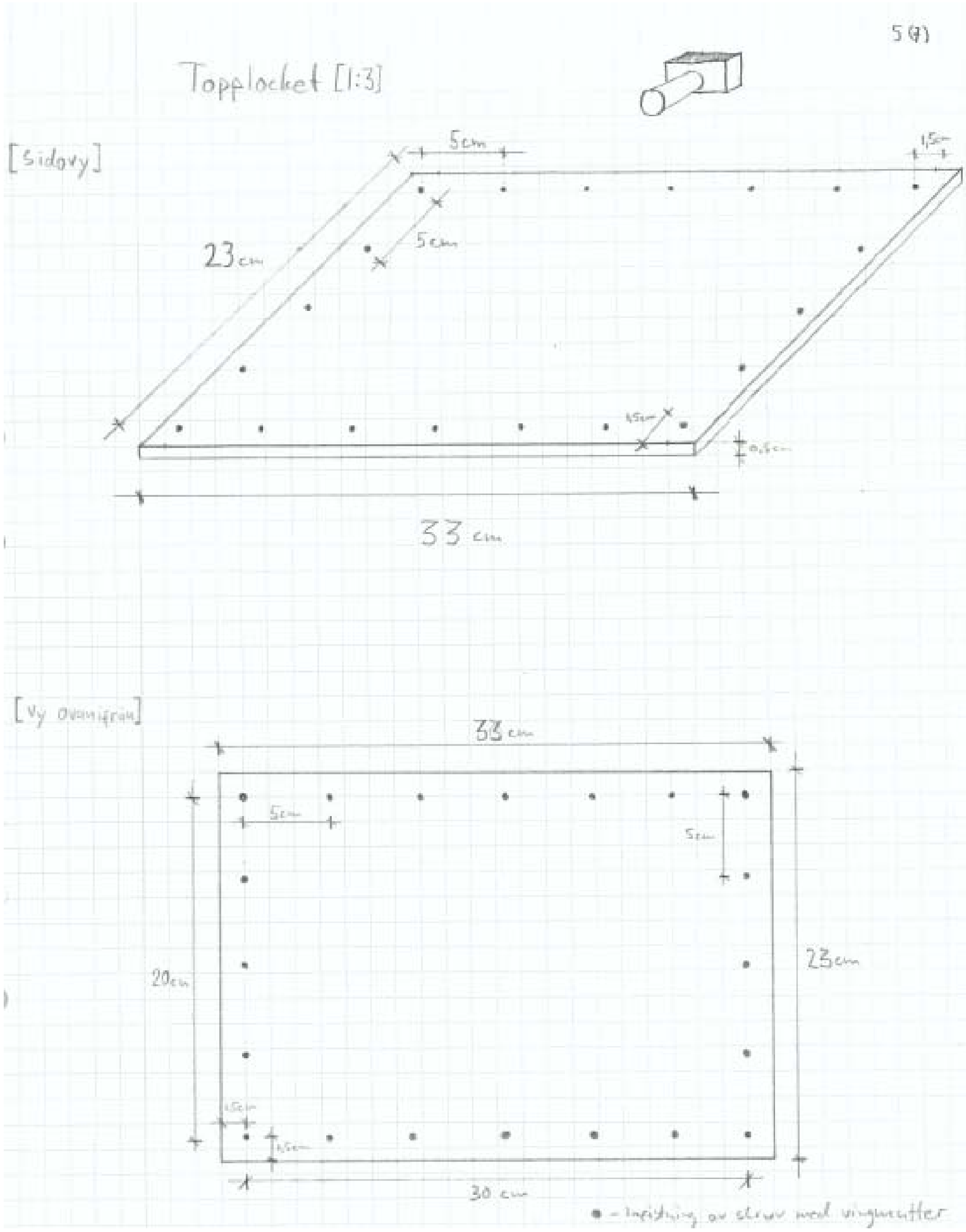
Appendix 2. Front view of the filter box.



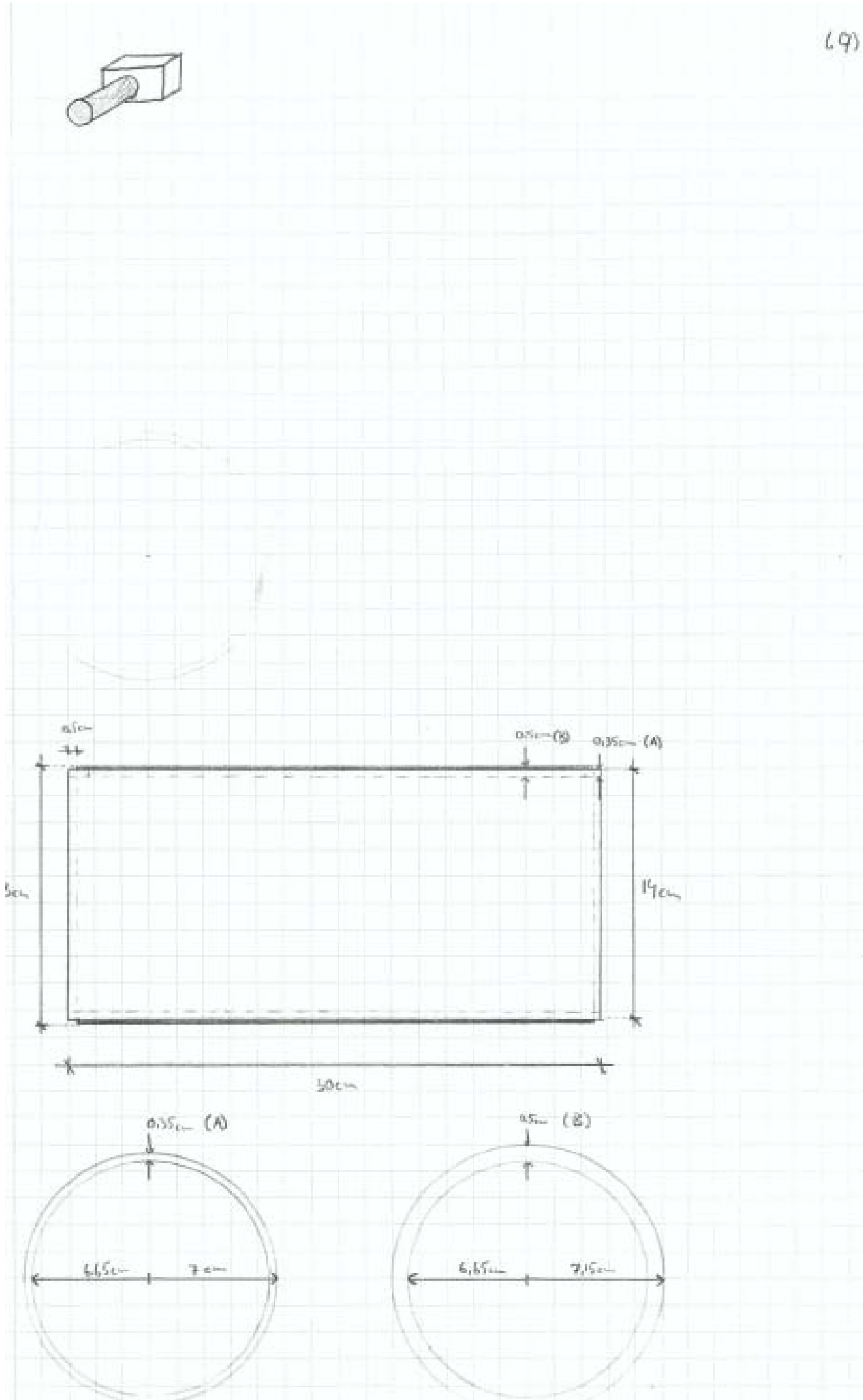
Appendix 3. Cross-section view of the filter box.



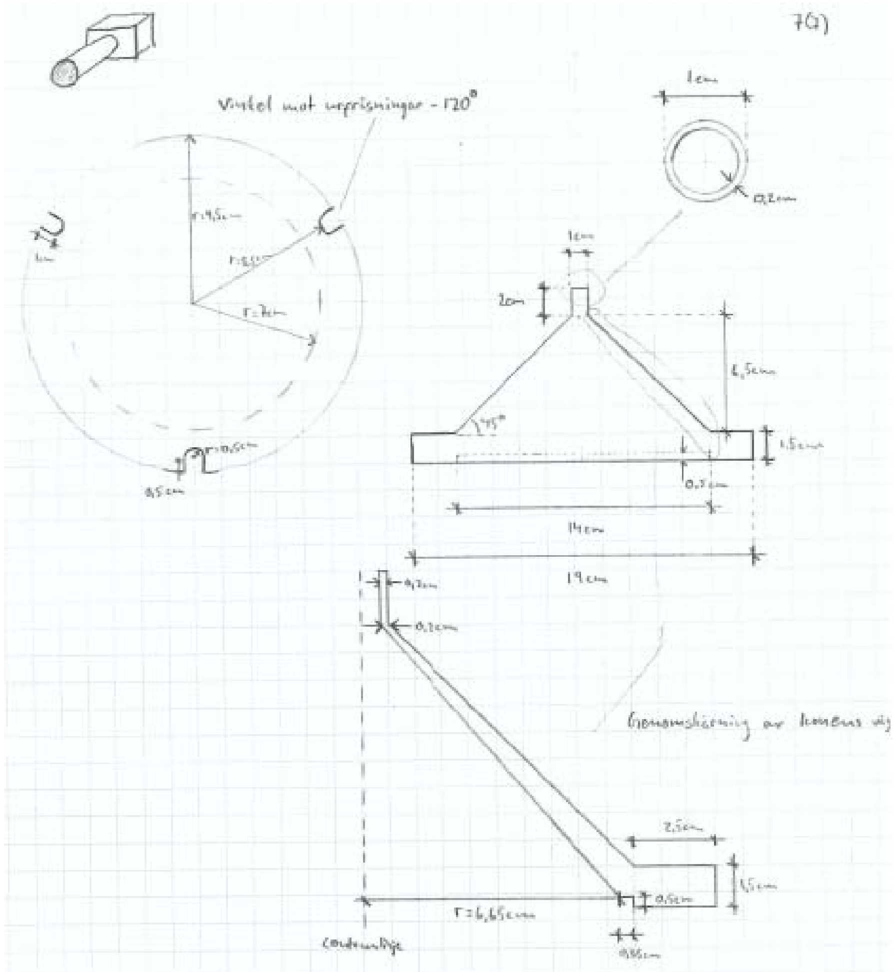
Appendix 4. View of bottom of the filter box.



Appendix 5. View of lid of the filter box.



Appendix 6. View of aluminium tube.. During the testing it was cut into two pieces with measurements of 100mm and 200mm respectively.



Appendix 7. View of cone head placed on the upstream side of the filter box.