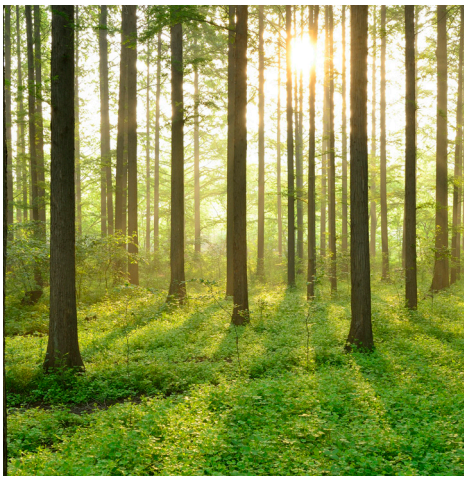
DESIGN OF GROUT CURTAINS

REPORT 2021:720



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Design of Grout Curtains

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Foreword

Ett nytt koncept – som även har en tydligare vetenskaplig koppling än de tidigare metoderna – för injekteringsridåer i nya dammar presenteras i denna rapport. Den har tagits fram av Suihan Zhang, KTH, tillsammans med hans handledare Fredrik Johansson i samband med Suihans doktorandprojekt ”Utveckling av designkoncept för injekteringsridåer”.

Projeketet genomförs inom kompetenscentret Svenskt vattenkraftcentrum och inleddes under 2018. Denna rapport har tagits fram för att ge verksamhetsutövare en bättre möjlighet att ta del av resultaten från doktorandprojektet och är ett mycket bra tilltag för att förbättra resultatspridningen.

Referensgruppen som är engagerad i projektet består av Jan Laue (LTU), Peter Viklander (Vattenfall), Sven Knutsson (LTU), Nadhir Al-Ansari (LTU), Åsa Burman (Skeab), Ingvar Ekström (Sweco Energy), Magnus Svensson (Fortum), Anders Isander (Uniper), Johan Lagerlund (VRD) samt Per Bolin (Keller).

Sammanfattning

Injekteringsridåer under dammar har tidigare i huvudsak utförts baserat på erfarenhet och empiri. I föreliggande rapport presenteras ett nytt designkoncept för injekteringsridåer under nya dammar. Designkonceptet tillhandahåller en analytisk lösning för dimensioneringen och baseras i huvudsak på existerande teorier om injektering i sprickigt berg. Konceptet möjliggör en bättre kontroll av dimensioneringen och förväntas kunna leda till en ökad kvalité på den utförda injekteringsridån samt förbättrad möjlighet till uppskattning av kostnader och tidsåtgång.

Injektering och utformning av injekteringsridåer under dammar är en vanlig åtgärd för att reducera bergmassans permeabilitet och minska läckaget och upptrycket. Utformningen av injekteringsridåer har i huvudsak baserats på erfarenhet och empiri ända sedan injektering började användas vid dammgrundläggning på 1800-talet. Vid utformningen av injekteringsridån baserat på empiriska metoder har fokus legat på att åstadkomma en tillräcklig tätning av bergmassan och en sluten ridå. De empiriska metoderna användes i stor utsträckning för att bl.a. bestämma lämpligt djup och bredd på injekteringsridån, hålavstånd, tolkning av utförda vattenförlustmätningar, lämpliga injekteringstryck och stoppkriterier, etc.

Även om de empiriska metoderna framgångsrikt har använts i flera projekt har de emellertid vissa nackdelar. Kvalitén på injekteringsridån blir i hög utsträckning beroende av erfarenheten och kunskapen hos de ingenjörer som är ansvariga för injekteringen. Olämplig användning av olika tumregler kan exempelvis leda till en konservativ utformning av injekteringsridån eller andra typer av problem. Bl.a. kan användning av stopkriterier såsom ”refusal criterion”, dvs. vid vilket flöde av bruk injekteringen ska upphöra, leda till onödigt långa injekteringstider utan att en motsvarande förbättring av tätheten uppnås, vilket kan leda till höga kostnader. Användning av höga injekteringstryck, vilket förespråkas i vissa sammanhang, kan leda till oönskade deformationer och upplyft av bergmassan och därmed en försämrad täthet. Vidare behandlar de empiriska metoderna endast indirekt risken för erosion av sprickfyllningsmaterial genom justering av vilka Lugeon värden som är acceptabla under olika förhållanden.

För att bättre hantera ovanstående begräsningar har ett analytiskt designkoncept för injekteringsridåer under dammar utvecklats, vilket presenterats i denna rapport. Konceptet baseras på nyligen utvecklade teorier för injektering av sprickigt berg samt principerna för observationsmetoden. I koncepter särskiljs dimensioneringen av själva injekteringsridån och utformningen av injekteringsarbetet. Dimensioneringskonceptet baseras på teorier rörande tolkning av sprickaperturer, spridning av injekteringsbruk över tid och bestämning av lämpliga injekteringstryck med hänsyn till upplyft av bergmassan. Det använder också stoppkriterier baserad på injekteringstid och injekterad bruksvolym. Hålavstånd bestäms baserat på beräknad bruksspridning och med hänsyn till erforderlig tjocklek på injekteringsridån med avseende på risken för erosion av sprickfyllningsmaterial samt en optimerad reduktion av upptrycket. Särskild vikt vid dimensioneringen läggs således på bestämningen av lämplig geometrisk

utformning av injekteringsridån, speciellt dess tjocklek vilken är kritisk med hänsyn till dess beständighet. Utformningen av injekteringsarbetet bestäms därefter för att uppnå den önskade geometriska utformningen.

Det är författarnas bedömning att det utvecklade konceptet för dimensionering av injekteringsridåer som presenteras i denna rapport ger ingenjörerna en förbättrad kontroll över dimensioneringen och injekteringsarbetet. Implementeringen av principerna för observationsmetoden i konceptet möjliggör också en stringent hantering av de osäkerheter som dimensioneringen är förknippad med. Förmågan att uppskatta tider och kostnader för injekteringsarbetena bedöms också förbättras.

Som de flesta andra dimensioneringskoncept som bygger på analytiska modeller har konceptet som presenteras i denna rapport begränsningar. Eftersom endast ett fåtal nya dammar byggs i Sverige idag har dimensioneringskonceptet ännu inte använts i ett riktigt projekt. Konceptet baseras också på teorier som kräver fortsatt forskning. Exempelvis har endast begränsad forskning genomförts kring erosion av sprickfyllningsmaterial. Nedbrytning av injekteringsridån, och hur detta kan påverka erosionsprocessen är också en fråga där kunskapen är begränsad. Det pågår även en diskussion kring noggrannheten i uppskattningen i bruksinträngningen över tid med de använda modellerna, där verifiering av bruksinträngningen i fält med de analytiska modellerna ännu inte genomförts. Inom samtliga av dessa områden är det önskvärt att ytterligare forskning bedrivs i framtiden.

Summary

Grout curtains under dam have previously mainly been designed based on experience and empirical techniques. In this report, the principles for a theory-based design concept for grout curtains under new dams is presented. The design concept provides an analytical solution for the design of the grout curtains and is based on existing theories on grouting in fractured rock masses. The concept enables a better control of the design and is expected to lead to an increased quality of the grout curtain and improved ability to estimate costs and time for the grouting work.

The construction of grout curtains has been an empirical technique ever since rock foundation grouting was first applied for the construction of dams in the 19th century. Grouting is usually taken as one of the ground improvement measures to reduce the rock mass hydraulic conductivity in the starting phase of the dam project. In the empirical design process the main aims have been to obtain a satisfactory sealing efficiency and proper closure of the curtain. In other words, the design of the grout curtain has not been clearly distinguished from the design of grouting work. "Rules of thumbs" and empirical design have been widely practiced in most of the aspects in grout curtain construction under dams, including depth and width of the grout curtain, spacing between grout holes, interpretations of Lugeon tests, grouting pressure and stop criteria.

Even though the empirical methods have been used with success in several projects during the years, certain disadvantages exist in the empirical design methods. In general, the quality of the grout curtain is highly dependent on the experience and knowledge of the grouting engineers. Improper application of "rules of thumbs" may lead to conservative design or other problems. For example, the widely used refusal stop criteria who determines at which flow the grouting should stop may lead to long grouting time without proportionally improved sealing effect. This could result in unnecessary high costs. High grouting pressure, which is favored by some engineers, may cause unexpected rock deformation or displacements (hydraulic jacking). In addition, the empirical methods only indirectly account for the risk of internal erosion of fracture infilling material by adjustment of acceptable Lugeon values for different conditions.

To deal with the above-mentioned problems, this report presents an analytical design concept for grout curtains under dams based on newly developed theories on rock grouting and the principles of the observational method. In this design concept, the design of the grout curtain and the design of the grouting work is distinguished. The new design concept utilizes theories on analytical interpretations of the fracture apertures, estimation of the grout spread over time and determination of the grouting pressure with respect to hydraulic jacking. It also introduces new stop criteria for grouting which are based on grouting time and injected volume of grout. The layout of the grout holes is no longer dependent on "rules of thumbs"; instead, it can be designed based on the estimated grout spread length and the desired grout curtain thickness determined with respect to the risk of erosion of fracture infilling materials and an optimized reduction of the

uplift pressure. Efforts are thus given on determining the geometry of the grout curtain, especially with respect to the thickness of the curtain which is crucial for its durability. The grouting work is then designed to achieve the desired grout curtain geometry.

It is the author's opinion that this new design concept can give more control to the engineers over the grouting design and improve the estimation of time and costs associated with the construction of grout curtains under dams. The implementation of the principles for the observational method also enables a more stringent consideration of the uncertainties involved in the analytical design. The ability to estimate the time and costs of the grouting work is also expected to increase.

As most other design concepts, the concept introduced in this report has limitations. As decreasing number of new dams are being built, especially in Sweden, this concept has not yet been implemented in a real project. The concept provides a framework of the principles for the design based on the state-of-the-art theories. However, for some of the theories further research are needed. For example, only limited research has been done on the erosion of fracture infilling materials. Degradation of the grout curtain, and how this may affect the erosion process, is also a topic where limited research has been done. There is also an ongoing discussion on the accuracy of the analytical solution for grout spread over time, where a verification of the solution in-situ has not yet been done. Further research on these topics is desired.

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1 Introduction

1.1 BACKGROUND

The grout curtain under a dam acts as an important role in the dam's functionality. The purpose of the grout curtain is to limit the leakage under the dam within an acceptable level with satisfying durability (ICOLD 1993). The grout curtain is also able to reduce the uplift pressure acting under the dam (Ruggeri 2004). A too high uplift pressure under dams could therefore threaten the stability of the dams and, as a consequence, it is important that the grout curtains are well-designed, properly constructed and well-maintained. A few of the Swedish dams have shown signs of increasing pore pressure and/or leakage, indicating that a degradation of these existing grout curtains may be ongoing or has already taken place. It could, therefore, also be necessary to repair the existing curtain by means of remedial grouting.

Grout curtains in rock foundation under dams have been designed based on experience and empirical methods, as described in several textbooks on grouting, including Houlsby (1990), Weaver (1991) and Weaver and Bruce (2007). However, in recent decades increasing theoretical knowledge about grouting has been obtained by research conducted mainly in KTH and Chalmers. This means that it is now possible to calculate the approximate spread of the grout in the fractures, to estimate which fractures that will be sealed with grout and what grouting pressure that shall be used to avoid jacking.

A numerical solution describing one- and two-dimensional penetration of grout flow was initially developed by Hässler (1991), which was then further developed by Eriksson et al. (2000) and Eriksson (2002). An analytical solution for this problem was later developed by Gustafsson et al. (2013) and its application was described by Gustafsson and Stille (2005). The theory has been applied on several grouting projects in tunnels, including the Äspö hard rock laboratory, the Northern Link project, the Bothnia Line (Stille et al. 2009) and the Cityline project in Stockholm (Tsuji et al. 2012 and Holmberg et al. 2012), in order to verify the theory's applicability. The research efforts on rock grouting have been summarized in the textbook *Rock Grouting – Theories and Applications* (Stille 2015).

The theory has also been tested on the dam projects Theun HinBoun in Laos and Gotvand Dam in Iran (Rafi et al. 2012). In these studies, the estimated grout spread in the fractures was calculated and it was also analyzed if jacking occurred by comparing curves of measured grout flow and pressure against predicted behavior. However, a complete theory-based design concept for grout curtains under dams has not yet been developed. A complete design concept for grout curtains needs to determine the spacing between the grout holes, the grouting pressure, the grout curtain thickness, the suitable grout mix and the stop criteria. The concept also needs to specify the requirements for the reduction of the rock mass hydraulic conductivity, the reduction of uplift pressure and the durability of the grout curtain. Since a complete design concept is missing, further research on this topic is needed.

1.2 AIM

The aim of this report is to present a design concept for grout curtains under new dams based on existing theories on rock grouting. This design concept will be a valuable tool for implementing the research previously performed by KTH and Chalmers into grouting practice in the Swedish hydropower industry.

1.3 DISPOSITION

The report starts in Chapter 2 with a state-of-the-art literature review, including the history of grouting under dams. The empirical practices for design of the grout curtain and the grouting work will be reviewed. Detailed information will be presented on important steps of the empirical design methods, including the design aspects concerning site investigations, geometry of the grout curtain, grout mix, grouting pressure, stop criteria, complete criteria and durability of the grout curtain.

Chapter 3 will introduce a theory-based design method for grout curtains under dams and its supporting theories. The design includes the phases of Preliminary design and Grouting execution. The introduction of the Preliminary design will be further divided into the sections: geological and hydrogeological investigations, requirements on the grout curtain, design of the grout curtain and design of the grouting work.

Chapter 4 will provide a preliminary design example using the design method introduced in Chapter 3. The fictitious project will be introduced followed by a description of the different steps that needs to be performed in the preliminary design of the grout curtain.

Chapter 5 will finish the report by discussing the developed design method compared to the previous empirical practices. Limitations and the needs for further research on specific topics related to the design method will also be discussed.

1.4 LIMITATIONS

This report introduces a design method for grout curtain under new dams. However, this theory-based analytical design method does not cover all aspects of dam grouting in a real project. The aspects not covered within this report are drilling methods and equipment, grouting facilities, contract issues and other related issues.

This design method assumes the rock mass to be described with well patterned fractures as discs. The design method shall be subject to careful evaluation or modification when facing poor rock mass qualities or other complex geological conditions.

In addition, the design concept developed in this report is mainly applicable for new grout curtains. Design of remedial grouting is not covered within this report.

2 Literature review – existing practice for design of grout curtains

2.1 THE BASICS

Dam foundation grouting is a process when fractures or other defects in the rock foundation are filled with grouts. By properly sealing the fractures, the hydraulic conductivity of the rock mass under the dam can be reduced to prevent water loss in the reservoir through the foundation, to reduce uplift pressure under concrete dams. Dam foundation grouting is usually performed with a cement based grout, where a cement suspension with water is the base of the grout mix.

Grouting under dams is usually conducted in the form of curtain grouting. A grout curtain under the dam can effectively reduce the rock mass hydraulic conductivity and interfere the natural gradient of the water pressure in rock mass. Grout holes are drilled in a single row or with multiple rows to form a curtain shape in the foundation. The geometric characteristic of the curtain, including depth, width and number of rows, are normally determined based on site conditions and the specific requirements of the project. A simple outline of a grout curtain is presented in Figure 2-1.

Grouting has long been performed as an empirical technique. This chapter will focus on introducing the history and the empirical practices that have been used by grouting practitioners.

2.2 THE HISTORY OF DAM FOUNDATION GROUTING

The grouting practice originated from Europe. According to Glossop (1960), a French engineer by the name of Charles Bérigny was the first to apply a grouting-like process to repair the foundation of a sluice at Dieppe, France in 1802. Pozzolanic mortar was used as grout and it was later proven satisfactory in terms of sealing the voids. In the following years, grouting was used in the construction or repairment of the locks of several ports in France.

Attempts to stop leakage by grouting were made in France in 1818 in Rochefort (not completed) and in 1831 on a lock on the Rhone-Rhine canal. The latter practice

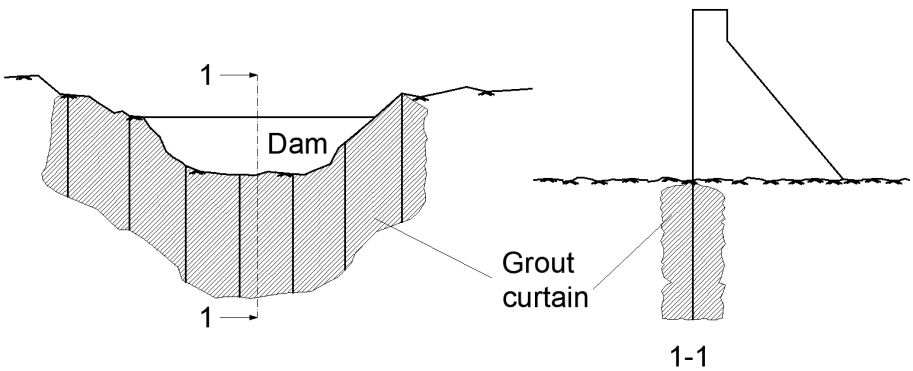


Figure 2-1: Outline of a grout curtain

was seen as a success, although small leakage remained. More grouting practices were performed in the following decades in connection with the development of pumping technology.

With years of grouting using mortar, engineers gradually turned to cement as a grouting material. Arguments remain which person that was the inventor of cement grouting. Nonetheless, in 1876, Thomas Hawksley performed the first known practice of cementitious grouting into the rock beneath an embankment dam at the Tunstall reservoir in England. Portland cement was used in this case to seal the defects and the sealing effect remained successful until 1886, when leakage appeared again. Further remedial grouting solved the problem. Grouting in rock masses was a significant step forward in the rock engineering practice, and it developed fast after it was introduced to the coalfields in northern France (Glossop 1961).

New Croton Dam (built in 1893) in the New York State in the U.S. is believed to be the first project to systematically apply cement grouting to seal the defects in its limestone foundation, although Glossop (1961) reported that reducing the uplift was the main objective instead of reducing leakage.

After several practices of grouted cutoffs on a small scale, a grout curtain was constructed for the Estacada Dam in Portland, Oregon in 1912. It was the first major application of cementitious grout curtain in the U.S. (Weaver and Bruce 2007). It was followed by a technical paper (Rands 1914) which reported the grouting details at the Estacada Dam. It is believed to be the first paper devoted to dam foundation grouting and the paper raised extensive discussion. The concept of the grouting at the Estacada Dam was a three-row grout curtain, where the third row of holes were drilled, tested and grouted for closure of the curtain. However, the effectiveness of the three-row grout curtain was not significant in terms of reduction of leakage (Weaver and Bruce 2007).

Glossop (1961) reported that the construction of the Hoover Dam (constructed between 1932 and 1935) was one of the first dams where a systematic design approach for its grout curtain was used.

In 1933, Maurice Lugeon developed and published his hydraulic conductivity criteria with an important hydraulic conductivity unit which was named after him. Lugeon's work contributed to a hydrogeological investigation method evaluating the hydraulic conductivity of the rock mass, and the testing results in Lugeon values can aid determining the grouting necessity under dams (which will be introduced in Section 2.4). His work was soon accepted in the industry and many later projects applied his method.

Statens Vattenfallsverk (1968) summarized the practices of dam foundation grouting in Sweden and generated a standard procedure for the rock foundation grouting to be performed.

Lombardi and Deere (1993) developed the grouting intensity number method (GIN-method). By taking the specific energy expended in the grout injection into account, the proper injection pressure could be determined. The GIN method has been used for dams in many countries. However, there exist discussions

questioning the applicability of the GIN method under certain conditions (Ewert 2003; Weaver and Bruce 2007; Rafi and Stille 2015a).

2.3 SITE INVESTIGATIONS

Site investigations should be performed prior to the design of the grout curtain (Statens Vattenfallsverk 1968, Houlsby 1990, Weaver and Bruce 2007, Stille 2015). The results of the investigations can be used to evaluate the necessity of grouting and to indicate a suitable grouting design. Geological and hydrogeological considerations are essential to grouting. By investigating and interpreting the conditions at site, engineers are usually able to reduce the geological and hydrogeological uncertainties related to the grouting work.

2.3.1 Geological investigations

Investigations of the geological conditions on site provides the knowledge of the fracture pattern in the rock mass to be grouted. It can help the engineers to optimize the grouting plan by designing the grout holes to properly intersect the fractures and to seal them effectively. It is neither necessary nor cost-friendly to have a detailed investigation covering every aspect of the geology only for grouting purpose. Given that the main objective of the grouting is to seal the fractures, the fracture pattern and the fracture characteristics are usually the main concern compared to other characteristic of the rock mass (although they should not be ignored either).

Geological investigation is essential for all sites and particularly for large sites, which potentially has large variety of rock formations. Thorough investigations should be performed with respect to different aspects of the grouting. At dam sites, the abutments and the storage basin should also be given attention to, aiming at finding threats on their seepage capacity. The exploration methods (including surface inspection, core drilling etc.) have been described in detail in various literatures (see for example Houlsby 1990 and Palmström and Stille 2014).

Houlsby (1990) concluded that the geologic features that could influence the grouting program are:

- Spacing of fractures. The denser the fracture pattern, the more difficult it is to perform a successful grouting.
- Fracture aperture and continuity. Larger opening of the fractures allows the grout to spread easier but will also imply that the rock mass may be more difficult to grout.
- Fracture orientation. Vertical grout holes may be optimum for fractures whose dip varies between 0° and about 60°, otherwise the intersection should be achieved using inclined holes. Common practice regarding characterization of fracture pattern is mapping of surface outcrops and drill cores together with BIPS-logging, which is able to film the boreholes and the fractures in order to analyze the fracture orientations and apertures.
- Uniformity of the site. Regular layout of grout holes may be sufficient for uniform fracture patterns. Varying inclination and irregular layout may be

required in the case of more complex fracture patterns or when the site is divided into different geological domains.

- Rock mass quality. Poor rock mass quality around the grout holes may cause difficulties in grouting, and packers cannot be used in such case.
- Rock mass strength. Grouting is usually easier in strong, massive and well-anchored rock.
- Stress conditions in the rock mass. Special treatments may be needed if high tectonic stresses exist in the rock mass.
- Erosion. After grouting, the residual seepage may erode the weak fracture walls or some infilling materials in the grouted zone. To avoid erosion, high standard of grouting is required.
- Chemical attacks. Higher standard of grouting may be needed if the rock (or the grouting material) is prone to chemical attacks.
- Karst and other voids. Large voids require special methods for them to be grouted.

In addition to the above recommendations from Houlsby (1990), the identification of the infilling materials is also an important task for the geological investigations (Palmström and Stille 2014). It may be conducted with the above mentioned BIPS-logging method, but is should be observed that infilling materials close to drill hole most of the time is flushed away while drilling. This can make it difficult to observe any infilling materials from the BIPS-logging or while mapping drill cores.

2.3.2 Hydrogeological investigation

Weaver and Bruce (2007) suggested that "*geohydrologic (hydrogeological) studies should be conducted where the results of the preliminary geologic studies indicate … highly permeable conditions*". Hydraulic conductivity testing is essential before grouting since the ultimate purpose of grouting is to reduce seepage. Lugeon test is the most commonly used type of testing. This test is performed by injecting water into the rock mass and measuring the water loss within a section of a borehole under stationary water injection pressure. The Lugeon tests are usually performed during exploratory testing or for grout hole testing.

Exploratory testing

Exploratory testing is the testing carried out before the design of the grout curtain is performed in order to get an overview of the rock mass hydraulic conductivity at the dam site. It is usually more thoroughly performed than grout hole testing.

Exploratory testing can be performed downwards by sections together with the drilling of the holes or it can be performed by sections in existing exploratory holes drilled for geological investigations. Houlsby (1990), stated that the stage length is adjustable and depends on the specific required precision. Around 5 m to 6 m may be sufficient, but sometimes even less. In a protocol for the Lugeon test presented in Statens Vattenfallsverk (1968), 3 m was used as the section length.

In a Lugeon test, packer or packers (usually a single packer) are placed in position to isolate the tested section from the rest of the hole. For each section in the borehole, a round of Lugeon test is conducted, and the test result is normally presented with Lugeon units (1 Lugeon unit = 1 liter/m/minute·1 (MPa)/actual

pressure (in MPa)). The Lugeon value solely should not be determined without knowledge on the fracture patterns, given the fact that the same Lugeon value can be reached by one large fracture or by many fine ones. Therefore, geologic investigations are desired as an aid to interpret the results from the Lugeon tests.

In each section, the Lugeon tests are often performed under staged pressures. Five pressures are usually applied, with 10 minutes holding time for each pressure. The five pressures follow the sequence: LOW-MODERATE-PEAK-MODERATE-LOW. The Lugeon values are to be calculated under each pressure. Six scenarios of results were described by Stille et al. (2012) as typical indications for different flow conditions that may be found from the tests, see Figure 2-2.

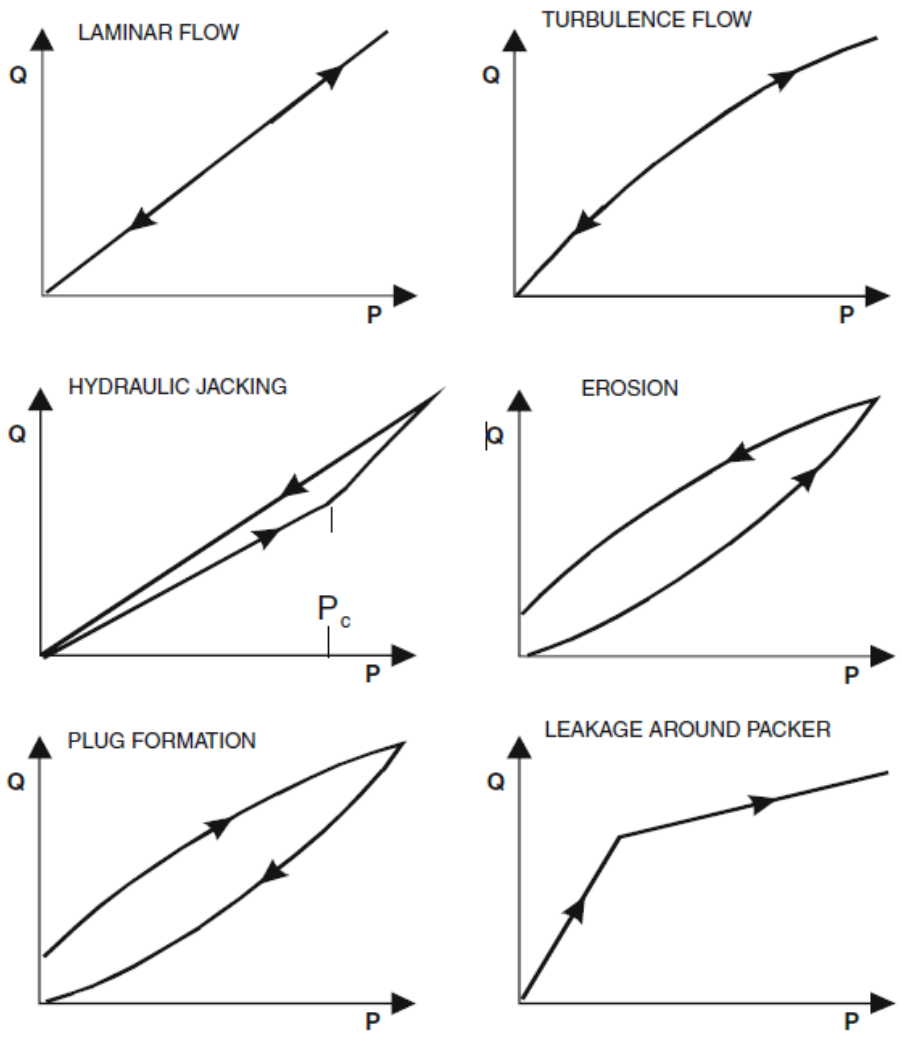


Figure 2-2: Interpretations on different Lugeon test results (from Stille et al. 2012)

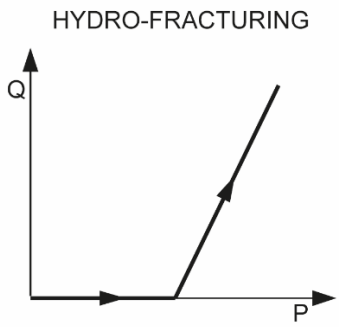


Figure 2-3: Interpretation of Lugeon tests indicating hydro-fracturing

Given that high pressure may create hydro-fracturing, another condition in addition to the previous five conditions was mentioned in Weaver and Bruce (2007). The behavior is presented in Figure 2-3.

Weaver and Bruce (2007) suggested that the determination of the highest testing pressure should be based on the strength of the rock to avoid hydro-fracture or jacking of the fractures. According to Littlejohn (1992), appropriate pressures for weak or soft rock masses may be 0.2, 0.4, 0.6, 0.4, 0.2 MPa and 1, 2, 4, 2, 1 MPa for strong rock masses. Houlsby (1990) pointed out that testing pressure is also related to the depth of the testing stage (see section 2.6.2).

Grout hole testing

Grout hole testing can be seen as "*a simpler version of the exploratory test*" (Houlsby 1990). This testing is performed in the grout holes before grouting. Compared to exploratory testing, the purposes of grout hole testing are to:

- Indicate the hydraulic conductivity of a certain staged section.
- Show the potential problems that could affect the grouting process.
- Work as a rehearsal of the grouting.

The techniques of grout hole testing are similar to exploratory testing. A recommended water injecting pressure of 0.1 MPa (constant) was given by Houlsby (1990). Therefore, there is no additional stages with different pressures for grout hole testing. But the flow should still be recorded within every period of time (5 min with three records in a total time of 15 min is sufficient). The average Lugeon value can be calculated from the recorded water flow if the three values are similar. If the water flow decreases with time, a longer time and more records might be needed until the values are stable. This undesirable situation occurs when the water losses increase with time, indicating erosion of infilling materials in the fractures. In this case, the pressure for the subsequent grout hole testing should be altered.

2.4 REQUIREMENTS ON THE GROUT CURTAINS

Grouting could be omitted at some dam sites with ideal geological conditions even though most of the sites cannot. The guideline on "whether grouting is necessary"

from Houlsby (1990) is widely accepted (see Figure 2-4). The criteria are given in terms of Lugeon value. According to the result of the Lugeon tests, any foundation part that has a hydraulic conductivity that is poorer than the criteria in the guideline warrants grouting. The criteria can also be taken as the requirements on the grout curtain which is to be constructed. The Swedish guidelines of dam safety RIDAS (Swedenergy 2011) also provides the requirements on the standard of grouting with respect to the residual Lugeon values after grouting. The requirements are also presented in Figure 2-4.

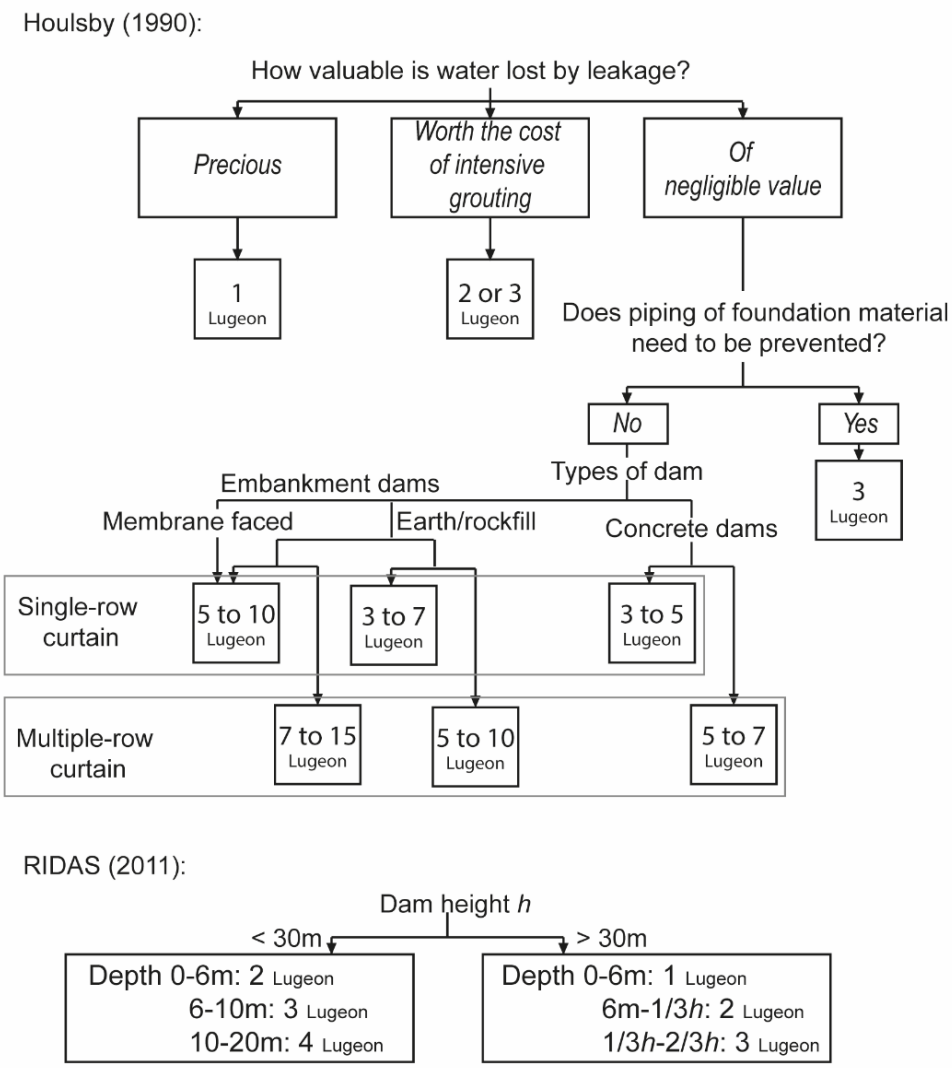


Figure 2-4: Lugeon value requirements on the grout curtains in the dam foundation (after Houlsby 1990 and Swedenergy 2011)

2.5 DESIGN OF THE GROUT CURTAIN

2.5.1 Inclined holes or vertical holes

The principle of drilling a grout hole is to intersect as many fractures as possible. Therefore, the inclination of grout holes is usually determined according to the fracture patterns in the rock mass that is to be grouted. Vertical holes are most suitable for the sets of fractures inclined in a regular and inclined symmetric manner (see Figure 2-5, left), or for fractures that are spaced closely where the grout may penetrate easily through the fractures. Otherwise the holes need to be inclined (see Figure 2-5, right).

The angle of inclination, usually measured from the vertical, does not need to be more precise than 5° or 10° in dam foundation grouting. Having a horizontal surface, the maximum allowed angle is about 45° and for steep surfaces, the drill angle can be larger. Houlsby (1990) introduced a way to determine the inclination of grout holes. In this method, hemispherical equal-area projection is used. The concept of this method is to determine the optimum inclination of grout holes directly on the equal-area projection, to mark the grout hole on the projection and to trade-off among various sets of fractures.

2.5.2 Depth and length of the grout curtain

Empirically, the depth of the grout curtain is mainly determined according to the geological conditions or the waterhead in the reservoir. Houlsby (1990) reported that a low hydraulic conductivity of the rock foundation shown from the investigations generally indicates that a shallow curtain is needed, whereas a highly permeable foundation requires a curtain with sufficient depth. Under such circumstances with high hydraulic conductivity, the curtain may be constructed to a depth equal to the head of the water head. Ewert (2003) suggested that the curtain depth being equal to the height of the dam.

In the Swedish guidelines of dam safety RIDAS (Swedenergy 2011), the limits of the grout curtain depth were recommended as: if the dam height is less than 30 m, the grout curtain may not be deeper than 20 m; if the dam height is more than 30 m, the grout curtain may not be deeper than 2/3 of the dam height.

To avoid water leakage or seepage around the ends of the grout curtain, the grout curtain should be extended into the abutments for a proper distance. This determines the total length of grout curtain.

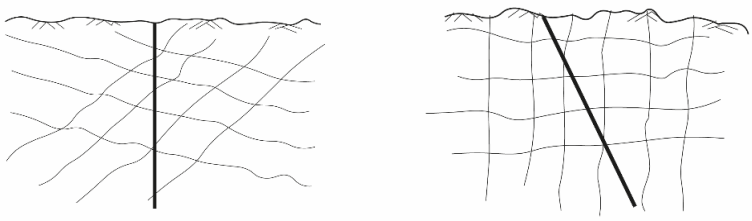


Figure 2-5: Vertical and inclined grout holes

2.5.3 Sequenced grout holes

Normally the grouting is performed in a grout hole in sections of depth, similar with the concepts of the Lugeon test. The grout curtain is suggested to be constructed in a sequenced manner. Houlsby (1990) advocated an arrangement for sequenced grout holes, as shown in Figure 2-7. In such a layout, efficient work and cost control can be performed by grouting different order of holes. Higher order of holes can be drilled with shallower depth than the previous order of holes for checking the quality of the grouting in previous order of holes. This check can provide indications on if higher order of holes themselves should be grouted or not. This criterion will be further described in Section 2.6.2. This sequenced grouting also allows additional specific grouting work for weak areas during the grouting progress.

Primary holes (*P*) are drilled and grouted to the bottom of the curtain. Secondary holes (*S*) are drilled and grouted to the second bottom stage and are extended to the same stage as the primary holes if primary holes show high water or cement take. Similar method applies for Tertiary holes (*T*) and even further higher order of grout holes.

Spacing for Primary holes was suggested by Houlsby (1990) to be 12 m for most sites and 6 m for sites where the hydraulic conductivity of the rock mass is so low that the connection between the holes is limited.

2.5.4 Single or multiple rows of the grout curtain

Different opinions exist on whether a single-row grout curtain is sufficient for its functionality or if a multiple-row curtain should be constructed. If the quality of the grouting work can be guaranteed, i.e. no unwanted openings exists between the grout holes, a single-row curtain can be accepted. However, in most of the cases when the quality of the grouting could not be fully guaranteed and a single-row curtain may not work as expected, multiple-row curtain is recommended by various practitioners.

Another reason for multiple-row curtains to be used is that their larger thickness can lower the hydraulic gradient. But no clear criteria on the thickness nor the hydraulic gradient was given in the classic textbooks (see for example Houlsby 1990 and Weaver and Bruce 2007).

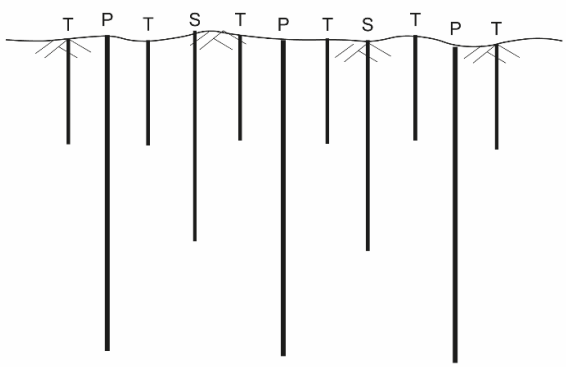


Figure 2-6: An example of sequenced grout holes

A multiple-row grout curtain usually consists of two outer rows and one center row. When constructing the multiple-row curtain, outer rows are usually first grouted at lower standard in terms of the reduction on water flow, not necessarily grouted to closure. This is followed by the grouting of the center row. This center row is performed with a higher standard so that this row will be grouted to closure (Weaver and Bruce 2007). In order to optimize the effectiveness of the grout curtain, spacing between rows should be chosen as less than twice the spread of the grouts so that the grouts can meet and closure can be achieved although this spread is difficult to evaluate in an empirical design.

A new formation of multiple-row grout curtain was introduced by Weaver and Bruce (2007). The two-row curtain with close rows and grout holes at opposing angles for two rows (see Figure 2-7). This arrangement of holes may be able to reduce the likelihood for large ungrouted openings and it has been adopted at several sites in the U.S. Weaver and Bruce (2007) commented that this design approach "*deserves more universal adoption as a basic design standard*".

2.5.5 Location of the grout curtain

Under concrete dams, grout curtains are usually constructed near the heel of the dam, either inclined or vertical (see Figure 2-8, for an example of a concrete buttress dam). Under embankment dams on the other hand, as suggested by e.g. Houlsby (1990), the grout curtain should be located under the core but not more downstream than the centerline of the core.

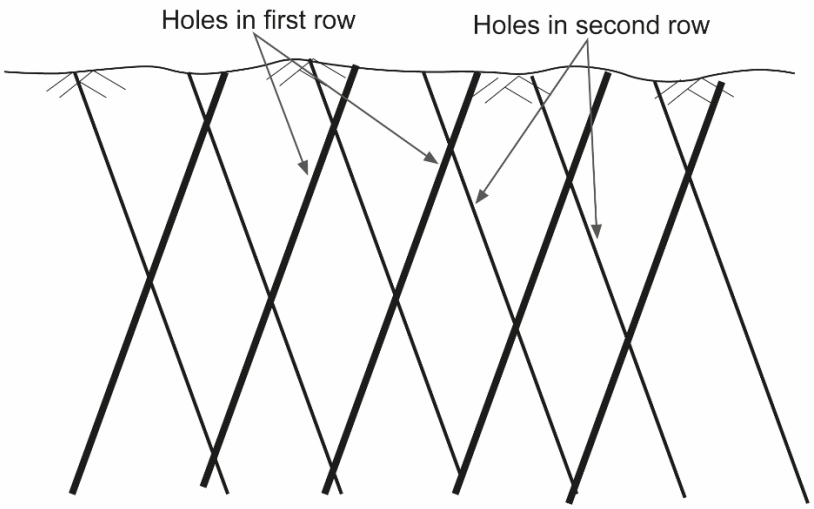


Figure 2-7: Two-row curtain with grout holes at opposing angles (after Weaver and Bruce 2007)

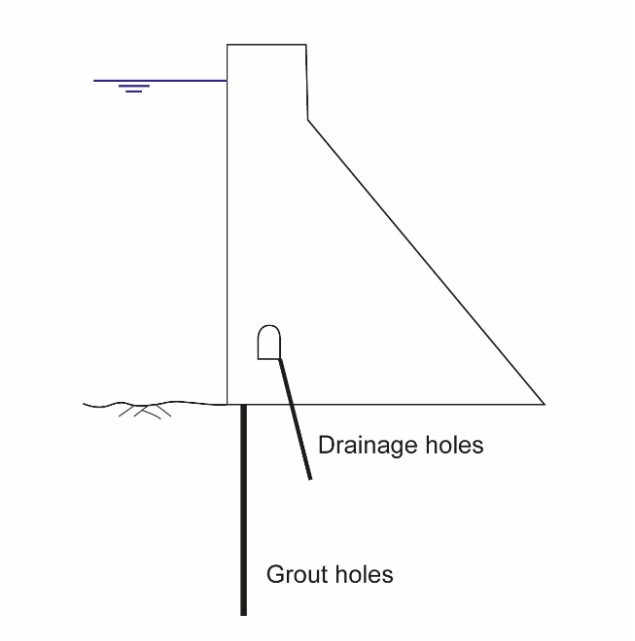


Figure 2-8: Position of grout curtain under a concrete dam

2.6 DESIGN OF THE GROUTING WORK

2.6.1 Grout mix

The main consideration of grout mix design is to guarantee favorable penetration and at the same time obtain sufficient strength and durability. The water- cement ratio (w:c ratio) is most-commonly used to specify the amount of cement in the grout mix or the "thickness" of the grout mix. The w:c ratio of the grout mix should not be too low, otherwise the flowability will be poor and the grout is unable to spread the desired distance. A higher w:c ratio indicates thinner mix, which is expected to have better flowability than denser grout mix with lower w:c ratio. However, too high w:c ratio could be undesirable in terms of bleeding and setting time. It affects the quality and durability of the grout curtain. In tunnel grouting in Sweden, the w:c ratio of 0.8 is usually used. Many other types of additives are usually added into the grout mix in order to get the required properties, such as clay, bentonite, or fillers (most commonly sand) and pozzolans (Houlsby 1990, Weaver and Bruce 2007).

2.6.2 Grouting pressure

"Rules of thumb" have long been accepted for choosing the grouting pressure: the European rule of thumb (1 kg/cm^2 per meter of depth) and the U.S. rule of thumb (1 lb/in^2 per foot of depth) as well as the normal Swedish practice are shown in Figure 2-9. However, the conditions in the foundation varies among sites and the rules of thumbs should not be seen as inflexible standards. The principle of determining the maximum allowable grout pressure is to find a pressure that could achieve maximum grout penetration without triggering unexpected deformation in the foundation or damaging the foundation in any form (hydro-

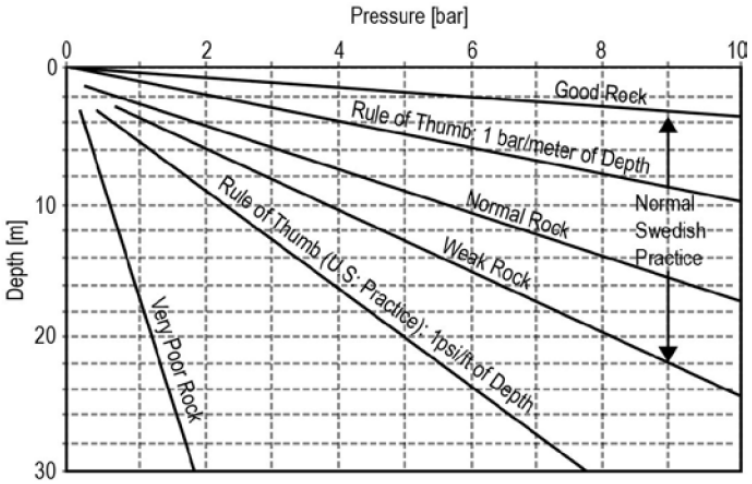


Figure 2-9: Grouting pressure in relation to the depth and rock quality (from Weaver 1991)

fracturing and jacking are two of the main issues that need to be considered). In Figure 2-9, the acceptable grouting pressure for different rock mass qualities with respect to the grouting depth can be observed.

Different philosophies regarding grouting pressure exist for the grouting work, namely penetration grouting and displacement grouting (Houlsby 1990). Penetration grouting represents the grouting work which is performed with relatively low injection pressure. Displacement grouting utilize high injection pressure so that small fractures in the rock mass open up and the grout could penetrate further. Additionally, when injection is finished and pressure is released, the fractures will tend to close and form a rather tight bond (Cambefort 1977). It is, however, important to know that high pressure may not be applicable on all sites (Ewert 1992). As mentioned previously, the choice of grouting pressure should not damage the foundation. Therefore, for soft rock which is relatively easy to be damaged by a high pressure, displacement grouting may be inappropriate.

The grout intensity number method (GIN method) developed by Lombardi and Deere (1993) is a commonly used method internationally and has been applied to many grouting projects. The GIN method takes into account the energy spent in the grouting. The grout intensity number is the product of the final pressure and the volume of the injected grout. They suggested that the GIN value should be maintained constant for different grout intervals in order to keep a constant grout penetration. As the grouting pressure builds up, the injected volume increases correspondingly, but the Pressure-Volume path should be limited by the hyperbolic curve which indicating a constant GIN value. Different grouting intervals should be limited by the same hyperbolic envelope, with finer fractures using higher pressure and larger fractures using lower pressure (as seen in Figure 2-10). The recommended pressure limit for most of the conditions is 3 MPa (30 bars) with a limiting volume of 200 L/m (Weaver and Bruce 2007). The GIN method has been proven successful for several projects (Lombardi and Deere 1993), although arguments arose after some failure of using this method. Rafi (2014) and Rafi and Stille (2015a) suggested that the GIN method should be used with caution when facing

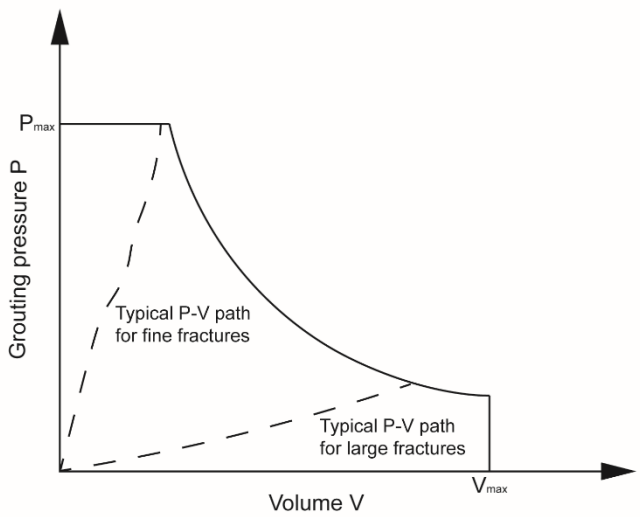


Figure 2-10: The limiting envelope of the grouting intensity number (GIN) and the typical paths for different grouting intervals (after Lombardi and Deere 1993)

complexities such as shallow fractures or fractures which require a higher degree of sealing. It may result in unfavorable jacking in the foundation with large grout intake. Adequate understanding should be put on those situations in order to succeed in conducting the grouting program.

2.6.3 Stop criteria and complete criteria

Closure is vital in curtain grouting. Only when the curtain is closed, the water cannot penetrate, as seen in Figure 2-11. Once the curtain has been closed with an acceptable overlap, the grouting process could stop.

Refusal is sometimes the criterion for stopping the grouting of each section in a sectional grouting process. Weaver and Bruce (2007) suggested that a period of at least 10 minutes is required for measuring the grout take which has already shown the sign of refusal. This holding time is to exclude other possible reasons that can cause a low grout take rate. However, this acceptance criterion could only indicate that fractures are likely to be sealed, but not able to ensure that closure in the grout curtain has been achieved. The same argument was made about the reduction in grout takes- it can only indicate the reduction of hydraulic conductivity qualitatively, not quantitatively. In addition, holding the grouting for 10 minutes for each single section is not time-efficient and may results in higher cost, which, however, may not lead to an improved quality.

Houlsby (1990) further argued that the correlation between the grout take and the hydraulic conductivity was rather weak. A criterion to externally assess the closure was thus suggested. This method was focusing on the effectiveness (hydraulic conductivity reduction). The split-spacing Lugeon tests in holes along with the grouting process were suggested by Houlsby, one presentation is shown in

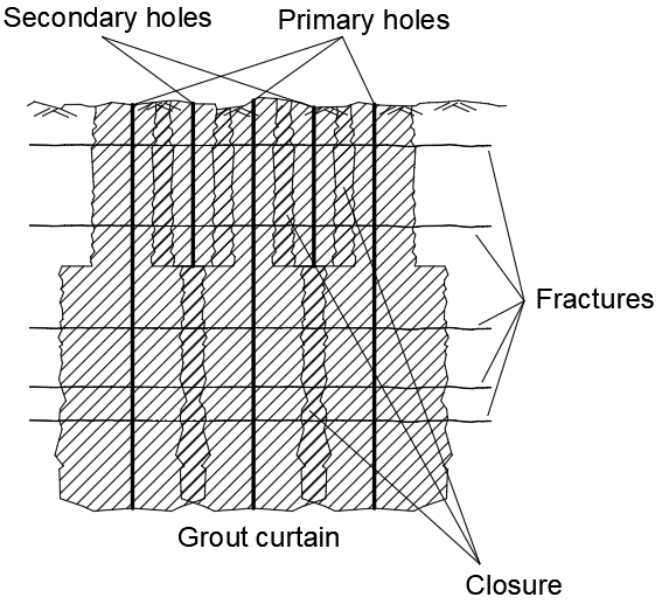


Figure 2-11: Concept of closure in a grout curtain

Figure 2-12. A Lugeon test should be performed in the primary holes before the grouting of each primary hole. After the grouting is finished, the secondary holes are drilled following a split-spacing principle. Then water test should be performed in the secondary holes before they are grouted. These Lugeon test results reflect the effectiveness of the earlier grouting work in primary holes. Thus, they should be assessed and compared with the requirements mentioned previously in Figure 2-4. The grouting should proceed and the Lugeon test should be repeated in the secondary holes (and higher orders, if necessary), until the results indicate a hydraulic conductivity that satisfies the requirements. Closure is believed to have been achieved at this moment and the grout curtain is finished. This criterion can be named the complete criteria.

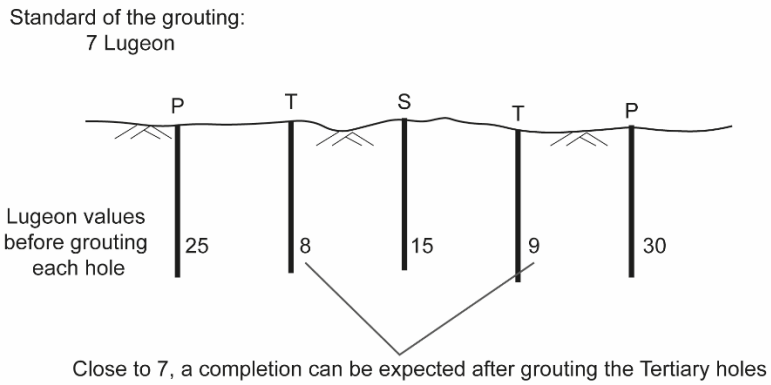


Figure 2-12: Complete criteria regarding hydraulic conductivity (after Houlsby 1990)

2.7 DURABILITY OF THE GROUT CURTAIN

Grout curtains could hardly avoid being degraded. One major factor affecting the durability of the grout curtain is the quality of the grouting execution. Apart from the execution, the durability of the grout curtain should also be taken into consideration in the planning phase.

Erosion is one of the main issues that threats the durability of the grout curtain. It could be caused by various factors, including unfavorable geochemical environment, chemical aggressive water, inappropriate grout mix, high hydraulic gradient and erodible rock mass or fracture infillings.

Deleterious minerals as well as aggressive acidic groundwater are able to initiate the deterioration of the grout curtain made of traditional Portland cement. Other grout materials can be used when the mentioned geochemical environment is detected, i.e. chemical grout.

Bleeding of the grout may occur if the water-cement ratio is too high, leading to an unsealed path between the non-hardened grout and the fracture boundary. This will degrade the sealing effect of the grout curtain. The effect caused by bleeding is illustrated in the right case in Figure 2-13. In addition, if the grout mix is too thin, water flow could cause degradation of the grout curtain during the grouting by means of dissolution or extrusion (see the left case of Figure 2-13).

A high hydraulic gradient is capable of making the water more threatening to the hardened grout. It is also very problematic when facing an erodible rock mass or infilling materials in the fractures. In such cases, especially under high hydraulic gradients, the infilling materials or the rock itself could be washed out, as seen in Figure 2-14, where the erosion of infilling materials creates new paths for water to flow in between the hardened grout and the fracture boundary. The hydraulic conductivity of a grouted rock mass could face an increase if serious erosion of infilling materials occurs. A possible solution on this is to construct the grout curtain with a larger thickness, e.g. a multiple-row curtain, to lengthen the path of water pressure reduction which thereby decrease the gradient.

Besides the internal erosion, the dehydration shrinkage of the hardened grout may also create new paths for the flow. This risk for dense grout mixes is not significant due to the presence of underground water, which prevents the dehydration. For the use of a thinner grout mix, however, the internal dehydration may occur due to the chemical bonded water regardless of the presence of underground water. (Rosenqvist 2020)

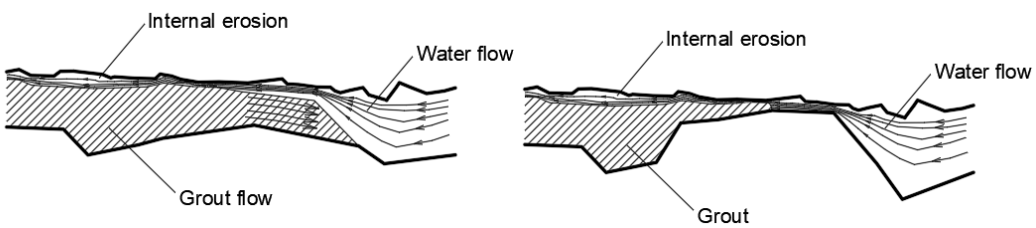


Figure 2-13: Internal erosion of grout curtain during (on the left) and after (the flow path between the non-hardened grout and the fracture boundary caused by bleeding, on the right) the grouting work

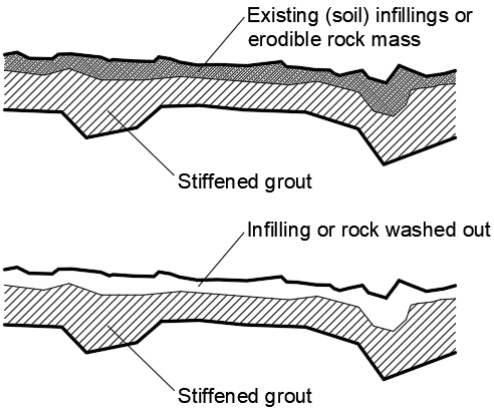


Figure 2-14: Negative effects from erodible rock mass or infilling materials

Although it might be possible to solve the durability problems by remedial grouting, the adverse effects for the new grout curtain should be considered carefully in the planning phase of the grout curtain design.

2.8 CONCLUDING REMARKS

As described in previous sections, grouting has long been based on empirical techniques which rely more on experience rather than theory. The verification of the grouting work is normally based on prescriptive methods. Although the grouting work can be performed with a good quality using empirical design criteria, a design based on a more theoretical background is able to give less variation on the grouting quality, resulting in a more effective and efficient grouting work.

With the extensive research conducted in recent years, mainly performed in Chalmers University of Technology and KTH in Sweden, a better understanding of the mechanisms influencing the grouting design have been provided, especially in the fields of grout spread in fractures, hydraulic jacking, cement-based grout materials and the geometry and hydraulic conductivity of the rock mass. In addition, the observational method has developed throughout the years and is proved applicable in grouting practices. These progresses have given strong theoretical foundation for grouting design based on theories. The related theories and design concepts for a theory-based design approach are introduced in the following chapters.

3 Design concept for grout curtains

Growing theoretical knowledge has facilitated the development of an analytical design method for grout curtain under dams. Although the analytical design of grout curtains can have certain limitations, the uncertainties involved in the grouting work can be reduced by using a method based on a more theoretical background compared to the previous empirical techniques described in chapter 2.

The general concept for the design of grout curtains developed and presented in this chapter is based on existing theory and the principles of the observational method. This method has been defined in Eurocode 7: Geotechnical Design, as generalized in Figure 3-1. The method is suitable for projects involving large epistemic uncertainties, where the final conditions are not known until the execution of the project. Given the large epistemic uncertainties involved in the grouting work, the observational method is therefore suitable to be applied. The observational method has e.g. been found applicable in dam foundation remedial grouting projects (Spross et al. 2016).

Following the principles of the observational method, a curtain grouting project under a new dam can be divided into two stages, preliminary design and grouting execution, as suggested in the flow chart in Figure 3-2.

The elements shown in the flow chart will be described in detail in this chapter, where the order of the sections follows the flow chart.

It is worth mention that due to the large complexity of the rock mass, the whole grouting design process could never proceed without involving engineering judgements based on empirical knowledge. In other words, some design aspects from the previous chapter are still valid and necessary when performing the design of a grout curtain. As in many other engineering fields, if not all, analytical theories and empirical knowledge are inseparable, and they support each other.

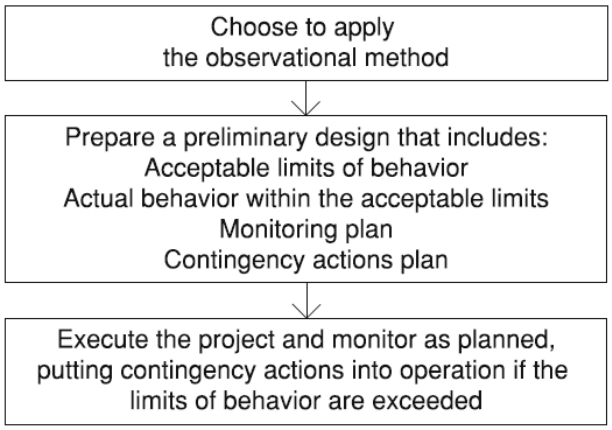


Figure 3-1: An outline of the principles of the observational method (from Spross et al. 2016)

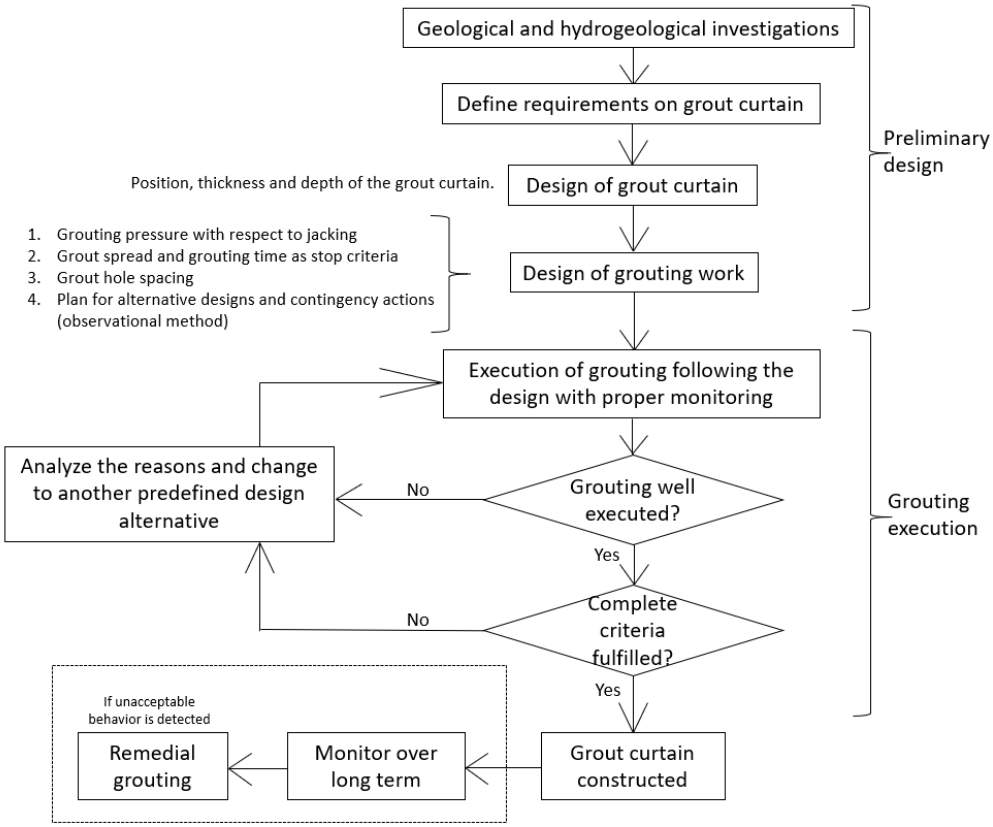


Figure 3-2: Flow chart for grout curtain design

3.1 PRELIMINARY DESIGN

The preliminary design of grout curtains provides a guideline for the grout curtain geometry. As shown in Figure 3-2, the preliminary design can be divided into four steps: (i) Geological and hydrogeological investigations, (ii) Definition of the requirements on the grout curtain, (iii) Design of the grout curtain and (iv) Design of the grouting work.

The first step, *Geological and hydrogeological investigations*, provides the engineers with indications on rock mass quality, hydrogeological conditions, fracture patterns and fracture properties in the rock foundation under the dam, thus giving reference for the following design steps.

The second step, *Definition of requirements on the grout curtain*, is preferably done together with the design of the dam. Based on the results of site investigations performed in the previous step, the requirements on the grout curtain should be determined with respect to the required hydraulic conductivity reduction of the rock mass, the uplift pressure on the dam and the long-term durability of the curtain, i.e. with respect to internal erosion of fracture infilling materials or degradation due to leaching and other chemical processes.

The third step is the *Design of the grout curtain*. In this step, the geometry including depth and thickness of the grout curtain and the position of the grout curtain are to be determined given the results from the previous steps.

The fourth and last step in the preliminary design is the *Design of the grouting work*. This step must be distinguished from the third step. Design of the grouting work is different from design of the grout curtain. Design of the grouting work includes to plan the execution of the grouting process in order to achieve the design of the grout curtain from the third step. In this fourth step, a grouting plan including: grout mix, grout hole spacing, grouting pressure and stop criteria are to be prepared. In terms of the following grouting execution, the alternative design options should be prepared in this step as well as the contingency plans for unfavorable behaviors and the acceptance criteria to activate the contingency plans in accordance with the principles of the observational method. Furthermore, to indicate the completion of the grouting work, complete criteria should also be provided. Note that the equipment, grouting technique and other planning aspects will not be discussed further in this report, even though they are also essential factors to consider in the preparation of the grouting work.

All steps above will be further described in detail in the following sections.

3.1.1 Geological and hydrogeological investigations

One of the main purposes for grouting the rock mass is to reduce the hydraulic conductivity of the rock mass. Investigations on geological and hydrogeological conditions give references to evaluations for the necessity of grouting during the pre-design phase of the dam project. The rock mass quality, fracture patterns and fracture properties in the rock mass can be indicated by such investigations.

Geological investigations

The geological investigations for grouting do not require more than the normal pre-investigations for a rock mass. Investigations includes drilling exploration holes as well as mapping of surface outcrops. The drill cores from the exploration drilling should be mapped in order to evaluate rock mass quality including identification of fracture orientation and their characteristics. It is important to have knowledge on the type of infilling material. However, infilling material could be flushed away during drilling and sometimes triple-tube core drilling could be required in order to obtain undisturbed samples. Standard procedures for this type of investigations can be found in many rock engineering textbooks, see e.g. *Rock Engineering* by Palmström and Stille (2014).

Hydrogeological investigations

The types and methods of hydrogeological investigations have been introduced previously in Section 2.3.2. Among the testing methods, sectional Lugeon tests in grout holes using packers (section transmissivity) is the most common investigation. Compared with the contribution from independent fractures, which is believed to be randomly distributed, the use of sectional tests is more practical since it can be measured directly (Stille 2015). Detailed instructions on the

procedure for water pressure tests could be found in the standard ISO 22282-3 (2012).

Lugeon tests provide results of the water loss within the testing section in Lugeon values. The Lugeon values can be used to calculate the hydraulic conductivity with the aid of Moye's formula, as

$$K = \frac{Q}{L \cdot \Delta H} \cdot \left[\frac{1 + \ln\left(\frac{L}{D}\right)}{2\pi}\right] \approx \frac{Q}{L \cdot \Delta H} \tag{3.1}$$

where Q is the flow of water in m³/s, L is the test length in meters and ΔH is the excess testing pressure in meters of water pillar. Although the Lugeon value (Lu) and the hydraulic conductivity (K) essentially share the same concept, a conversion of units between them is needed as indicated in

$$K \approx Lu \cdot 1.3 \cdot 10^{-7} \quad [\text{m/s}] \tag{3.2}$$

From the hydraulic conductivity (K) one can calculate the section transmissivity (T_s) as

$$T_s = K \cdot L \tag{3.3}$$

Calculation of fracture aperture from measured section transmissivity

Gustafson (2012) suggested that the fracture transmissivity in an ungrouted rock mass can be expressed by a power law distribution (Pareto distribution) and the total section transmissivity is described as the sum of all fractures according to

$$T_s = \sum_{i=1}^{N} T_{f,i} = T_{max}\left[\frac{1}{1^{\frac{1}{k}}} + \frac{1}{2^{\frac{1}{k}}} + \cdots + \frac{1}{N^{\frac{1}{k}}}\right] = T_{max} \cdot S(k, N) \tag{3.4}$$

where N is the number of independent fractures, $T_{f,i}$ is the fracture transmissivity for the i_{th} fracture, T_{max} is the maximum single fracture transmissivity, k is the distribution parameter, which is 0.4-0.5 in normally fractured rock masses. $S(k,N)$ has been calculated with different k values and number of fractures by Gustafson (2012). Given that k is around 0.4-0.5, the maximum fracture's transmissivity could take up to more than 75% of the total transmissivity of tens of fractures. When it comes to 100 or 1000 fractures, the maximum fracture transmissivity could still take up to more than 50% of the total section transmissivity. Hernqvist et al. (2014) also found that the flow in a section is dominated by the maximum fracture, its contribution could vary between 30% and 100%, see Figure 3-3. However, attention should be given to the fact that this result may not be generalized to all rock masses. Equation (3.4) assumes that all fractures are water-bearing and that the flows in different fractures are independent, whereas in reality the fracture are often inter-connected and some fine fractures are not water-bearing. As a result of this assumption, the maximum fracture transmissivity in the section may get overestimated. For this reason, equation (3.4) is not directly used in the design

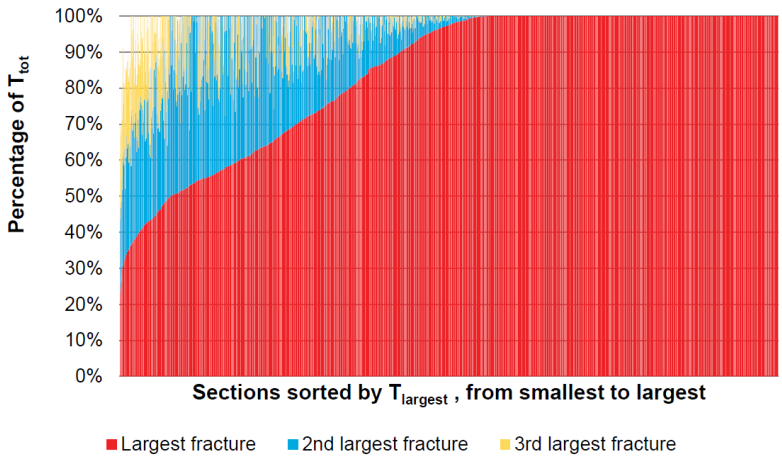


Figure 3-3: Transmissivity distribution from large fractures in 5m sections of vertical boreholes (from Hernqvist et al 2014)

process, the contribution of transmissivity from the largest fracture in a section should be carefully assumed based on the fracture patterns and properties.

Given that most of the transmissivity within a section is contributed from the maximum fracture, calculation of the aperture of the maximum fracture within each testing section is possible using the total section transmissivity. However, "most of the transmissivity" does not indicate a fixed proportion. Therefore, assumptions need to be made with the aid of the fracture patterns observed from geological investigations. The coefficient k_2 is introduced as the ratio between section transmissivity (T_s) and maximum fracture transmissivity (T_{fm}), which normally varies between 1.1 and 2.0. The maximum hydraulic aperture (b_{mh}) can thus be calculated according to the cubic law in Eq.(3.5) to (3.7), modified from Stille (2015) as

$$T_{fm} = \frac{\rho_w g}{12\mu} b_{mh}^3 \tag{3.5}$$

$$T_{fm} = \frac{T_s}{k_2} \tag{3.6}$$

$$b_{mh} = \sqrt[3]{\frac{T_s \cdot 12\mu}{k_2 \rho_w g}} \tag{3.7}$$

where μ is the dynamic viscosity of water ($1.3 \cdot 10^{-3}$ Pa·s at 10°C), ρ_w is the density of water (1,000 kg/m^3).

The hydraulic aperture can be calculated from the section transmissivity. However, it is the physical aperture that determines the grout take. To relate the results of the hydrogeological investigations to the real conditions in the rock mass, many have studied the relation between physical aperture and hydraulic aperture. Zimmerman and Bodvarsson (1996) found an approximated relation between hydraulic aperture (b_h) and the normal distribution parameters of the physical aperture (median μ_b and standard deviation σ_b) as

$$b_h^3 = \mu_b^3 \left(1 - \frac{1.5\sigma_b^2}{\mu_b^2}\right)(1 - 2c) \tag{3.8}$$

where c is the relative contact area between the fracture boundaries.

The results from Zimmerman and Bodvarsson (1996) indicate that the physical aperture is roughly 1.5-2 times of the hydraulic aperture, with a ratio σ_b/μ_b between 0.65-0.8.

3.1.2 Requirements on the grout curtain

A successfully constructed grout curtain under a dam is expected to: (i) lower the permeabitily of the rock mass, (ii) reduce the uplift pressure acting on the base of the dam, and (iii) prevent internal erosion within the grout curtain, particularly with respect to the erosion of any infilling material in the fractures. The requirements on the grout curtain with respect to these three points are presented below.

Hydraulic conductivity of the rock mass

The requirements with respect to the lowering of the hydraulic conductivity of the rock mass should be fulfilled in the preliminary design. The expected residual hydraulic conductivity (K_{exp}) with respect to water can be chosen according to the class of grouting as shown previously in Figure 2-4 after a unit conversion from Lugeon to hydraulic conductivity, or according to the specific requirements of the projects. The requirement is presented in Eq.(3.9). The expected residual hydraulic conductivity (K_g) should be no higher than the required residual hydraulic conductivity (K_{exp})

$$K_g \leq K_{exp} \tag{3.9}$$

Uplift pressure

The grout curtain potentially reduces the uplift pore water pressure acting on the dam body. It is especially beneficial for concrete dams. This reduction effect could help to reduce the use of materials on the dam body and thus lower the cost. However, in many guidelines and recommendations for dam safety, the reduction of uplift pressure introduced by the grout curtain is not recommended to be accounted for. One such example is the Swedish guidelines on dam safety, RIDAS (2017). Despite this, with respect to the stability of the dam, it is still beneficial to achieve as low uplift pressure as possible (H_{min}) brought by the grout curtain without sacrificing its effectiveness of reducing hydraulic conductivity and its durability over long term.

Internal erosion of fracture infilling materials

The internal erosion of the infilling material in the fractures should be prevented. High flow velocity, as a result of steep hydraulic gradient, may erode the infilling materials in the grouted zone and may create new flow paths during the dam operation. This phenomenon threatens the durability of the grout curtain and jeopardizes the stability of the dam. Depending on the type of the infilling materials, this requirement can be expressed as a limitation on the hydraulic gradient (i_g) within the grout curtain

$$i_g \leq i_{crit} \tag{3.10}$$

where i_{crit} is the critical hydraulic gradient to prevent the infilling material from being eroded.

3.1.3 Design of the grout curtain

The design of the grout curtain should be performed after the results of the site investigation have been analyzed and the requirements on the grout curtain have been defined, and the design should be done before the design of the grouting work. The grout curtain is preferably designed together with the dam body so that the curtain's contribution on the project cost could be accounted for. After this step, the position, thickness and depth of the grout curtain will be provided together with a recommendation on the grout mix. An illustration of parameters to be determined in the design of a grout curtain is presented in Figure 3-4. Due to the complexities and spatial variations in the rock mass quality, a grout curtain's geometry does not necessarily have to be constant along the dam. It is a risk if the design of the entire curtain is performed only based on assuming a single rock mass condition under the dam. Instead, it is important that the results from the site investigations are analyzed thoroughly and that the curtain is designed with respect to different geological domains that may be present in the rock mass.

At the beginning of the design of the grout curtain, a preliminary grout mix should be chosen, which will be the basis of the further design steps. The principle of grout mix choice is to allow the grout to penetrate in the fractures and to achieve a certain extent of sealing to lower the hydraulic conductivity of the rock mass as required.

Stille (2015) suggested that the penetrability of the grout should be evaluated according to the following steps: The first step is laboratory testing where the smallest aperture that the grout is able to penetrate (b_{min}) is measured together with

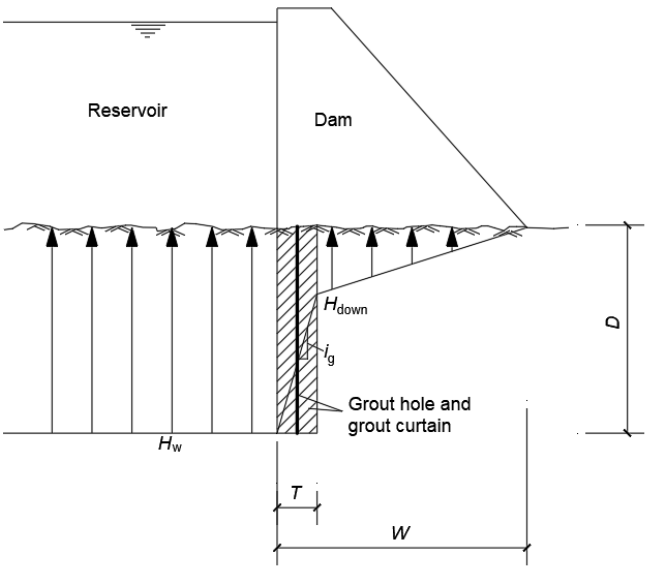


Figure 3-4: Illustration of the design of the grout curtain

the upper limit above which the grout can flow freely (b_{crit}). This step is vital for the grout mix design. In the second step, these limits must be related to the hydraulic aperture and compared with the measured hydraulic apertures. Stille (2015) mentioned that b_{min} could be estimated to be around 1/3 to 1/4 of b_{crit}.

The choice of grout mix could be verified by calculating the residual hydraulic conductivity using Eq.(3.11) (Stille 2015). This estimated residual hydraulic conductivity (K_g) should not be higher than the required expected residual hydraulic conductivity (K_{exp}) as mentioned in Section 3.1.2.

$$K_g = \frac{k_1 k_2}{k_3^3 k_4^3} \frac{1}{L} \frac{\rho g}{12\mu} b_{crit}^3 \tag{3.11}$$

where k_1 is the ratio between mean and maximum section transmissivity of all sectional tests in the project. k_1 value normally varies between 0.2 and 0.6 and indicates the overall tightness of the rock, as tight rock may lead to a small value of k_1 and a more permeable rock mass may give a higher value. k_2 is the ratio between maximum section transmissivity and maximum fracture transmissivity. k_2 normally varies between 1.1 and 2.0, as described in Section 3.1.1. The ratio between maximum physical aperture and maximum hydraulic aperture, described in Section 3.1.1, is expressed as k_3. k_3 normally varies between 1.5 and 2.0. k_4 is the ratio between measured critical aperture (b_{crit}) and the fracture aperture for which a fully sealed fracture can be expected, which in this report is defined as the boundary fracture aperture. L is the length of the borehole section.

Although an analytical expression exists to calculate K_g, large uncertainties exists in the determination of this parameter. The calculated K_g may differ with a factor of up to 40 (Stille 2015). As a solution to this problem, four ratios (k_1 to k_4) were employed, which makes it easier to locate the source of uncertainties and perform sensitivity analyses if necessary. Among the four ratios, the choice of k_4 is noteworthy. If k_4 is assumed to be 1, meaning that all the fractures with aperture above b_{crit} will be fully sealed, the calculated residual conductivity will be very small, which is ideal in terms of hydraulic conductivity reduction. Nonetheless, ideal conditions are seldom met in construction projects and a lower value should be chosen for k_4 for two reasons: (i) whereas no grout clogging will occur and 100% of the grout is expected to penetrate in fine fractures with aperture slightly above b_{crit}, it is still practically difficult to fully seal these fractures at a distance from the grout hole. The time for each grouting run is limited but it would take very long time and much effort having the ambition to fully seal the finest fractures. (ii) if a lower k_4 can fulfill the conductivity requirement, it is unnecessary to use a higher value, not to mention that not all the fine fractures are water-bearing within a section.

Location of the grout curtain

In principle, the grout curtain is located under the dam body. However, the exact position under the dams varies with the type of dam. For concrete dams, the existence of grout curtain helps to reduce the uplift pressure. In order to maximize this benefit, the grout curtain should be located close to the heel of the dam, but

preferably not reaching outside the projection of the dam body. Therefore, the position of the grout holes' row (or rows for multiple-row curtains) is also dependent on the thickness of the curtain due to the spread of the grout. In the design, an evenly distributed disk (2D) spread is assumed, which indicates that the grout holes' row is located half of the thickness of the curtain away from the dam heel. For embankment dams, the grout curtain is usually located under the core of the dam. A position near the centerline of the core but no further downstream is recommended. More detailed explanations could be found in Houslby (1990).

Thickness of the grout curtain

The thickness of the grout curtain is controlled by the requirements for two factors: uplift pore water pressure and hydraulic gradient in the curtain.

By assuming that the rock mass could be modelled as a continuum media which follows Darcy's law and assuming an equal amount of flow through the grouted (curtain) area and in the ungrouted area, a relationship between the lowered water head downstream the curtain (H_{down}) and the thickness of the grout curtain (T) could be calculated according to

$$H_{down} = \frac{b_{crit}^3 (W - T)}{b_{crit}^3 (W - T) + k_3^3 k_4^3 b_{mh}^3 T} H_w \tag{3.12}$$

where b_{mh} is the maximum hydraulic aperture from hydrogeological investigations, b_{crit} is the aperture above which the grout can flow freely, W is the width of the dam base, and H_w is the reservoir waterhead.

As shown in Figure 3-4, the total uplift force (F_{up}) could be calculated geometrically as

$$F_{up} = \left[\frac{H_{down}(W - T)}{2} + \frac{(H_{down} + H_w)T}{2}\right] \rho g \tag{3.13}$$

In order to find the thickness of the grout curtain that gives the smallest uplift force, the first derivation of F_{up} over T should be equal to 0, as shown in Eq.(3.14). The thickness that is needed could be found by solving the function of T. This could also be done with a trial and error procedure. Similar concept on finding the optimum thickness of grout curtain was described by Chai and Cui (2012).

$$\frac{dF_{up}}{dT} = 0 \tag{3.14}$$

After obtaining the thickness of the grout curtain (T) with respect to the uplift water pressure, the second factor, the hydraulic gradient in the curtain (i_g) should be taken into consideration as another controlling factor. The purpose of this procedure is to ensure that internal erosion of weak infilling materials in the grouted fractures cannot be initiated. At the present time, there is still no satisfying method to find the critical gradient for erodible materials in rock fractures against intrusion flow. Axelsson (2009) provided a recommendation on calculating the critical gradient (i_{crit}) based on the critical velocity of different riverbed materials against erosion. However, a mistake was made for the erosion resistance of cohesive soil when referring to the well-known Hjulström graph (Hjulström 1935),

causing a ten-time lower resistance thus a significant lower critical velocity. It could also be questioned if the analogy between riverbed hydraulic conditions and the hydraulic conditions in fractures is appropriate. The implementation of Axelsson's recommendations is therefore not encouraged. ICOLD (2017) reported different thresholds for different types of erosion that may occur inside embankment dams. The recommendations on contact erosion given in this bulletin may be used as a first rough assumption on the problem with internal erosion of fracture infilling materials during grouting. Currently, an ongoing research project within the Swedish Hydropower Centre (SVC) is aiming to investigate the erosion and critical gradient of infilling materials in the fractures.

The relation between the thickness of the grout curtain (T) and the hydraulic gradient in the curtain (i_g) can be derived based on Eq.(3.12) and is presented in

$$i_g = \frac{H_w - H_{down}}{T} = \frac{k_3^3 k_4^3 b_{mh}^3}{b_{crit}^3 (W - T) + k_3^3 k_4^3 b_{mh}^3 T} H_w \qquad (3.15)$$

If the calculated i_g from T is not higher than i_{crit}, the T can be used as the final thickness of the grout curtain. If on the contrary i_g is higher than i_{crit}, a new thickness needs to be back calculated from Eq.(3.15), and this new thickness shall be used.

Depth of the grout curtain

The flow path at larger depths in the rock mass becomes longer, which implies a lower gradient and a lower flow rate, as shown in Figure 3-5. To reduce the water flow under a certain limit, the grout curtain does not necessarily need to be very deep. The depth of the grout curtain is recommended to be determined according to the hydrogeological investigations. If the results indicate that the hydraulic conductivity below a depth of the rock mass is already lower than the required value (K_{exp}), this boundary depth could be the depth for grout curtain (D).

The depth of the grout curtain should be determined with caution when the results from the hydrogeological investigations do not show a clear decreasing conductivity with depth. Then, the empirical limit could be used under such conditions, for example according to Houlsby (1990) or RIDAS (2011). When unexpectedly permeable rock mass is found, the grout curtain should be deeper than the permeable zones.

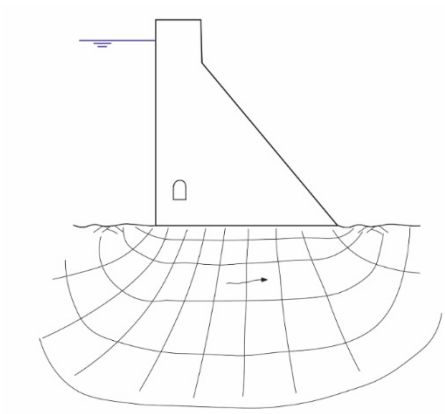


Figure 3-5: Flow net under a concrete dam without a grout curtain

3.1.4 Design of the grouting work

Varying rock mass formations can lead to different geometry of the grout curtain, meaning that the design of the grouting work can also vary along the dam. Grout holes in different geological domains or in different rock mass qualities will be given different design for the grouting. A suggested design process for design of the grouting work could be described as follows:

1. Use the preliminary grout mix (more than one grout mix can be used throughout the design) chosen in the design of the grout curtain. Test the grout properties which are important for the grouting work. The same type of grout could be used throughout the grouting process in the grout hole of interest whereas the grout mix is subject to changes under different conditions.
2. Set an initial grouting pressure for each section of grouting in every grout hole. Estimate the grout spread length (in the boundary fractures) curve versus time for each section depth in every hole. Based on this curve, a reasonable stop criterion with respect to time and corresponding grout spread length shall be chosen. The pressure and spread length should be subject to pressure check against potential unfavorable displacements (jacking) caused by the grouting.
3. Design the spacing of the grout holes based on the estimated grout spread length. The hole-spacing should be able to result in the designed thickness of the curtain.
4. Design the monitoring plan. Prepare alarm limits for unfavorable behaviors and the corresponding contingency plans. Set up the complete criteria for the grouting work.

Although the four steps are sorted, it is not a linear design, meaning that designers may have to go back from one step to a previous one to refine the previously determined values and continue with the design with new values until an optimum design is achieved.

Different aspects which were included in the suggested design process will be described in the following pages. This report mainly focuses on the first three steps. The fourth step will also be described, but not in depth.

Grout properties

Following the preliminary choice of grout mix presented in Section 3.1.3, further test on grout properties should be performed. Apart from penetrability, the rheological properties of the grout should also be tested to predict the grout spread and flow. The rheological properties needed for grout include the yield value (τ_0) and its viscosity (μ_g) given the assumption that the grout could be modeled as a Bingham fluid.

Prediction of grout spread and flow under constant injection pressure

The rheological properties of cement-based grout mixes can be assumed to behave according to a Bingham fluid, which means that the shear stresses must exceed a

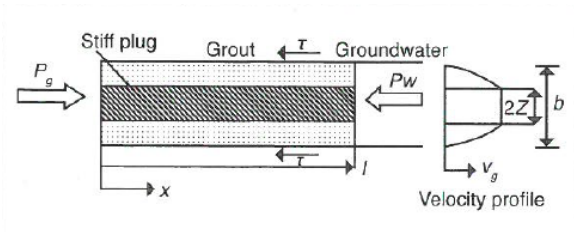


Figure 3-6: Grout flow through a fracture (from Gustafson 2012)

yield value (τ_0) for the fluid to flow, see Figure 3-6. This model has been found to describe the grout spread in fractures (2D discs for normal grouting cases) with satisfying accuracy (Funehag and Thörn 2018). During the grouting process, the grouting pressure, rather than the grout flow, can be directly controlled. The theories about grout spread is mainly based on the assumption of a constant injection pressure. At this point of the design, an grouting pressure (P_g) can be chosen to start the calculation and it may be subject to change if the grout spread is too short or if the pressure is so high that it could trigger the hydraulic jacking (this check will be introduced in detail in the following part).

Under the assumptions that the grout mixes behave as a Bingham material, the grout will stop spreading when the shear stress in the grout is consistently lower than the yield value (τ_0). At this limit state, the shear stress at the fracture boundaries can be seen as equal to the yield value. By establishing the force equilibrium, the maximum grout spread (I_{max}) can be calculated as (Gustafson and Stille 1996)

$$I_{max} = \left(\frac{\Delta P_g}{2\tau_0}\right) b \tag{3.16}$$

$$\Delta P_g = P_g - P_w \tag{3.17}$$

where ΔP_g is the effective grouting pressure as shown in Eq.(3.17), b is the physical aperture of the fracture, which can be approximated as 2 times (Stille 2015) of the hydraulic aperture (b_h). Nonetheless, different fractures have different apertures which gives different maximum spread of the grout. The boundary fractures influence the quality of the grout curtain in terms of hydraulic conductivity reduction. This will also influence which sealing efficiency that could be reached with a well-executed grouting. Therefore, calculation is necessary to use boundary fracture aperture instead of b in Eq.(3.16). Shorter spread length is usually expected in finer fractures, thus in order to achieve a certain thickness of the grout curtain, the spread length in the boundary fractures that the grout is able to penetrate usually controls the grout hole layout.

Knowing the maximum grout spread, it is then important to find the time needed to reach the required grout spread. A relation between grout spread and time is thus required. Gustafson et al. (2013) provided an analytical solution describing time as a function of grout spread length. In their solution, relative grouting time (t_D, defined as real time t divided by characteristic time t_0) and relative grout spread (I_D) were introduced:

$$t_D = \frac{t}{t_0} \tag{3.18}$$

where t_0 is:

$$t_0 = \frac{6\Delta P_g \mu_g}{\tau_0^2} \tag{3.19}$$

where μ_g is the viscosity of the grout, and.

$$I_D = \frac{I}{I_{max}} \tag{3.20}$$

The ratio (γ) between the maximum grout spread (I_{max}) and the radius of the borehole (r_b) influences the grout spread over time, but to a rather small extent. The solution was given for time as function of grout spread by Stille et al. (2009), where

$$I_D = \sqrt{\theta^2 + 4\theta} - \theta \tag{3.21}$$

where for a 2D radial spread:

$$\theta = \frac{t_D}{2(3 + t_D + 0.23 \ln t_D)} \tag{3.22}$$

Another solution for the 2D case was given by Eriksson and Stille (2005) as

$$I_D = 0.580 + 0.450 \log t_D + 0.135(\log t_D)^2 + 0.015(\log t_D)^3 \tag{3.23}$$

The difference between the results from the two solutions (Eq 3.21, 3.22 and Eq. 3.23) is small, the second solution is easier to use for the prediction of the flow since its derivative is easier to calculate, which will be described in the following paragraphs. It is recommended that a curve being plotted showing the relation between the real grout spread (I) and the real grouting time (t). It helps designers to find a realistic grout spread since the maximum spread (I_{max}) would only be achieved after an infinite period of time. This curve also helps finding the stop criteria, which in this design method is grouting time required to reach the desired grout spread.

It is worth mentioning that there is still discussion on the correct way to calculate the spread of a Bingham material in a 2D disc (El Tani and Stille 2017, El Tani 2012), although the difference among the solutions is negligible (Stille 2015).

Stille (2015) suggested that the total injected grout volume in a 2D fracture (V_{tot}) to be calculated according to the geometrical volume of a disc (note that here the physical aperture is to be used) as

$$V_{tot} = \pi I^2 b = \pi (I_D \cdot I_{max})^2 b = \pi I_D^2 \left(\frac{\Delta P_g}{2\tau_0}\right)^2 b^3 \tag{3.24}$$

For a section consisting of several fractures, total volume can be calculated as

$$V_{tot} = \pi I_D^2 \left(\frac{\Delta P_g}{2\tau_0}\right)^2 \cdot \sum b^3 \tag{3.25}$$

Thus, the flow (Q) in a 2D fracture assuming radial spread can be described as

$$Q = \frac{dV_{tot}}{dt} = 2\pi I_D \frac{dI_D}{dt_D}\frac{1}{t_0}\left(\frac{\Delta P_g}{2\tau_0}\right)^2 \cdot \sum b^3 \tag{3.26}$$

Control of grouting pressure with respect to hydraulic jacking

The grouting pressure must exceed the ground water pressure for the grout to flow. The choice of grouting pressure affects the deformation in the rock fractures and can cause hydraulic jacking if the pressure is too high. A high grouting pressure can drive the grout to spread farther and lead to enlarged aperture and thus seems preferable. However, these effects also indicate that the expected sealing efficiency could be difficult to obtain which will possibly result in an unfavorably higher residual hydraulic conductivity of the rock mass after grouting and longer grouting time (Rafi and Stille 2015b). Therefore, the grouting pressure should be determined properly with respect to the requirements for maximum allowed deformation and ultimate jacking.

Jacking of horizontal fractures during grouting is expected to be initiated when the grouting pressure (P_g) exceeds the critical pressure P_i (usually the depth of the fracture times the unit weight of the rock mass), as shown in Figure 3-7. In zone 1 in Figure 3-7, a low effective grouting pressure could not detach the contacted asperities. When P_g exceeds P_i, as shown in zone 2, the fracture opens up. Once the grouting pressure enters zone 2, the deformation depends not only on the pressure and elastic modulus of the rock mass, but also on the radius of grout spread (I). For example, in Figure 3-7, if grout spread $I_2>I_1$, for a given grouting pressure P and the same elastic modulus of the rock mass, longer grout spread I_2 leads to larger deformation δ_2.

An explanation on the mechanism of elastic jacking can be found in Figure 3-8. In the figure, the pressure in the spreading grout is assumed to be linearly decreasing from the grout hole (maximum pressure) to the grout spread front (zero pressure). Only when the grouting pressure (P_g) exceeds the critical pressure (P_i), the pressurized grout starts to initiate elastic jacking. The difference between P_g and P_i is the excessive pressure P_e (as described in Eq.(3.27)) and it is the cause of elastic jacking. Within the direct acting area of P_e (circular area with a radius of r_c if 2D radial grout spread is assumed) on an infinite persistent fracture, the deformation of the fracture (called jacking deformation Δa_j) is almost constant from grout hole to r_c. Outside of r_c the excessive pressure indirectly deforms the fracture through stress redistribution in the neighboring area, resulting in a relationship between the deformation Δa and the distance from the point of concern to the grout hole (r).

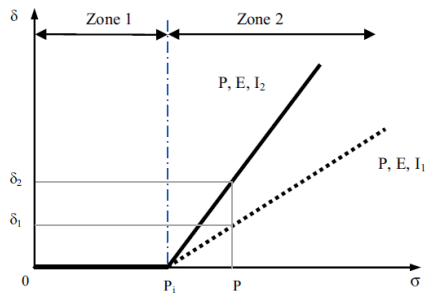


Figure 3-7: Deformation of a fracture in relation to the grouting pressure (from Rafi and Stille 2015b). No deformation occurs in Zone 1 while in Zone 2, longer grout spread (I_2>I_1) results in larger deformation (δ_2>δ_2) of the rock mass for a given *P* and *E*.

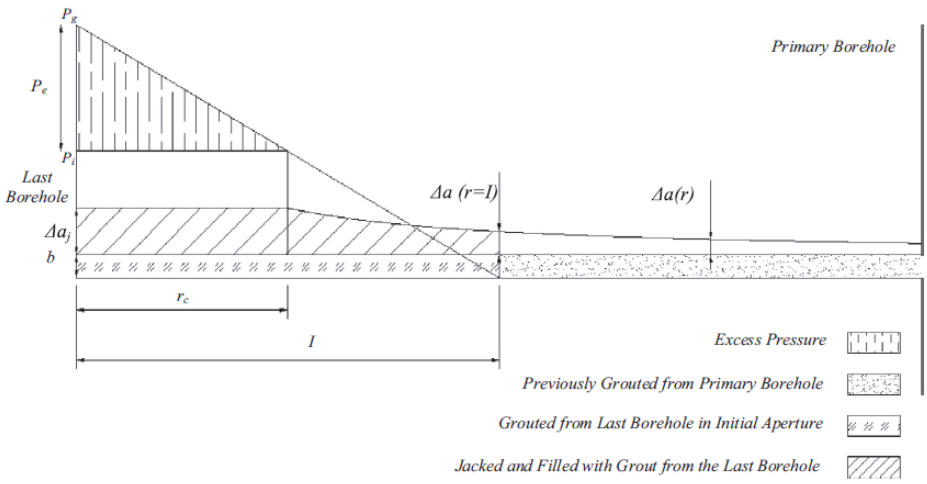


Figure 3-8: Mechanism of elastic jacking (from Rafi and Stille 2015b)

The excessive pressure is expressed as

$$P_e = P_g - P_i \tag{3.27}$$

Gothäll and Stille (2009) obtained an expression of the deformations along a fracture at a sufficiently large distance from the borehole (r>>0):

$$\Delta a(r) = \frac{4}{3}\frac{P_e r_c^2}{Er}(1-\nu^2) \tag{3.28}$$

where E is the elastic modulus of the rock mass, ν is the Poisson's ratio of the rock mass, r_c is the radius of the area on which the excessive pressure (P_e) is directly acting, r is the radial distance from the point of concern to the grout hole. For the infinite persistent fracture, the constant jacking deformation (Δa_j) can be expressed as

$$\Delta a_j = \frac{4}{3}\frac{P_e r_c}{E}(1-\nu^2) \tag{3.29}$$

If the pressure is assumed to decrease linearly from the borehole to the spread front, the grout spread (I) can be expressed as Eq.(3.30) and the jacking deformation can be further expressed as Eq.(3.31) (Stille et al. 2012). As indicated in Figure 3-8,

the deformation within radius r_c (Δa_j) is the maximum deformation for a given grout spread and thus should be compared to the maximum allowed deformation (δ_{acc}).

$$I = \frac{P_g}{P_e} r_c \tag{3.30}$$

$$\Delta a_j = \frac{4}{3} \frac{(1-\nu^2)}{E} P_e I \frac{P_e}{P_g} \tag{3.31}$$

For comparison, the old Swedish practice described by Statens Vattenfallsverk (1968) suggested that δ_{acc} should be approximately 0.2-0.3 mm.

Instead of calculating the resulting deformation, another alternative to determine the acceptable grouting pressure was recommended by Stille et al. (2012). This alternative relates the grouting pressure with the grout spread. Predefined maximum allowed deformation (δ_{acc}) leads to a relation between grout spread (I) and grouting pressure (P_g). The normalized effective grouting pressure (P_n) and the normalized grout spread (I_n) are defined as

$$P_n = \frac{\Delta P_g \cdot k_2'}{3\rho g h} \tag{3.32}$$

$$I_n = \frac{I}{h} \tag{3.33}$$

where ΔP_g is the effective grouting pressure, k'_2 is a parameter indicating the non-contacted part of the fracture, usually assumed to be approximately 1, ρ is the density of the rock mass and h is the depth of the fracture.

By normalizing Eq.(3.31), this limit can be presented as

$$P_n + \frac{P_w}{3\rho g h} \leq \frac{k}{3 I_n} + \frac{1}{3} \tag{3.34}$$

where P_w is the water pressure in the fracture, k is equal to

$$k = \frac{3}{4} \frac{E}{(1-\nu^2)} \frac{\delta_{acc}}{\rho g h^2} \frac{P_g}{P_e} \tag{3.35}$$

An example of this limit is plotted in Figure 3-9 as "Acceptable jacking".

Apart from controlling the deformation within the acceptable level, the uncontrolled ultimate jacking should also be avoided. Based on the principle of GIN method, Brantberger et al. (2000) developed an expression that presents the relation between normalized effective grouting pressure (P_n) and normalized grout spread (I_n) under this requirement, later modified by Stille et al. (2012) as shown in Eq.(3.36). This limit is plotted in Figure 3-9 as "Ultimate jacking".

$$P_n + \frac{P_w}{\rho g h} \leq 1 + \frac{1}{I_n} + \frac{1}{3 I_n^2} \tag{3.36}$$

In general, the grouting pressure should not exceed (or intersect) either of the two limits in Figure 3-9 to ensure that no unfavorable jacking occurs.

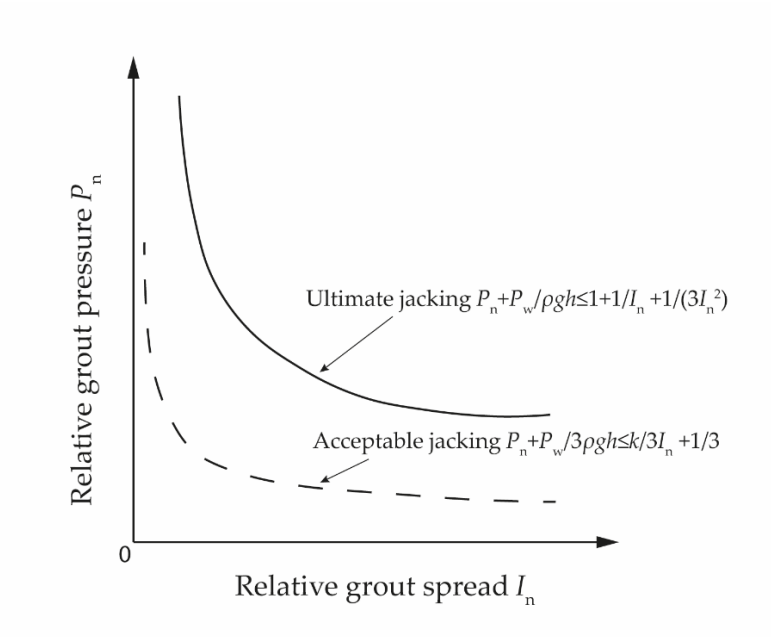


Figure 3-9: Relation between grouting pressure and grout spread for the limits of ultimate jacking and acceptable jacking (after Stille et al. 2012)

Spacing between grout holes

Having the calculated grout spread curves versus grouting time in the boundary fractures and the expected thickness of the grout curtain (T) from Section 3.1.3, the designer can start to design the spacing between the grout holes. The basic principles are presented graphically in Figure 3-10. In the figure, the overlapping of the grout spread determines the thickness of the grout curtain. Thus, the spacing between the grout holes can be determined based on the geometrical calculation of the calculated grout spread.

Since the required thickness of the grout curtain is determined previously in the design of the grout curtain, it is necessary to determine which grouting pressure and grouting time that should be used to obtain the required thickness given a certain spacing between the gout holes. The grout spread curves versus grouting time in the boundary fractures are able to give different spread lengths with different grouting time under different pressures. Several distances between the grout holes

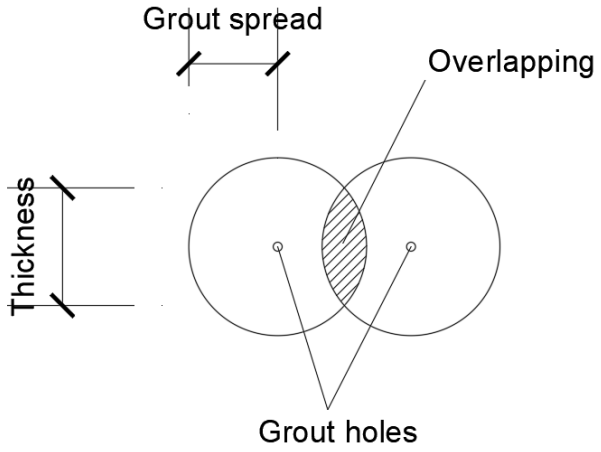


Figure 3-10: Overlapping of the grout spread determines the thickness of the grout curtain

could be found for the same grouting section in order to obtain the desired thickness. In general, two options are available: (i) use a higher grouting pressure and a longer grouting time for each hole, which results in longer grout spread and thereby a larger required spacing between the holes, or (ii) use a lower grouting pressure and a shorter grouting time for each hole, which results in a shorter grout spread and smaller spacing between the holes. Between the two options, it is the designers' decision on which one to choose. The decision could be related to many aspects including costs and equipment. A cost optimization could be done by the designers in order to obtain the most optimal distance between the grout holes for the required thickness.

Grouting work in shallow sections of a grout hole is prone to jacking. Thus, in these sections the grouting pressure and the grout spread are limited by the jacking criteria described in the previous section. For a certain thickness (overlapping), the hole spacing required in shallow sections is therefore usually needed to be smaller than in deeper sections of the grout hole where higher grouting pressure is applicable. In such cases a sequenced grout hole layout can be implemented, where the layout of the primary holes could be designed according to their deeper sections whereas in the shallower sections of the primary holes, the design should share the same parameters as the secondary holes in terms of grouting pressure, time and other parameters. Smaller spacing is thereby usually chosen between adjacent primary and secondary holes.

Under ideal conditions, the designed layout would fulfill all requirements and no additional grout holes are needed. However, ideal conditions are hard to find in a real rock mass. It is therefore necessary to always verify that the required sealing of the rock mass has been obtained during the grouting execution in line with the principles of the observational method. If water pressure tests conducted in control holes after the grouting show that additional grout holes are needed, a higher order of grout holes should be performed. These holes should be planned and designed in advance according to the same principles as for the other holes.

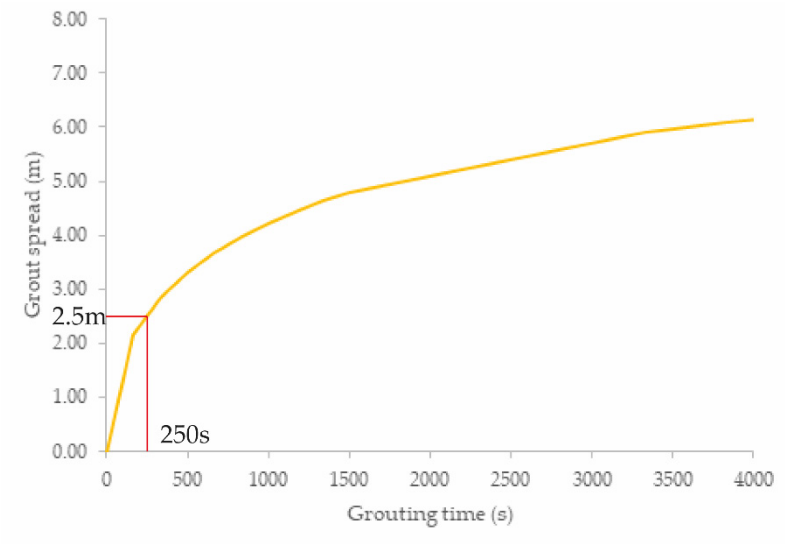


Figure 3-11: An example of time as the stop criteria for the grouting with a section

Stop criteria

In comparison to the widely used empirical grouting methods, which usually take flow-vs-time as stop criteria (or "refusal"), the recommended stop criteria in this report is the time and volume of injected grout.

As soon as the grout spread reach the anticipated spread length (I_{stop}) in the boundary fractures within the section that is to be grouted, the grout process for this section should stop. In practice, the grouting of a section can be stopped when relative grout spread reaches around 0.4. Since drilling and grouting new grout holes is more efficient compared to continuing grouting for long time to get a slightly longer penetration. The corresponding time (t_{stop}) to reach I_{stop} thus indicates the stop of the grouting in the section. An example is given in Figure 3-11, where the required spread length is 2.5m. The corresponding time is about 250s, which then becomes the stop criteria for the grouting in this section.

The expected volume of injected grout can also be set a stop criterion. This volume is calculated through Eq.(3.25) and adding some additional margin to fill the grout hole. Both the grouting time and the injected volume should be recorded during the grouting work, when both criteria are met, the grouting should stop and the grouting work in a borehole can be considered to be finished.

Alternative designs, contingency plans and complete criteria

Alternative designs should be prepared to deal with the possible range of geological uncertainties that is judged to exist at the site. They should be implemented when rock mass properties or other design factors are detected during the grouting execution that lies outside the range of the main design, so that the grouting plan can be quickly changed to deal with the new situation.

A proper contingency plan, including a monitoring plan, alarm limits for unfavorable behaviors and the corresponding contingency actions, needs to be

Table 3-1: Example of a brief contingency plan for hydraulic jacking during grouting

Target behavior	Hydraulic jacking
Monitoring plan	Extensometers shall be installed in the areas that are prone to hydraulic jacking or in the locations where rock displacement are strictly restricted. The staffs in the grouting team should observe the extensometers and record the displacements during each grouting round. Flow rate should be recorded when grouting in each round by reading the flowmeters.
Alarm limit	Maximum vertical uplift displacement should be no more than 0.2mm. Flow rate witnesses an increase from the flow prediction.
Contingency action	Once a displacement over 0.2mm and/or a corresponding flow increase are recorded, hydraulic jacking is considered likely to occur. Grouting pressure should immediately be lowered until the displacement is reduced below 0.2mm. The grouting should be stopped if the displacement does not recover. In such case, a thorough check and adjustments on the preliminary design should be performed.

established for the grouting team to follow. Many issues shall be paid attention to e.g. hydraulic jacking, variation in fracture properties in the rock mass, clogging of grout, overly high grout take etc. A complete plan is required for each of the concerned issues that could arise during the grouting process. The plan for each issue needs to be prepared separately. An example concerning hydraulic jacking is given below in Table 3-1. Note that the plan has been simplified and the real design must be more thorough and detailed.

Complete criteria are also desired in the designing phase as a signal indicating that the entire grouting work for the dam project is completed. One of the options for such criterion is the residual hydraulic conductivity as mentioned above. By drilling the control holes close to the already grouted curtain area and performing Lugeon tests, the grouting personal can get values of the residual hydraulic conductivity in the grout curtain. The criteria are, therefore, that this tested residual hydraulic conductivity is lower than the pre-defined requirement described in Section 3.1.2. If the criteria are fulfilled for all grouted areas of the grouting project, the grouting work can be seen as completed and the grout curtain is constructed. If the criteria are not fulfilled, supplementary grouting including higher order of grouting holes or additional rows of grouting may be performed according to the site's geological and hydrogeological conditions.

3.2 GROUTING EXECUTION

Provided with a complete preliminary design for the grout curtain, the grouting personals (contractors) must follow the design in most of the conditions where the natural rock mass properties are not deviating too much from the interpretations based on site investigations. During the execution, the grouting team should be able to perform the grouting with high quality and with good control during the whole process. Successful application of the observational method demands good judgements (to discover and locate the problems) and quick response (with

effective and flexible applications of the contingency actions) when abnormal events occur during the construction.

Grouting work is preferably executed by competent teams who have extensive experience on grouting and good understanding of the grouting process. Proper and strict supervision should also be performed from different parties including clients and designers.

3.3 LONG-TERM MONITORING

For high consequence dams, the functions of the grout curtains are usually subject to long term indirect monitoring through measurements of the pore pressure in the foundation. In such cases, a proper monitoring program should be utilized.

It is important to keep monitoring the pore pressure in the rock foundation throughout the service life of the dam. If the pore pressure in the rock foundation increases, which may indicate a degradation of the grout curtain, or the grout curtain shows other signs of degradation, it may be necessary to perform remedial grouting.

In principle, monitoring should be also be performed with respect to the hydraulic gradient over the grout curtain, even though this is rarely done in practice. A high hydraulic gradient may potentially erode the grout curtain and thus threaten the durability of it. Hydraulic gradients higher than the critical value should be seen as a warning signal and an indication that remedial grouting is necessary. A low local gradient may also indicate problems that the flow in other areas may become high. The hydraulic gradient can be monitored in detail by installing piezometers at both the upstream side and the downstream side of the grout curtain. However, this is rarely done in practice and in most of the cases a full reservoir pressure is assumed upstream of the grout curtain and the pore pressure is only measured downstream of the curtain.

4 Example of preliminary grout curtain design

In this chapter, a design example of a grout curtain under a new dam will be presented, using the design method from Chapter 3. This design example will only include the preliminary design phase in the flow chart in Figure 3-2, as the execution of the grouting is expected to follow the recommended principles in the previous chapter.

4.1 BACKGROUND

The dam project in this example is fictitious. All parameters related to the dam and the geological conditions in the rock mass are assumed but are reasonable for a fractured hard rock mass in crystalline rock. The geometry of the dam and the rock mass conditions are presented in Figure 4-1. Assumptions related to the project include:

- The dam to be built is a concrete gravity dam. Its height is 20 m with a tailwater head at the rock surface ($\pm$ 0 m). The width of the dam (W) is 20 m.
- Before the construction of the new dam, the ground water level is assumed to be at the rock surface ($\pm$ 0 m).
- The water head (H_w) is assumed to be 20 m above the ground when the reservoir is filled, approximately the same as the height of the dam.

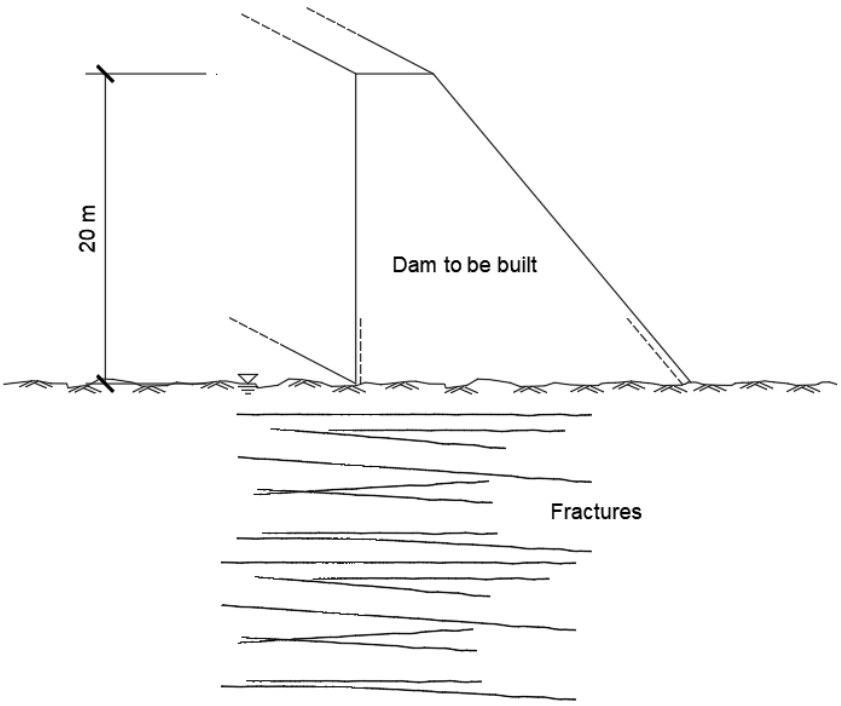


Figure 4-1: Section through the dam and the foundation rock mass

- Simplified conditions of the fracture orientations and their locations are assumed. The rock mass properties are assumed to be constant along the dam body.
- The exploratory holes and the grout holes are vertical and the sections for water testing and grouting are assumed to be 4 m in length. Sections are distributed as follows: 0-4 m, 4-8 m, 8-12 m, 12-16 m, 16-20 m, 20-24 m and 24-28 m, in order to examine the necessity of grouting even for depths deeper than the dam height.

4.2 PRELIMINARY DESIGN

4.2.1 Geological and hydrogeological investigations

Geological investigations were performed in the pre-investigation stage, the rock mass properties are presented in Table 4-1.

Table 4-1: Rock mass properties

E (GPa)	Poisson's ratio ν	γ (kN/m3)	Infilling material
40	0.2	26	Fine sand for all depths, fully filled

The hydrogeological investigation was done by performing Lugeon tests. The records of one of the exploration holes are presented in Table 4-2.

Table 4-2: Records of water loss measurement in hole nr.1

Hole nr	Measurement depth in rock		Test length (m)	Test pressure (kg/cm² or bar)	Time (min)	Water loss (liter)
	from (m)	to (m)				
	0	4	4	3	2	40
	4	8	4	3	2	30
	8	12	4	3	2	35
1	12	16	4	3	2	15
	16	20	4	3	2	8
	20	24	4	3	2	2
	24	28	4	3	2	1

By applying Eq.(3.1) to (3.3), and (3.7), the hydrogeological properties and the estimated fracture apertures can be calculated. The results from the calculations are presented in Table 4-3. In the calculation, the coefficient k_2 is assumed as 1.25.

Table 4-3: Hydrogeological properties within sections and estimated apertures

Depth in rock (m)		Lugeon value (liter/m·min·MPa)	Conductivity (m^3/s·m·m)	Section transmissivity T_s (m^3/s·m)	Estimated maximum hydraulic aperture b_{mh} (m)	Estimated maximum physical aperture b_m (m)
0	4	17	2.78E-06	1.11E-05	0.000240	0.000480
4	8	13	2.08E-06	8.33E-06	0.000218	0.000437
8	12	15	2.43E-06	9.72E-06	0.000230	0.000460
12	16	6	1.04E-06	4.17E-06	0.000173	0.000346
16	20	3	5.56E-07	2.22E-06	0.000140	0.000281
20	24	1	1.39E-07	5.56E-07	0.000089	0.000177
24	28	0	6.94E-08	2.78E-07	0.000070	0.000140

The results shown in Table 4-3 indicate that with the increase of the depth, the rock mass around the exploration holes has decreasing hydraulic conductivity and smaller estimated fracture apertures. In this design example, it is assumed that the rock mass near this exploration hole (including the areas where the grout holes are to be drilled) share the same property and the design of the grout curtain is to be performed only for this region. In real projects, intensive geological and hydrogeological investigations are desired and different results should lead to different designs for different regions. Grout hole testing is omitted in this calculation for simplicity, thereby this design is performed based on the exploration testing.

The grout mix that is going to be injected was pre-determined to INJ 30. The viscosity of the water and the grout mix properties (INJ 30 with a w:c of 0.8) are shown in Table 4-4 and Table 4-5 respectively, the data is from Stille (2015).

Table 4-4: Properties of water

Dynamic viscosity μ (Pa·s)	γ_w (kN/m^3)
0.0013	10

Table 4-5: Properties of grout mix (INJ 30 with a w:c ratio of 0.8)

Viscosity μ_g (Pa·s)	Yield shear strength τ_0 (Pa)	b_{min} (µm)	b_{crit} (µm)
0.02	6	62	90

4.2.2 Requirements on the grout curtain

According to Section 3.1.2, the requirements on the grout curtain are defined according to Table 4-6.

Table 4-6: Requirements on the grout curtain

Residual conductivity (Lugeon)	Uplift pressure	Internal erosion of infilling materials (fine sand)
1	Minimum	Avoid

4.2.3 Design of the grout curtain

Choice of grout mix

The choice of the grout mix should ensure that the first requirement on the grout curtain is fulfilled. The preliminary choice of grout mix (in this case INJ30 with a w:c ratio of 0.8) is subject to a check at this stage with respect to the expected residual hydraulic conductivity (K_g). The check is performed by using Eq.(3.11). The values used and the result are presented in Table 4-7. In the table, k_4 is 0.45 meaning that the boundary fracture aperture is assumed to be 200 µm, this aperture will also be the referencing aperture for the grout spread calculation.

Table 4-7: Check of grout mix INJ30

k_1	k_2	k_3	k_4	K_g	Lu	Result
0.47	1.25	2	0.45	9.37E-08	0.6	OK!

The design of grout curtain starts with the thickness of the curtain (T). Different values of thickness of the grout curtain are listed firstly and the corresponding uplift forces acting on the dam base are calculated following Eq.(3.12) and (3.13). This step is followed by the control of the hydraulic gradient with respect to internal erosion of infilling material according to Eq.(3.15). The critical gradient is in this example assumed equal to the critical velocity for contact erosion in embankment dams, where v_{crit} is equal to approximately 0.01 m/s (ICOLD 2017). As discussed in chapter 3, this value may not be correct, but constitute a reasonable estimation.

Eq.(4.1) is used to calculate the critical gradient based on the critical velocity. k_{sand} is the hydraulic conductivity of the fine sand, which is equal to 0.001m/s. The calculation is presented in Table 4-8. Although most of the thicknesses fulfill the requirements on preventing internal erosion, the 4 m thickness gives the optimum reduction of the uplift and thus chosen as the grout curtain thickness in this example.

$$i_{crit} = \frac{v_{crit}}{k_{sand}} \tag{4.1}$$

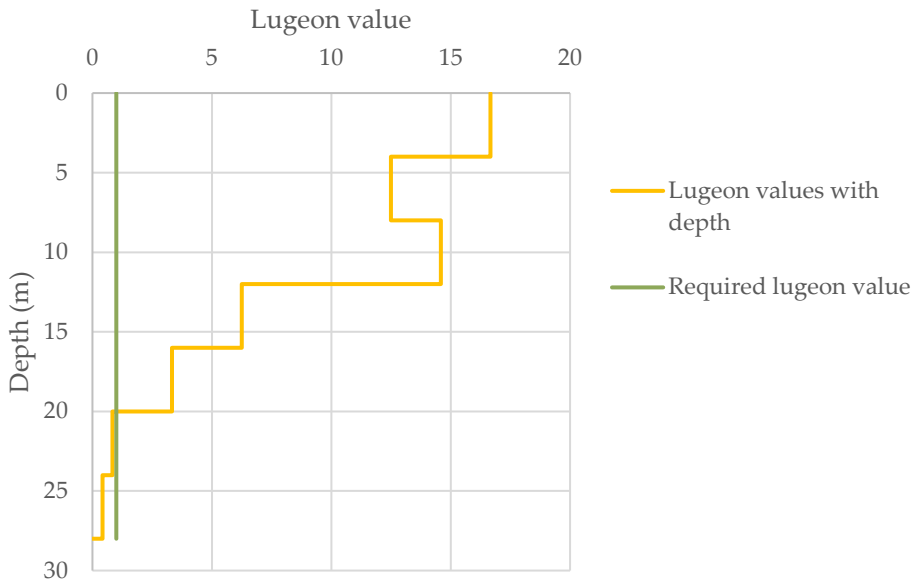


Figure 4-2: Determination of the depth of the grout curtain

Table 4-8: Determination of the thickness of grout curtain

T (m)	b_{mh} (m)	b_{crit} (m)	H_{down} (m)	F_{up} (kN/m)	i_g (m/m)	i_{crit} (m/m)
0			20.0	2000.0	-	
1			11.6	1256.2	8.4	
2	0.00024	0.00009	7.9	987.2	6.1	10
3			5.8	880.2	4.7	
4			4.5	847.8	3.9	
5			3.6	855.7	3.3	

After the thickness is chosen, the depth of the grout curtain is to be determined. The method to determine the depth is described in Section 3.1.3. For this design example, the process is presented in Figure 4-2. The depth of the grout curtain is based on that methodology determined to be 20 m.

4.2.4 Design of the grouting work

Prediction of grout spread, flow and stop criteria

The grout spread (I) in relation with time is to be calculated after the grout mix is decided. The design calculation follows the method from Section 3.1.4. First, the maximum grout spread (I_{max}) and the corresponding characteristic grouting time (t_0) are found by applying Eq.(3.16) and (3.19), see Table 4-9. In this design example, only sections of 0-4 m and 4-8 m are designed, the deeper sections could follow the same grouting plan as the 4-8 m section. Effective grouting pressure is initially chosen to 0.2 MPa for the 0-4 m sections due to its shallow depth and 0.5 MPa for

Table 4-9: Calculation parameters for I_{max} and t_0

Section	Effective grouting pressure ΔP_g (MPa)	Yield shear strength τ_0 (Pa)	Boundary fracture aperture b (m)	Estimated maximum spread I_{max} (m)	Grout viscosity μ_g (Pa·s)	Characteristic grouting time t_0 (s)
0-4m	0.2	6	0.0002	3.33	0.02	667
4-8m	0.5		0.0002	8.33		1667

the 4-8 m sections. A control against jacking will be performed on the grouting pressures in the following step of the design.

The grout spread against time (I vs t) can then be analyzed and plotted with the aid from the relative grout spread (I_D) and relative grouting time (t_D). Eq.(3.18) to (3.23) are employed for the analyses. Results are presented for both sections with corresponding stop criteria in Figure 4-3 and Figure 4-4. As shown in the figures, the stop criterion for grouting in the 0-4 m sections is 200 s of grouting time, giving a grout spread of 1.3 m (with corresponding relative spread of 0.4). The stop criterion for grouting in the 4-8 m section is 500 s of grouting time, giving a grout spread of 3.3 m (with corresponding relative spread of 0.4). It should be clarified that the stop criteria are based on the grout spread in the boundary fracture with aperture assumed to be 200 μm.

On the other hand, it is the grout spread in the fracture with maximum aperture within a section that control the jacking behavior and grout injection volume. In this example, the maximum fracture aperture within a section has been found by interpreting the Lugeon tests, as in Table 4-3. The corresponding grout spread at the moment grouting stops in the fracture with maximum aperture within section 0-4 m is 3.2 m, while for section 4-8 m it is 7.3 m. Consequently, grout overspread could occur, potentially creating problems such as grout spread into adjacent grout holes, which could clog them. One of the options to deal with this problem is to drill the holes in time and space sequences. For instance, the first drilling/grouting cycle could include every other primary hole, leading to distances between the primary holes that are twice as large. When the drilling/grouting in the first cycle have finished and the grout has developed part of its strength, the rest of the primary holes can be drilled and grouted in the second cycle. The same procedure could be used by the drilling and grouting for the secondary holes or holes of higher order. In addition, alternative designs and contingency plans based on the principles of the observational method shall also be prepared to deal with any other potential risks of grout overspread that may be identified.

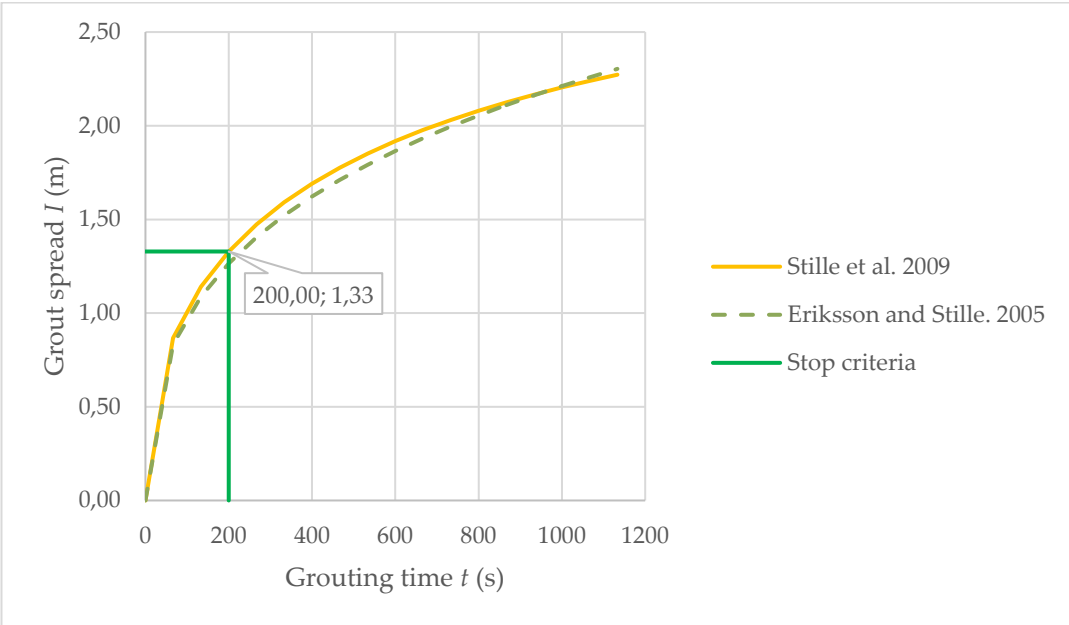


Figure 4-3: Grout spread vs grouting time for section 0-4 m

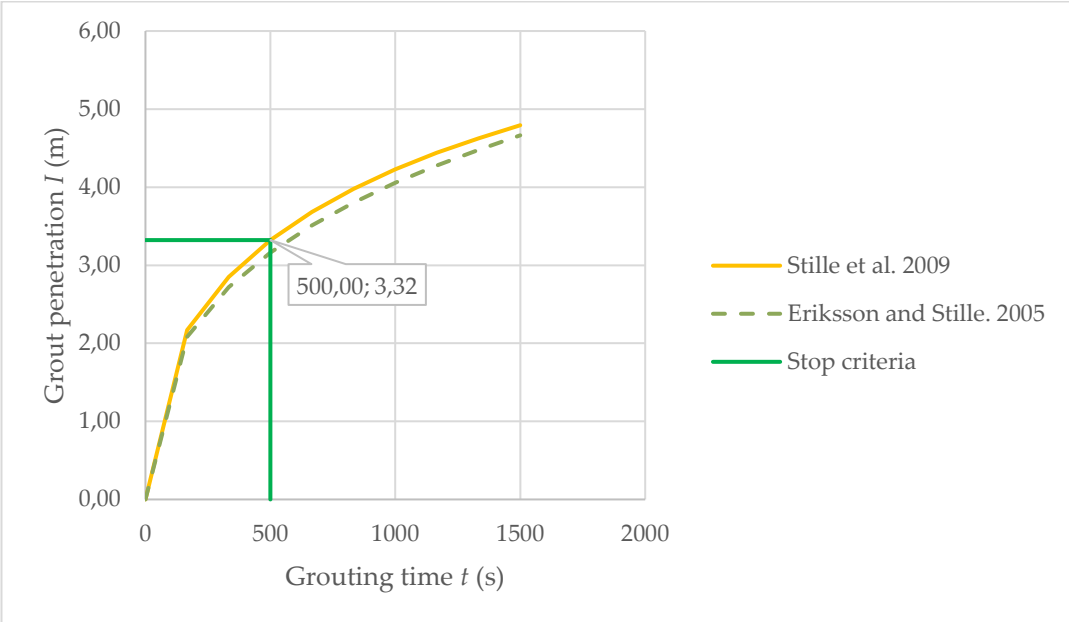


Figure 4-4: Grout spread vs grouting time for section 4-8 m

In addition to the grout spread calculations, the expected flow rate variation of the grout with time can be predicted according to Eq.(3.26) in order to have a reference when conducting the grouting work. For this prediction, the expression of relative grout spread versus relative time (Eq.(3.23)) provided from Eriksson and Stille (2005) is used. The predicted flow rate (Q) against grouting time (t) is presented in Figure 4-5 and Figure 4-6.

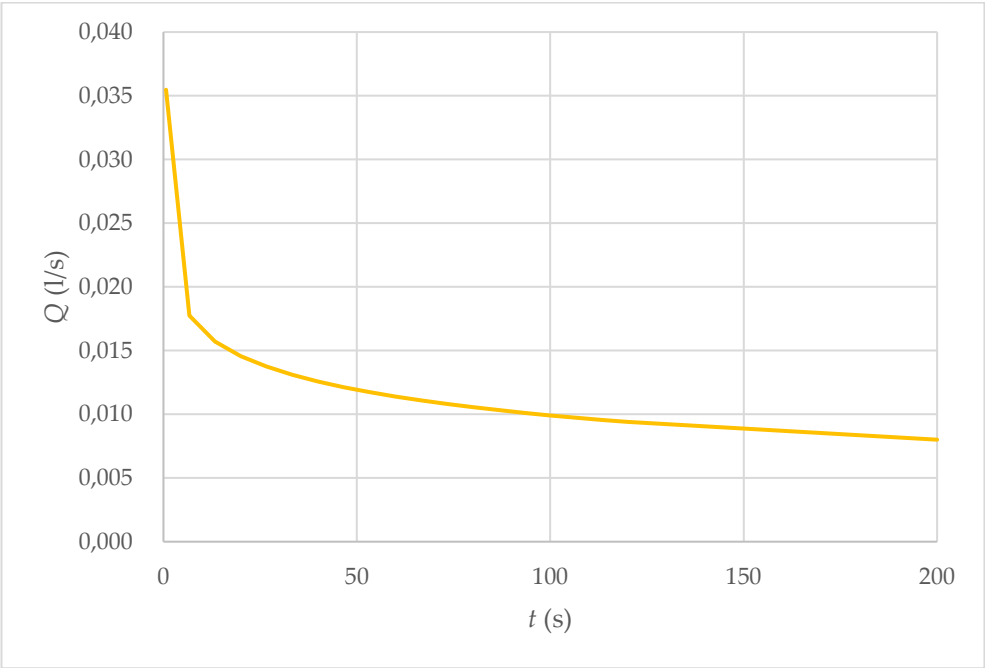


Figure 4-5: Predicted flow vs grouting time for section 0-4 m

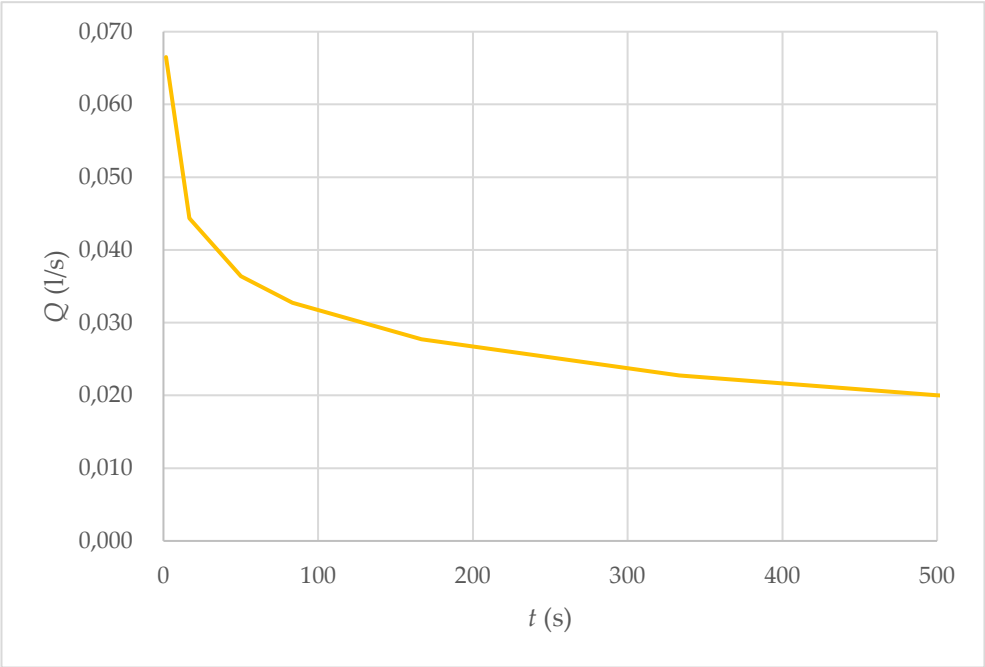


Figure 4-6: Predicted flow vs grouting time for section 4-8 m

Control of grouting pressure with respect to hydraulic jacking

The previously chosen grouting pressures are controlled with respect to hydraulic jacking. The control follows the method in Section 3.1.4. Eq.(3.32) to (3.36) are utilized with a reference to Figure 3-12. The calculation parameters are presented in Table 4-10, in which the depth of the fractures in this example is assumed to be located in the midpoint of each section. The graphs for control of the pressure are plotted in Figure 4-7 and Figure 4-8. As seen in the graphs, neither of the pressure lines intersects with the pressure limits, indicating that the chosen grouting pressures are acceptable to prevent jacking.

Table 4-10: Calculation parameters for grouting pressure check

Depth of fractures (m)	k'_2	Effective grouting pressure ΔP_g (MPa)	Relative pressure p_n	Pore water pressure p_w (MPa)	Acceptable displacement δ_{acc} (µm)
2	1	0.2	1.28	0.02	200
6		0.5	1.07	0.06	

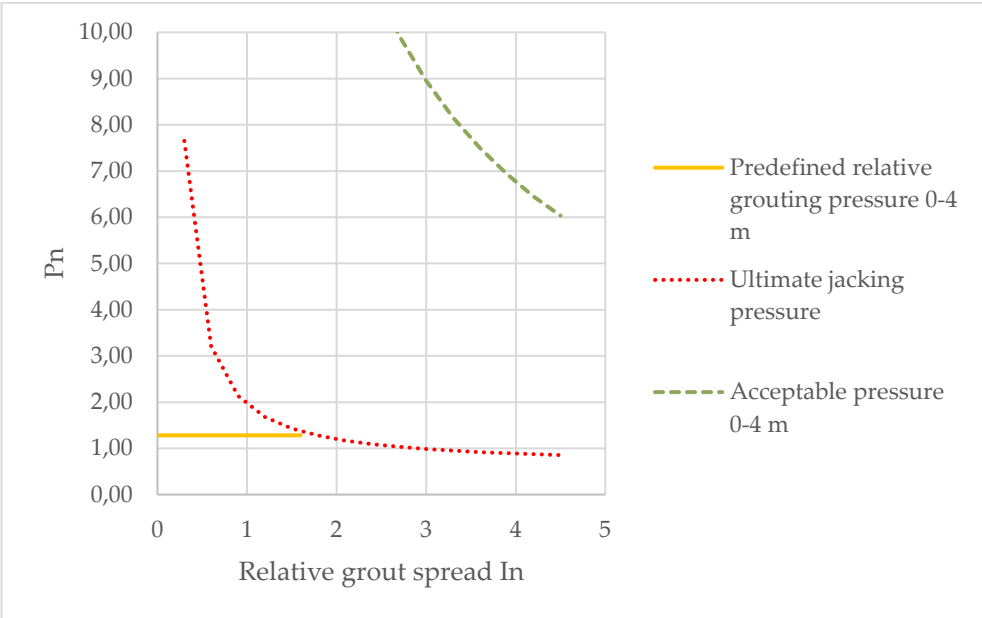


Figure 4-7: Grouting pressure check for section 0-4 m

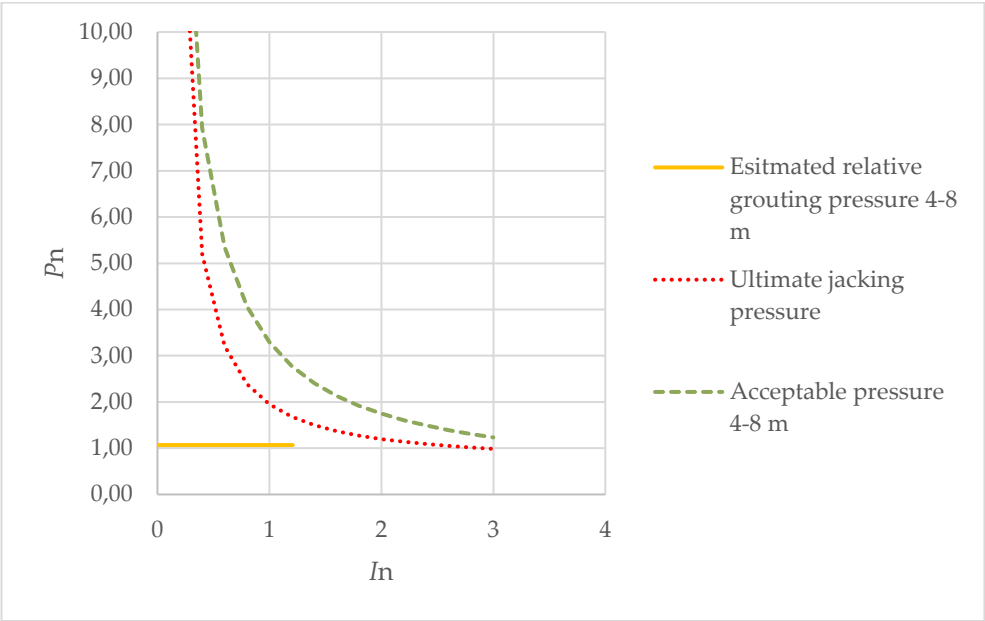


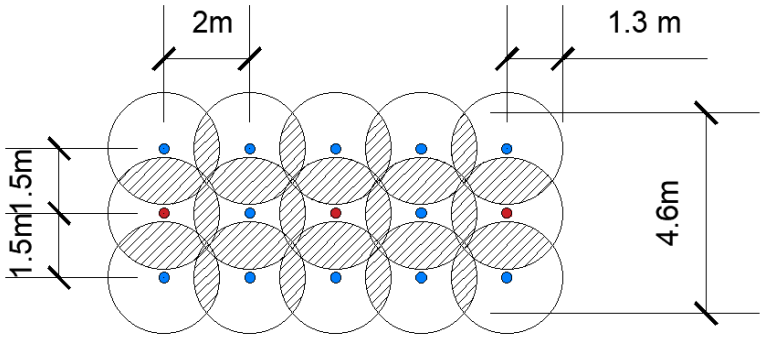
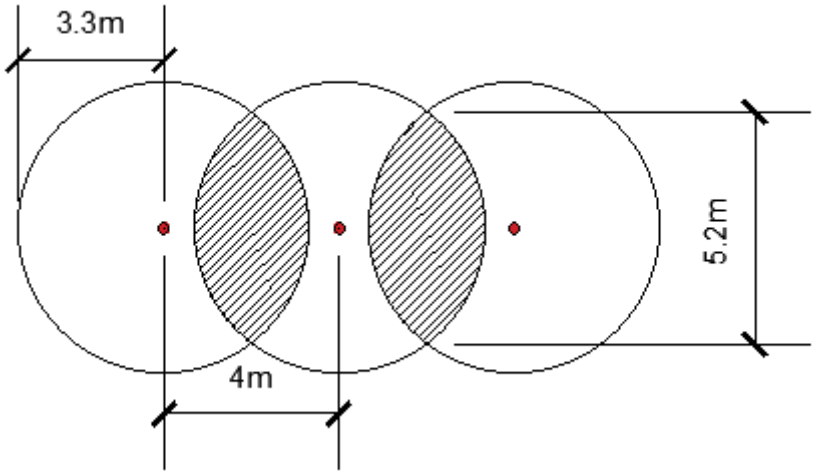
Figure 4-8: Grouting pressure check for section 4-8 m

Layout of the grout holes

Following the principle in which the thickness of the grout curtain is determined by the overlapping of the grout spread, the layout of the grout holes for this example is designed as shown in Table 4-11. It can be observed that the grout spread in section 0-4 m is limited to prevent unfavorable jacking. Consequently, the grout hole layout of this shallow section is similar to a compaction grouting, with closely distributed holes and three rows, the spacing between rows is 1.5 m. Section 4-8 m and deeper sections, however, have less restriction on grouting pressure. Thus, these sections could have a wider spacing (4 m). The difference in spacing makes it possible to plan for primary and secondary holes, where primary holes are drilled the whole depth of the curtain (20 m) with a spacing of 4 m (marked as red in Table 4-11), while the secondary holes are drilled only to a depth of 4 m at the midpoint between the primary holes in the same row of primary holes. In the additional two rows, the spacing between adjacent secondary holes is 2 m to obtain the expected overlapping.

As already mentioned, the drilling/grouting cycle for all holes should not be performed at once. Instead, half of the primary holes (one in every two primary holes with spacing of 8 m) can be first drilled and grouted. It is then followed by the drilling and grouting for the rest of the primary holes. This time lagging is used to reduce the risk of grout overspread and entering the adjacent ungrouted holes.

Table 4-11: Grout hole layout

Grouting Sections	Grout hole layout
0-4 m	
4-8 m and deeper	

Primary holes:
Secondary holes:
Overlapping:

4.2.5 Summary of the preliminary design

Grout curtain design

The grout curtain is designed with regards to its thickness (T), depth (D), expected hydraulic gradient in the curtain (i_g) and expected residual hydraulic conductivity (K_g). The results are summarized in Table 4-12.

Table 4-12: Grout curtain design results

T (m)	D (m)	i_g (m/m)	K_g (Lugeon)
4	20	3.9	0.6

Grouting work design

The performed calculations show that one row of primary grout holes and multiple rows of secondary holes are sufficient to construct a grout curtain with a thickness of 3 m. The designed layout of the grout holes is illustrated in Figure 4-9. The grouting parameters for each grouting section, including the grouting pressures and stop criteria, are shown in Table 4-13. Low grouting pressure should be used in section 0-4 m due the thin overburden and the strict requirement on unfavorable jacking. The grouting parameters for all deeper sections than 0-4 m are identical, with relatively larger pressure and shorter grouting time.

The grouting time and the injected grout volume are the two stop criteria for each grouting section. The grouting time is based on the grout spread in the boundary fracture that the grout is expected to penetrate whereas the injected volume is based on the grout spread in the largest fracture within the respective section. Different sections have different largest fracture apertures. Therefore, the estimated volume varies with depths under the same grouting pressure and grouting time, as shown in Table 4-13.

The grouting parameters shown in Table 4-13 is the primary grouting plan for all the grout holes drilled in the rock mass domain of interest. Different grout holes share the same grouting parameters in same depths. The design for the other sections of the same curtain in different rock mass domain shall be performed separately.

As shown in Table 4-13, the same grout mix is to be used for different depths. This can be a simplification for this example. The grout mix does not have to be constant throughout all the drilling and grouting, different mix can be used to deal with different conditions. For example, in deeper sections where the grouting can be performed with higher grouting pressure, thicker mix can be employed to reduce the risk of overspread whereas in shallow sections the grout penetrability can be maximized under a low grouting pressure by using a thinner mix. Different mix can also be employed between holes when needed.

Several assumptions are made throughout the design process in this example. For instance: on the aperture distribution, on the fracture infilling material and infilling conditions, and so on. They are all potential sources of uncertainties for the design. These assumptions need to be made with caution based on thorough investigations and good judgements. It is also important to be aware of the assumptions involved so that the corresponding alternative designs or the contingency plans can be prepared on some of the assumptions in case the assumptions are observed to be invalid during the grouting process.

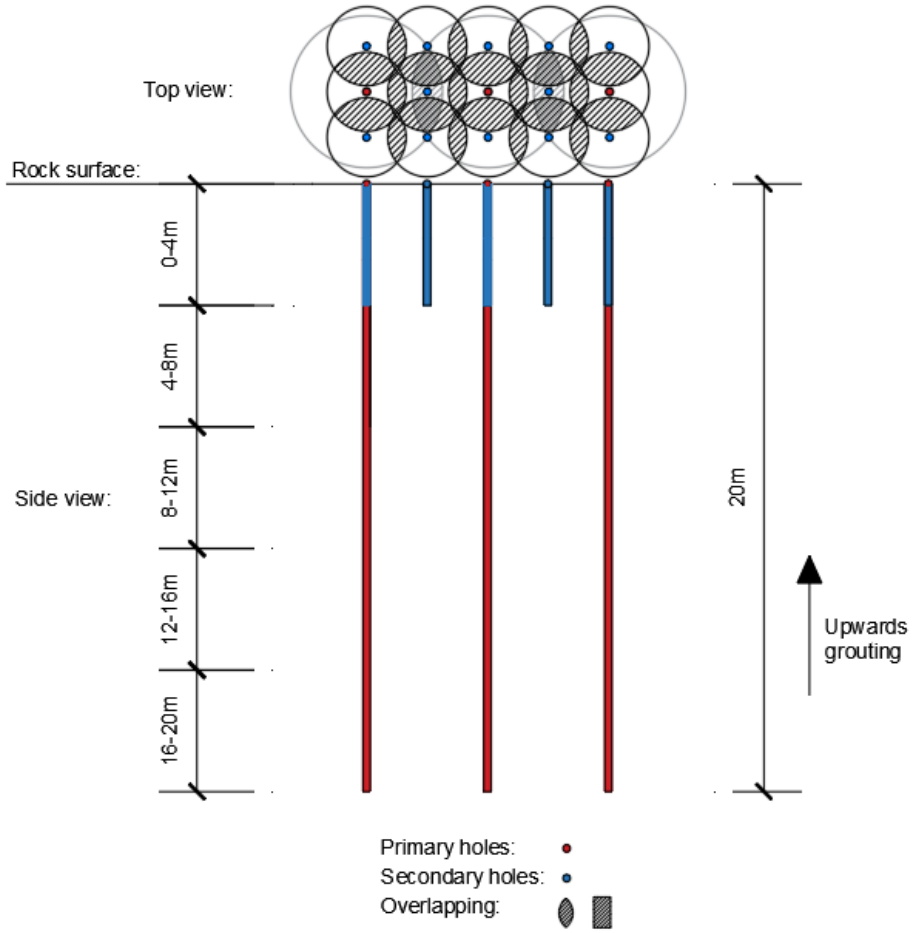


Figure 4-9: Designed layout of grout holes

Table 4-13: Grouting plan for each grouting section

Grout mix	Grouting section (upwards grouting)	Grouting pressure (MPa)	Grouting time (s)	Injected volume per hole per section (liter)
	16-20 m	0.7	500	24
	12-16 m	0.6	500	45
INJ30 w:c=0.8	8-12 m	0.6	500	105
	4-8 m	0.6	500	92
	0-4 m	0.2	200	19

5 Concluding remarks

5.1 COMPARISON AGAINST PREVIOUS DESIGN PRACTICE

The new theory-based design method developed and presented in this report is able to give quantitative estimations for the design of grout curtain geometries under dams and the grouting process for the curtain, instead of a qualitative planning as advocated in empirical methods from e.g. Statens Vattenfallsverk (1968), Houlsby (1990) and Weaver and Bruce (2007).

The theory-based method presented in this report and the empirical methods share the same grouting objective, which is to reduce the hydraulic conductivity of the rock mass. It is essentially the design and planning of the grouting to achieve the objectives that are fundamentally different in the developed analytical method compared to the empirical methods, even though the new method also adopts some empirical practices which are not theoretically described. For the first time, the thickness of the grout curtain is theoretically determined with respect to reducing the rock mass hydraulic conductivity, reducing the uplift pore pressure and avoiding the internal erosion of the infilling materials. The empirical methods and the theory-based method implement different philosophies to formulate the stop criteria. The empirical criteria are controlled solely by the injecting behaviors (e.g. flow rate) whereas the theory-based method provides the active design of the stop criteria and are only subject to change if unexpected behaviors emerge.

The advantages of a theory-based design method include:

- A predicted behavior of the grouting process can be gained during the planning phase. It provides a reference for the engineers and enables them to analyze the layout of the grout holes. The theory-based design also makes it easier to estimate the total cost and time of the grouting work, which could benefit the contract issues.
- Given a preliminary design as a reference, behaviors which are different from the expected ones can be observed more easily and be analyzed in time and changed if needed. The application of the observational method can increase the method's ability to deal with uncertainties.
- With the three-criteria approach to determine the grout curtain thickness, the durability of the grout curtain may increase, especially if infilling materials are present in the fine fractures.
- The stop criteria based on calculations of the grouting time and injection volume can be a more efficient approach compared to the classical empirical stop criteria.
- Computer-aided design is possible to be developed from the theory-based design thanks to large involvement of analytical calculations in the new design method.

5.2 LIMITATIONS AND FUTURE PROSPECTS

5.2.1 Limitations

Although much research has been done in grouting, limitations still exist in the developed design method presented in this report.

Even though some parts of the new design method have been studied in comparison with other dam projects, e.g. the Gotvand dam project (Rafi et al. 2012) and the THX dam project (Rafi 2014). Further applications using this new method are desired with respect to grout curtains under dams. In chapter 4, an example design was performed to illustrate the steps in the preliminary design phase. However, it was performed under fictitious conditions with assumptions and simplifications. Real geological conditions at a dam construction site are complex. The robustness of this design method when dealing with highly complex conditions is yet to be tested. This method is open for further improvements after knowledge has been gained from more research and case studies.

5.2.2 Future prospects

The recommended fields for further studies given by the authors include, but is not limited to: deeper understanding on erosion of infilling materials in rock fractures, degradation and erosion of hardened grout over time, the analytical expression of water flow or grout spread in a combination of channels and discs in the rock mass, and on the measurements of the grout flow to make the flowmeter more reliable. Additional knowledge within these areas will be valuable for the future development of this design method.

Given sufficient analytical theories, a computer design program is an interesting idea for the design of grout curtains. The concept of a computer-aided design could be applied. A program could be developed, taking into account all parameters (drilling time/cost, material cost, number/arrangement/depth of holes, pressure of injection etc.) to estimate and optimize the design of the grout curtain, as well as the total cost regarding the grouting work.

As introduced previously, uncertainties are reduced by the theory-based design method but are still involved in the calculations. Given the fact that uncertainties could not be completely avoided in rock foundation engineering, research efforts with respect to reducing or managing the uncertainties related to grouting are desired. Reliability based design of grouting in combination with the observational method may be an interesting way forward for the future development of rock foundation grouting.

Remedial grouting may be needed when the grout curtain is not functioning properly or when other unfavorable behaviors occur during the operation of the dam. Some existing old dams that are in operation in Sweden are facing problems from grout degradation. In Sweden, as well as internationally in countries having a population of old dams, remedial grouting might be of more concern than new grouting in the coming decades for the hydropower industry, since few new dams are being built in these countries. One of the most significant challenges regarding remedial grouting is the existence of the dam and the reservoir and thus the high

head difference between reservoir and the tailwater. The grout may get eroded during grouting due to the high water velocity in the fractures. Viscous fingering may also occur after grouting when the water pressure exceeds the pressure in the grout. The viscous fingering into the grout caused by water has not been studied in depth. Although remedial grouting has been discussed briefly in recent years, the discussions were mostly empirical. New theories regarding grout erosion and fingering, together with application of the principles of the observational method are potentially able to facilitate an analytical design method for remedial grouting. Further studies are desired on this topic.

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DESIGN OF GROUT CURTAINS

Earlier, the design of grout curtains has mainly been performed based on experience and empirical techniques. In this report, a theory-based design concept for grout curtain under new dams is presented.

In comparison to the traditional empirical method, the theory-based design concept presented in this report is expected to provide the engineers with a better control of the grouting process resulting in improved quality and efficiency of the grouting work.

The design concept also enables an improved estimation of costs and time associated with the construction of the grout curtains.

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