R-09-09

# Underground design Laxemar Layout D2

## Grouting

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This report concerns a study which was conducted for SKB. The conclusions and viewpoints presented in the report are those of the authors. SKB may draw modified conclusions, based on additional literature sources and/or expert opinions.

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# Summary

Restrictions must be observed with regard to permitted inflow of water in different functional areas in connection with the construction of the underground facility of the final repository. To ensure that permitted seepage is not exceeded sealing is to be carried out by grouting.

The purpose of the grouting design work is to show that stated restrictions with regard to seepage in the underground facility can be achieved by grouting. This is to be done by:

- Showing that technique is available which, in anticipated conditions at the relevant site, can satisfy stipulated requirements.
- Estimating the amounts of grout and other resources that are needed.

For ramp and shaft a considerable ingress of water can be expected down to a depth of 400 m, without grouting. Tunnels at deposition level and in the hydraulic rock mass domains HRD_C and HRD_W give in most of the conditions a relatively moderate inflow, that is between 1 and 15 litre/min, 100 m tunnel. In the most water-bearing parts of these two domains high inflow can occur, that is more than 50 litre/min 100 m tunnel. In most parts of the tunnels at deposition level in hydraulic rock mass domain HRD_EW007, the inflow will be more than 50 litre/min 100 m tunnel. The two larger deformation zones, NS059A and NE107A, cause considerable inflow of water where they cross the transport tunnels. Considerable inflow of water in tunnels also occurs at maximal transmissivity in shorter deformation zones.

From the calculations of inflow before grouting, experience of performed grouting and assessment of the sealing effect and hydraulic aperture, the following grouting measures are recommended:

- Various degrees of difficulty in grouting measures are anticipated in the ramp and sink shaft. Both a systematic and selective pre-grouting with cement will be necessary in order to fulfil the requirement on ingress of water. Complementary grouting with silica sol will be needed in certain sections, such as passage of deformation zones.
- Sealing by grouting in the raisebored shafts is not considered sufficient as the only sealing measure in order to fulfil the requirement on inflow of water. This is probably most likely for the shafts in the deposition area with 500 m long boreholes. However, grouting should be made in order to reduce the large inflows for additional measures. Alternatives to grouting as a sealing method must thus be considered. A further alternative for the drilled shafts could be to change excavation technique to the shaft sinking method in which grouting can be carried out as pre-grouting at the face of the shaft.
- Selective pre-grouting with cement complemented with silica sol is considered suitable for most rock caverns in the central area, transport tunnels and main tunnels, but some tunnel sections with systematic pre-grouting and silica can be anticipated.
- In deposition tunnels which are located in hydraulic rock mass domain HRD_C and HRD_W selective grouting with silica sol is most likely.
- Systematic pre-grouting with silica sol as the main grout will be necessary in all deposition tunnels and in main and transport tunnels in hydraulic rock mass domain HRD_EW007.
- Probe holes are also to be done in positions for deposition holes in HRD_EW007. These holes are drilled with the purpose to assess the inflow to a deposition hole before the drilling of the hole. Complementary grouting of the probe hole may be needed.
- Deformation zones will cross the tunnels at deposition level. Preparedness should be available in unfavourable conditions for more time-consuming grouting, several grouting rounds, an increased use of silica sol and special equipment.

The table below presents a summary of amounts of grout for the functional areas. The difference between estimated maximum and minimum amounts is considerable. This reflects the uncertainty about the conditions that will be met in tunnel excavation and grouting.

| Functional areas/ underground openings | Volume of grout, before exvation, Min.–max. ($m^3$) |
|---|---|
| **Accesses and shafts** | |
| Ramp and Shafts (6 pcs) | 465–1,835 |
| **Central area** | |
| Rock caverns | 140–410 |
| **Deposition area** | |
| Deposition, transport and main tunnels | 5,700–23,350 |

It can be concluded that grouting measures are available for different grouting scenarios. With regard to feasibility the grouting measures should also be realistic in relation to current know-how and experience. To sum up, it is not considered realistic to carry out grouting in large parts of the underground facility with what is known as proven and well-known technique as cement grouting. This applies in particular to deposition tunnels for which the requirements on inflow are strict, and for all tunnels in domain HRD_EW007, where conductivity of the rock mass is relatively high and also in the drilled shafts. In ramp, central area, sink shafts, and also main and transport tunnels in domain HRD_C and HRD_W it is judged possible to carry out grouting using proven and well-known technique to meet the requirements on ingress.

SKB has taken part in developing a new grouting technology to cope with the strict requirements on inflow in deposition tunnels. This new technology has been tested to a limited degree in conditions similar to Laxemar and results demonstrate that the requirements on ingress can be fulfilled. However, this grouting method is time consuming and there are several practical issues that remain to be investigated.

It should be noted that systematic grouting will be needed to a large extent in deposition tunnels. Systematic grouting should, if possible, be avoided according to the design premises, UDP /SKB 2008a/. Whether or not the extent of systematic grouting is acceptable must be investigated.

Furthermore, drilled deposition holes are not to be sealed according to UDP. However, if grouting is not made in locations for deposition holes, the loss of deposition holes may be significant, especially in hydraulic rock mass domain HRD_EW007. Thus it is recommended to make a systematic grouting of possible locations for deposition holes in this hydraulic rock mass domain.

# Sammanfattning

Vid byggandet av slutförvarets undermarksanläggning måste restriktioner avseende tillåtet vatteninläckage till olika anläggningsdelar beaktas. För att säkerställa att tillåtet inläckage ej överskrids ska injektering utföras.

Syftet med projekteringen avseende injekteringsarbetena är att visa att angivna restriktioner avseende inläckage för undermarksanläggningen kan uppfyllas genom injektering. Detta ska göras genom att:

- Visa att teknik finns som, vid förväntade förhållanden på den aktuella platsen, kan uppfylla ställda krav.
- Bedöma vilka mängder av injekteringsmedel och andra resurser som behövs.

För ramp och schakt kan det konstateras ett stort vatteninläckage ner till djupet 400 m, utan injektering. Tunnlar på deponeringsnivå som är belägna i de hydrauliska domänerna HRD_C och HRD_W ger vid de flesta förhållandena ett relativt måttligt inflöde, dvs mellan 1 och 15 liter/min, 100 m tunnel. Inflödet utan injektering till tunnlar på deponeringsnivå och som är belägna i hydraulisk domän HRD_EW007 är generellt alltid högt. Vatteninläckaget till transporttunnlarna blir också stort när de korsas av de två större deformationszonerna, NS059A och NE107A. Betydande vatteninläckage till tunnlarna blir även fallet för mindre deformationszoner vid maximal transmissivitet.

Från beräkningar av inflöde före injektering, tidigare injekteringserfarenheter, samt analyser av olika aspekter som påverkar injekteringens svårighetsgrad, kan injekteringen sammanfattas enligt följande:

- I rampen och sänkschakt förväntas olika svårighetsgrad av injekteringsmetodik förekomma. Både en kontinuerlig och selektiv förinjektering med cement blir nödvändigt för att klara inläckagekravet vid olika tunnelsträckor. Vid vissa sektioner, som passage av deformationszoner, kommer injekteringen att behöva kompletteras med silica sol.
- Tätning genom förinjektering i de raiseborrade schakten bedöms inte vara tillräcklig för att klara inläckagekravet. Detta gäller sannolikt främst schakten i deponeringsområdet, vilka endast kan injekteras från markytan med 500 m långa borrhål. Injektering måste dock utföras för att minska de största inläckagen. Alternativa tätningsmetoder måste således tas fram för dessa schakt. En annan åtgärd är att driva schakten genom schaktsänkning, vilket innebär att förinjektering kan utföras i samband med schaktdrivningen.
- Selektiv förinjektering med cement bedöms vara lämpligt för flertalet bergrum i centralområdet, transporttunnlar och stamtunnlar, men längs vissa tunnelsträckor kan kontinuerlig förinjektering och silica sol förväntas.
- Selektiv injektering med silica sol som huvudsakligt injekteringsmedel kommer att utföras i deponeringstunnlar i de hydrauliska domänerna HRD_C och HRD_W.
- Systematisk förinjektering med silica sol som huvudsakligt injekteringsmedel kommer att vara nödvändig i samtliga deponeringstunnlar samt stam- och transporttunnlar i hydraulisk domän HRD_EW007.
- Sonderingshål kommer att borras i lägen för möjliga deponeringshål i hydraulisk domän HRD_EW007. Dessa sonderingshål utförs med syfte att bedöma inflödet till ett deponeringshål innan det borras. Kompletterande injektering utförs i sonderingshålen vid behov.
- Deformationszoner kommer att korsa tunnlarna på deponeringsnivå. Vid ogynnsamma förhållanden skall beredskap finnas för en mer tidskrävande injektering, flera injekteringsomgångar, en ökad användning av silica sol och specialutrustning.

I tabellen nedan sammanfattas uppskattade injekteringsmängder för de olika anläggningsdelarna. Skillnaden mellan beräknade max- och minmängder är stor. Detta speglar osäkerheten om vilka förhållanden som kommer att påträffas vid tunneldrivningen och injekteringen.

| Anläggningsdel/ undermarksanläggning | Injekteringsmängd, före berguttag Min.–max. ($m^3$) |
|---|---|
| **Nedfarten och schakt** | |
| Ramp och schakt (6 st) | 465–1,835 |
| **Centralområde** | |
| Berghallar | 140–410 |
| **Deponeringsområde** | |
| Deponerings- transport- och stamtunnlar | 5,700–23,350 |

Bedömningen är att det finns injekteringsmetoder för de olika injekteringsscenarier som kan förväntas. Med hänsyn till genomförbarheten bör dock också injekteringsmetoderna vara realistiska med hänsyn till rådande kunskapsläge och erfarenheter. Sammanfattningsvis bedöms det inte vara realistiskt att utföra injektering i stora delar av anläggningen med så kallad beprövad och välkänd teknik såsom cementinjektering. Detta gäller speciellt deponeringstunnlar, där inläckagekraven är stränga, och alla tunnlar i hydraulisk domän HRD_EW007, där bergmassans konduktivitet är relativt hög samt i de borrade schakten. I ramp, centralområde, sänkschakt samt stam- och transporttunnlar i de hydrauliska domänerna HRD_C och HRD_W bedöms det vara möjligt att utföra injekteringen med beprövad och välkänd teknik så att inläckagekraven klaras.

SKB har varit med om att ta fram en ny injekteringsteknik för att klara de stränga inläckagekraven i deponeringstunnlar. Denna teknik har provats i begränsad omfattning vid liknade förhållanden som Laxemar och resultaten visar att inläckagekraven kan klaras. Emellertid är injekteringsmetodiken tidskrävande och det finns flera pratiska aspekter kvar att utreda.

Det kan konstateras att systematisk injektering kommer att krävas i ett stort antal deponeringstunnlar. Enligt projekteringsförutsättningarna, UDP /SKB 2008a/ ska dock systematisk injektering om möjligt undvikas, varför det måste utredas om omfattning av den systematiska injekteringen kan accepteras eller inte.

Vidare anger UDP att borrade deponeringshål inte ska tätas. Om injektering inte utförs i lägen för deponeringshål, kan bortfallet av deponeringshål bli stort speciellt i den hydrauliska domänen HRD_EW007. En systematisk injektering av lägen för deponeringshål rekommenderas därför i denna hydrauliska domän.

# Contents

# 1 Introduction

## 1.1 Background

Restrictions must be observed with regard to permitted inflow of water in different functional areas in connection with the construction of the underground facility of the final repository. To ensure that permitted seepage is not exceeded sealing is to be carried out by grouting. Requirements related to grouting works are stated in the Underground Design Premises/D2 (UDP) /SKB 2008a/, e.g. requirements on maximum permitted inflow to different underground openings and also requirements on composition of the grout.

The preliminary grouting design here presented has been carried out based on design premises in UDP /SKB 2008a/ and engineering descriptions of the rock mass presented in Site Engineering Report Design Step D2, Guidelines for Underground Design, Laxemar Site (SER) /SKB 2008b/.

## 1.2 Purpose

The purpose of the grouting design work is according to UDP /SKB 2008a/ to show that stated restrictions with regard to seepage in the underground facility can be achieved by grouting. This is to be done according to UDP by:

- Showing that technique is available which, in anticipated conditions at the relevant site, can satisfy stipulated requirements.
- Estimating the amounts of grout and other resources that are needed.

## 1.3 Implementation

An overall description of the design methodology is given in UDP /SKB 2008a/. For the grouting design work the following design activities are to be carried out according to UDP /SKB 2008a/:

- Assessment of "ground behaviour".
- Configuration of grouting methodology.
- Assessment of "system behaviour".
- Assessment of amounts and other resources.
- Assessment of feasibility and uncertainties.

Chapter 2 presents, by way of introduction, the premises for the grouting design work concerning geology and hydrogeology, the underground facility and grouting measures.

In the assessment of "ground behaviour" the probable inflow of water to the different functional areas before grouting is presented (see Chapter 3).

As a basis for choice of grouting measures, analyses of aspects regarding the difficulty of grouting have been made. These analyses are presented in Chapter 4.

A large number of grouting works have also been studied to obtain a basis for the configuration of grouting measures. These are presented in Appendix A.

The configuration of grouting measures refers to a specification of how the grouting is to be performed on the basis of "grouting types" (see Chapter 5).

In the assessment of "system behaviour" the probable inflow of water to the different parts of the facility after grouting is presented (see Chapter 6).

The assessment of amounts and other resources concerning grout, total length of boreholes and also the need of equipment for special grouting measures (see Chapter 7).

In the assessment of feasibility and uncertainties a feedback has been made to the purpose of the design (see Chapter 8). The criteria for evaluation of the feasibility of the proposed grouting measures, based on recommendations in /Emmelin et al. 2007/, are the following:

- The grouting measures are to be realistic in relation to present know-how and experience.
- The grouting measures are to be robust in relation to anticipated variations in characteristics of the rock mass.
- A process for handling prevailing uncertainties should be presented.
- Assessments of amounts, time needed and cost and also that these may not be unreasonably large.

The assessment of feasibility and uncertainties also constitute a basis for the technical risk assessment, which is made as a separate activity in design step D2 according to /SKB 2008a/.

For the design in step D2 the application of the observational method implies, according to UDP /SKB 2008a/, that the following is to be carried out:

- Acceptable behaviour for the construction is to be stated.
- Possible behaviour is to be assessed.
- Extent and which parameters that should be measured and checked in the construction stage are to be stated.

What is acceptable behaviour with regard to grouting is stated by SKB in the form of requirements on maximum permitted inflow of water to various underground openings. Accordingly, maximum permitted inflow to the various underground openings is one of the design premises, as presented in Chapter 2.

Possible behaviour is judged as the amount of water inflow to various underground openings before grouting, i.e. “ground behaviour”, and after grouting, i.e. “system behaviour”. These assessments are presented in Chapter 3 and Chapter 6 respectively.

The extent of parameters that should be measured and checked in the construction stage is presented in Chapter 5.5.

Alternatives to grouting in order to mitigate environmental effects due to ground water table drawdown have not been included in the study.

## 1.4 Nonconformities to the design premises

According to UDP /SKB 2008a/ the rock mass is to be divided into “ground types”, giving a general description of the rock mass and also the values of a number of parameters with regard to rock mechanics and hydrogeology. It has been decided that “ground types” are not to be applied in the assessment of water inflow and the configuration of grouting methods, which was the instruction in UDP /SKB 2008a/. The reason for this was that the hydrogeological description of “ground types” was not deemed suitable for use together with the other hydrogeological description in SER /SKB 2008b/. Nonconformities to the UDP /SKB 2008a/ with regard to the above are described in the respective Chapter of this report.

Geometries and relative location of the functional areas, especially the central area, are taken from a preliminary version the UDP /SKB 2008a/ without consideration to later adjustments. The reason why such adjustments have not been observed is because details in the final layout lack significance for result and conclusions.

## 1.5 Terminology

Some of the terms and concepts used in this report are explained below. The list comprises terms and concepts that are specific for SKB, for grouting, the rock construction process, or for other reasons need to be explained or defined in order to describe the discussed concepts in a stringent way. The terms used in this report are noted in Table 1-1.

**Table 1-1. Terminology.**

| Term | Explanation | Reference |
|---|---|---|
| SER | "Site Engineering Report, Guidelines for underground design step D2" /SKB 2008b/. A report that presents an engineering description of the rock mass for design step D2. | |
| UDP | "Underground design premises/D2" /SKB 2008a/. A steering document for rock engineering design work in step D2. | |
| Functional area | Part of underground facility of the final repository. Functional areas are repository access, central area and deposition area. | /SKB 2008a/ |
| Repository access | Functional area including access ramp and shafts to central area. | /SKB 2008a/ |
| Central area | Functional area including rock caverns and tunnels for personnel, operation and maintenance. | /SKB 2008a/ |
| Deposition area | Functional area for canister deposition including deposition tunnels, main tunnels and deposition holes. | /SKB 2008a/ |
| Deformation zone | Employed as a general notation of an essentially 2D structure characterised by ductile or brittle deformation, or a combination of the two. Those deformation zones which are possible to correlate between the surface (lineament with a length > 1,000 m) and an interpreted borehole intercept, or alternatively between one or more borehole intercepts, or exhibit an interpreted true thickness >10 m are modelled deterministically, and are thus explicitly accounted for in the 3D RVS model. Deformation zones at Laxemar that are correlated to surface are denoted ZSM followed by two to eight letters or digits. An indication of the orientation of the zone is included in the identification code. Other deterministic deformation zones are denoted KLXxx_DZxx (the digits corresponding respectively to the borehole ID and the DZ ID from Extended Single hole interpretation). | /SKB 2008b/ |
| Fracture domain | A fracture domain is a rock volume outside deformation zones in which rock units show similar fracture intensity characteristics. Fracture domains at Laxemar are denoted FSMxx. | /SKB 2008b/ |
| Hydraulic domain | As for the bedrock the groundwater system was divided into 3 hydraulic domains which are 1) HRD (hydraulic rock mass domain) which represents the fracture domains between the deformation zones, 2) HCD (hydraulic conductor domain) which represents deformation zones and 3) HSD (hydraulic soil domain) which represents the overburden. The division in hydraulic domains represents the basis for hydrogeological modelling. | /SKB 2008b/ |
| Rock domain | A rock domain refers to a rock volume in which rock units that show specifically similar composition, grain size, degree of bedrock homogeneity, and degree and style of ductile deformation have been combined and distinguished from each other. The term rock domain is used in the 3D geometric modelling work and different rock domains at Laxemar are referred to as RSMxxx. | /SKB 2008b/ |
| Rock unit | A rock unit is defined in the single-hole geological interpenetration on the basis of the composition, grain size and inferred relative age of the dominant rock type. Other geological features including the degree of bedrock homogeneity, and the degree and style of ductile deformation also help to define and distinguish some rock units. N.B. Defined rock units differ between boreholes. | /SKB 2008b/ |
| Grouting type | Description of principles with regard to extent and execution of pre-grouting. | /SKB 2008a/ |
| Systematic pre-grouting | Several successive planned full grouting fans. | |
| Selective pre-grouting | Grouting of a number of boreholes or a full grouting fan, that is made after assessment on site of investigation holes or probe holes. | |
| Underground opening | The underground openings required to accommodate the sub-surface facilities.<br>– The actual location and geometry of the underground openings.<br>– The rock surrounding the openings affected by the rock excavation, support and grouting works.<br>– Civil works and stray materials remaining when the underground openings are backfilled. | /SKB 2008a/ |

# 2 Premises

## 2.1 Geology and hydrogeology

According to SER /SKB 2008b/ the Laxemar area consists of a mixture of magmatic rocks such as granites, syenitoids, dioritoids and gabbroids. Furthermore, rock matrixes occur as granites and pegmatites. According to SER /SKB 2008b/ the local model area is divided into several rock domains, of which the three largest are designated RSMA01, RSMD01 and RSMM01. The domains RSMA01, RSMD01 and RSMM01 consist mainly of Ävrö granite, Quartz monzodiorite and Ävrö quartz monzodiorite respectively with a high content of diorite/gabbro. In addition, a division has been made in fracture domains (FSM) and hydraulic rock mass domains (HRD) according to SER /SKB 2008b/.

Figure 2-1 shows fracture domains and deformation zones.

Moreover, the Laxemar area has been divided in hydraulic rock mass domains. These hydraulic domains are linked to the fracture domains but one of the hydraulic domains contains three fracture domains, so the total number of hydraulic rock mass domains is four. According to SER /SKB 2008b/ these hydraulic domains are:

- HRD_N coinciding with fracture domain FSM_N
- HRD_EW_007 coinciding with fracture domain FSM_EW_007
- HRD_C being the combination of fracture domains FSM_C, FSM_NE and FSM_S
- HRD_W coinciding with fracture domain FSM_W

The four hydraulic rock mass domains are situated according to Figure 2-2.

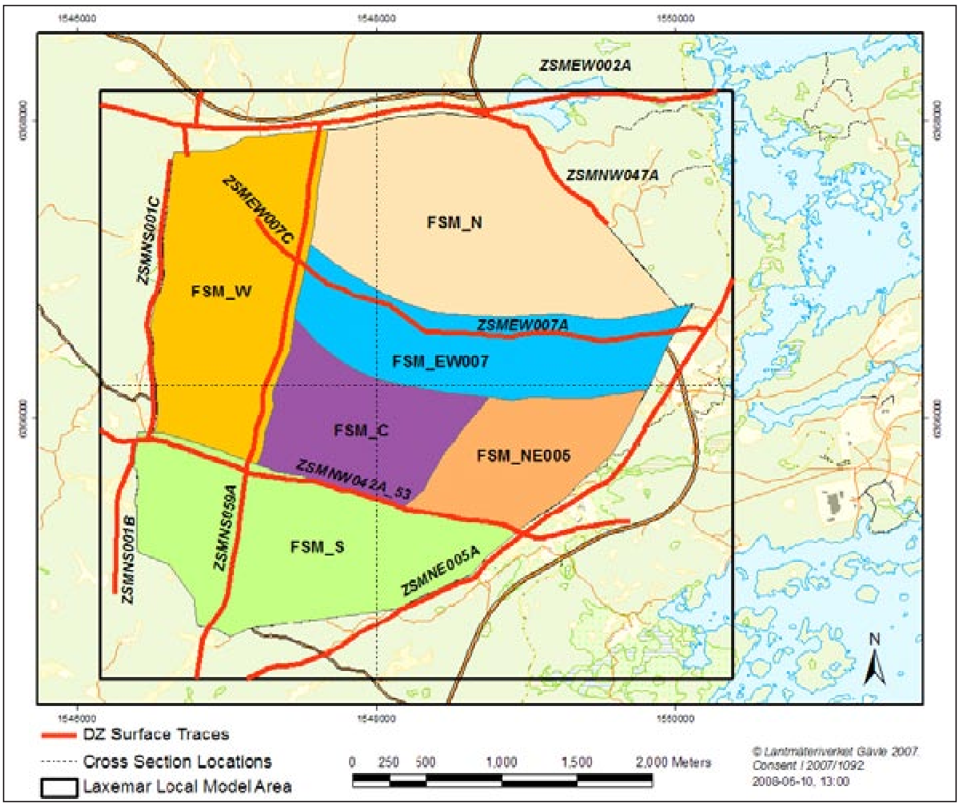


*Figure 2-1. Surface projection of SDM-Site Laxemar fracture domains (FSM_x) and bounding deformation zones (ZSM_x) in Laxemar. The black box represents the limits of the Laxemar local model, while the colored polygons represent the surface limits of the fracture domains. /SKB 2008b/.*

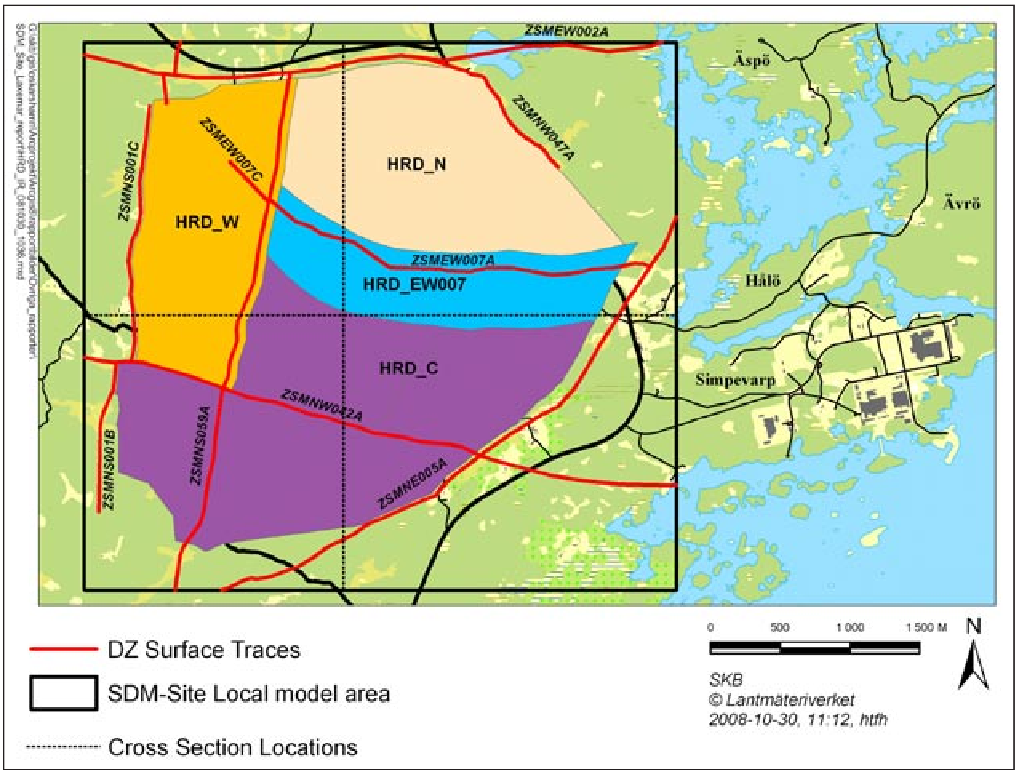


***Figure 2-2.*** *Illustration of the Hydraulic Rock Domains /SKB 2008b/.*

Deformation zones are of various lengths and thickness and also mechanical and hydraulic characteristics. Vertical and steeply dipping deformation zones dominate the picture and comprise 48 zones whereas a further 12 zones are gently dipping. Figure 2-3 presents the deformation zones at level –500 m according to SER /SKB 2008b/.

According to SER /SKB 2008b/ the deformation zones have been divided into five main groups depending on orientation:

- Northeast-southwest
- North-south
- West-east to northwest-southeast sub vertical to moderate dip to south
- West-east to northwest-southeast moderate dip to north
- Gentle dipping

Deformation zones with trace length at the ground surface longer than 3 km (larger zones) require a respect distance between deposition tunnel and zone, due to the risk of seismicity caused by post-glacial rebound /SKB 2008b/. Deformation zones with shorter trace length than 3 km (shorter zones) have no respect distance. These shorter zones can be crossed by deposition tunnels, see layout /Leander et al. 2009/, but deposition holes may not be placed in these shorter zones.

Hydraulic characteristics of the various hydraulic rock mass domains and deformation zones are presented in Chapter 3.3.1.

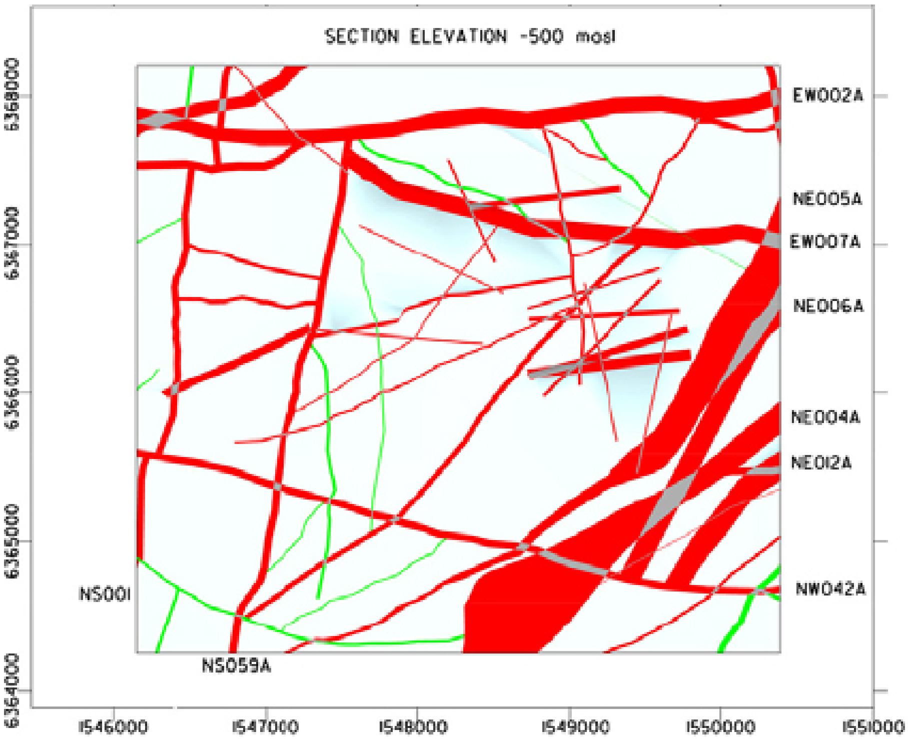


***Figure 2-3.*** *All deformation zones in a horizontal section at depth 500 m. The model is viewed to the north /SKB 2008b/.*

## 2.2 The final repository facility

The accesses from the operational area to the central area of the underground facility consist of a ramp and four vertical shafts, see Figure 2-4.

The central area consists of a number of tunnels and shafts positioned in a complex geometry in relation to one another, see Figure 2-4. The central area is dominated by nine large rock caverns. The rock caverns have a span between 13 to 16 m and a length between 56 to 65 m (see /SKB 2008a/).

The deposition area (at elevation –500 m) consists of main tunnels and deposition tunnels with their deposition holes. At the same level transport tunnels and exhaust shafts (denominated shaft SA01 and SA02) to the surface are located. The layout at the deposition level is shown in Figure 2-5.

The layout of the underground facility is described in more detail in the layout report for Laxemar /Leander et al. 2009/.

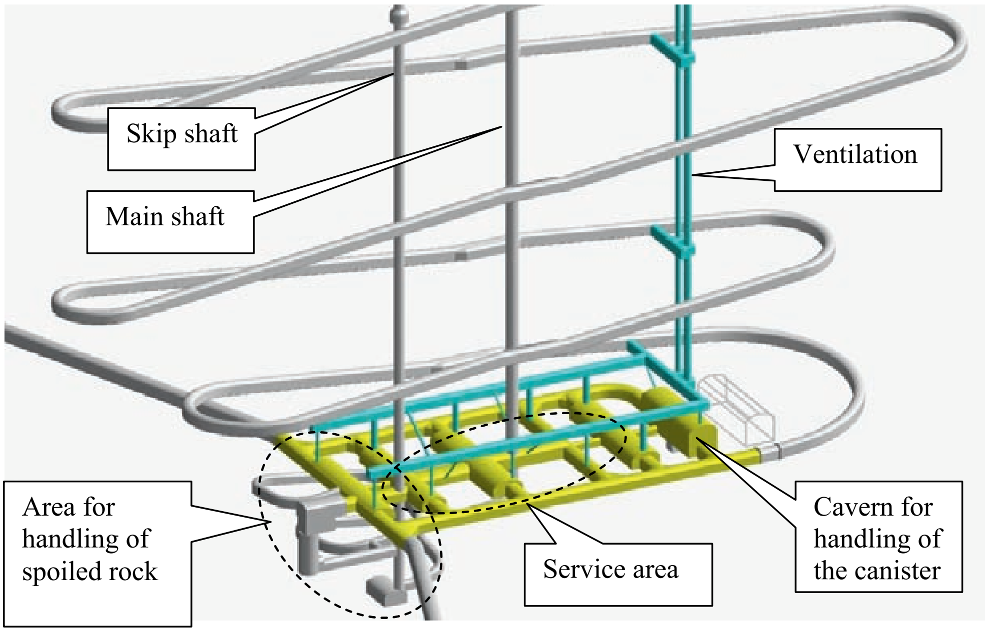


*Figure 2-4. Overall view of the central area and accesses (ramp and shafts), figure from /SKB 2008b/. (Please note that this figure is not exactly up to date with the present layout of the central area, but the figure is considered to be close enough to explain the general features of the layout.)*

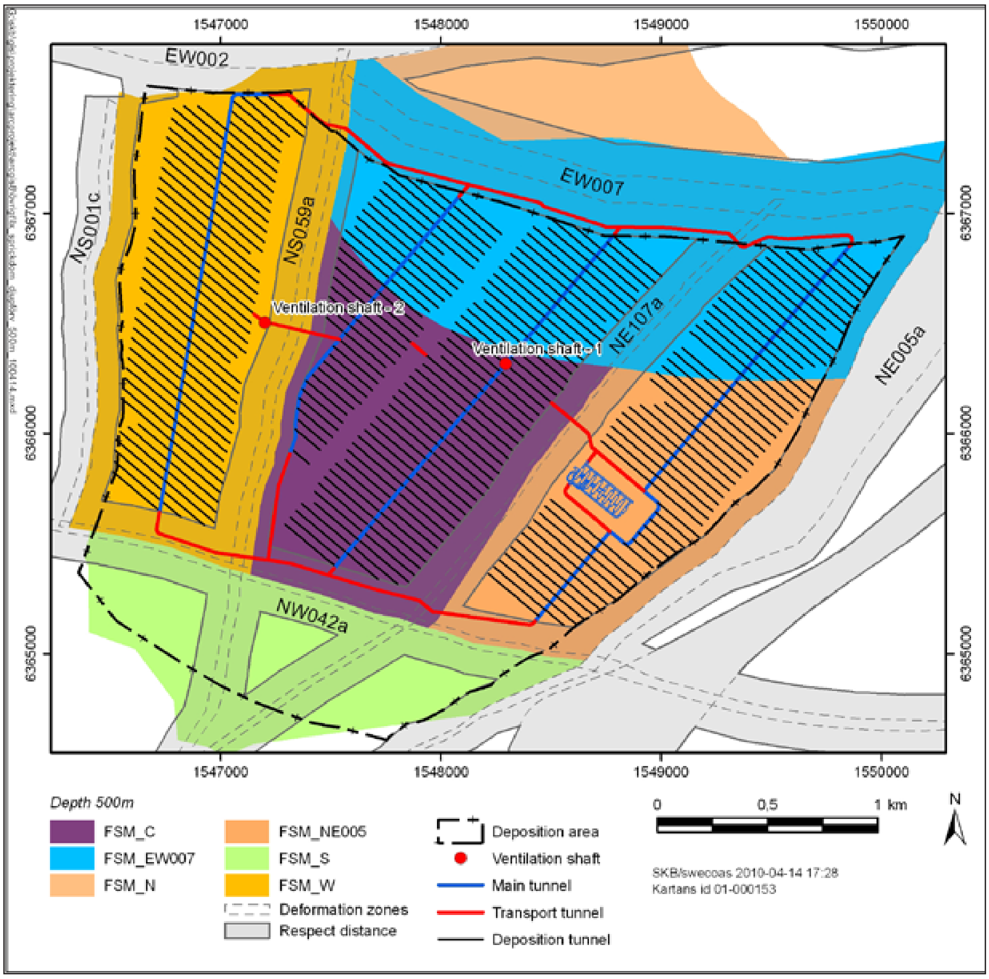


*Figure 2-5. Layout at deposition level including deformation zones, according to /Leander et al. 2009/.*

## 2.3 Requirements on grouting

This section summarises the requirements and conditions used in assessing water inflow, configuration of grouting measures and also assessment of amounts.

- Premises according to UDP /SKB 2008a/ were followed. According to UDP /SKB 2008a/ the following conditions are to be observed in configuring the grouting methodology.
  - SKB will present properties and recipes of currently available grouts and these grouts shall if possible be used. The need of other properties of the grout than those given by SKB shall however clearly be adressed. Recipes of grouts are presented in Appendix C.
  - Existing techniques for the grouting measures are to be used.
  - If otherwise equal methods are discussed, the method giving the lowest material use should be favoured provided that the objectives are fulfilled.
  - Systematic pre-grouting should, if possible, be avoided in deposition tunnels.
  - Boreholes may not be positioned so that they risk interfering with the location of potential deposition hole. However, this requirement does not apply for grouting in deformation zones since no deposition holes will be permitted in such locations.
  - According to UDP /SKB 2008a/ the grouting measures are to be based on the estimated inflow of water before grouting ("ground behaviour") and "grouting types" (GrT), which are stated in SER /SKB 2008b/. The following grouting types (GrT) are defined in SER as follows:

    **Grouting type 1 (GrT1):** "Discrete fracture grouting"

    **Grouting type 2 (GrT2):** "Systematic tunnel grouting"

    **Grouting type 3 (GrT3):** "Control of large inflow and high-pressure"

    According to UDP /SKB 2008a/ a number of parameters are to be described for the respective grouting type. These are fan geometry, grout and also the principles for execution including pressure and controls. For GrT3, special execution and special equipment are also to be described if this is necessary.
- Requirements on grouting are stated in UDP /SKB 2008a/.
  - Acceptable inflow of water to the various underground openings in the underground facility:
    - Deposition holes: point leakage 0.1 l/min.
    - Deposition tunnels: 1.7 l/min, 100 m; point leakage 1 l/min.
    - Shaft and ramp: 10 l/min, 100 m.
    - Other underground openings: 10 l/min, 100 m.

    The requirements concerning maximal seepage per 100 m for different underground openings have been interpreted to mean that the requirements are to be fulfilled for the total length of the opening (for example main tunnels). Based on rough estimates and experience from other grouting work it is considered improbable that the requirements can be fulfilled in a random stretch of 100 m. For deposition tunnels the requirement has been interpreted as applying for each individual deposition tunnel.
  - The grout may not contain substances that could impair the barrier functions and pH is to be less than 11. This requirement has been dealt with in the design by suggesting only grouts that are provided by SKB. The compositions of these grouts have been tested within the framework of SKB's present work of development.
  - The technical life time of deposition tunnels and deposition holes is 5 years. Corresponding time for other rock constructions is 100 years.
  - Deposition holes are not to be sealed. This requirement has been observed in that deposition holes with point leakage >0.1 l/min are rejected. Inflow of water to deposition holes shall according to UDP /SKB 2008a/ be limited by choosing location of the hole in the rock.
- Hydrogeological characteristics according to SER /SKB 2008b/ are to be used.
- The basis for analyses and discussion is the current knowledge and competence concerning design and execution that is described in /Emmelin et al. 2007/.

# 3 Ground behaviour – assessment of water inflow before grouting

## 3.1 Introduction

According to UDP /SKB 2008a/ the inflow of water is to be calculated for different functional areas. The assessment of inflow is to be based both on the most probable conditions and on the most unfavourable conditions.

A deviation from UDP /SKB 2008a/ is that no division into "ground types" has been made. Assessments of water inflow have instead been based on presentations of hydrogeological characteristics in SER /SKB 2008b/ for hydraulic rock mass domains at different depths and for deformation zones.

The assessment of water inflow has been made using analytical calculation methods. More detailed assessments of the water inflow are made within the framework of the site modelling.

## 3.2 Calculation methodology

According to /Bergman and Nord 1982/ the calculation of water inflow into a tunnel can be made using Equation 3-1 and 3-2. Equation 3-1 is applicable to the hydraulic rock mass domains (HRD) and Equation 3-2 to the deformation zones (HCD). The equations are applicable to both non-grouted and grouted circular tunnels, but can also be used for rough calculations of other geometries.

$$Q_t = \frac{2 \cdot \pi \cdot K \cdot H \cdot L}{\ln\left(\frac{2 \cdot H}{r_t}\right) + \left(\frac{K}{K_g} - 1\right) \cdot \ln\left(1 + \frac{t}{r_t}\right) + \xi} \qquad 3\text{-}1$$

$$Q_t = \frac{2 \cdot \pi \cdot T \cdot H}{\ln\left(\frac{2 \cdot H}{r_t}\right) + \left(\frac{K}{K_g} - 1\right) \cdot \ln\left(1 + \frac{t}{r_t}\right) + \xi} \qquad 3\text{-}2$$

in which

H = tunnel depth, below groundwater table (m)

K = hydraulic conductivity of the rock mass (m/s)

$K_g$ = hydraulic conductivity of the grouted zone (m/s)

L = tunnel length (m)

T = transmissivity for deformation zone ($m^2$/s)

t = thickness of grouted zone (m)

$Q_t$ = inflow in steady state conditions ($m^3$/s)

$r_t$ = tunnel radius (m)

$\xi$ = skin factor (dimensionless)

$K_g$ is set to K for a non-grouted tunnel.

The significance of the different parameters in Equation 3-1 is presented in Figure 3-1.

Since the requirements in UDP /SKB 2008a/ are expressed per unit length for the different underground openings, the inflow in the hydraulic rock mass domains are calculated per 100 metre tunnel, i.e. the tunnel length (L) is set constant at 100 m in Equation 3-1.

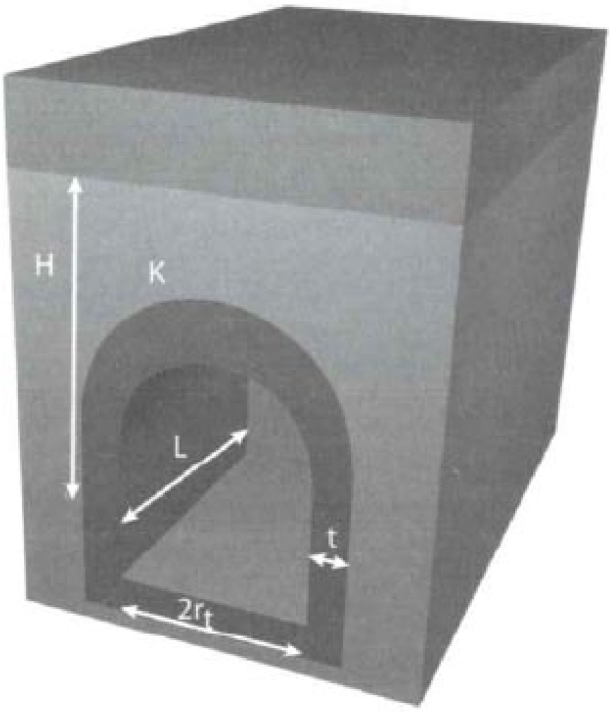


*Figure 3-1. Illustration of the parameters in Equation 3-1. K is the hydraulic conductivity of the rock mass and $K_g$ is the hydraulic conductivity of the grouted zone with thickness t (from /Eriksson and Stille 2005/).*

The inflow to a shaft has been assessed with the aid of Equation 3-3. The equation was given as a basis in design step D1 /SKB 2004/.

$$Q_s = \frac{2 \cdot \pi \cdot T_m \cdot \Delta s}{\ln\left[\frac{R_0}{r}\right] + \left[\frac{K}{K_g} - 1\right]\ln\left[1 + \frac{t}{r_s}\right] + \zeta} \qquad \text{3-3}$$

For Equation 3-3 the following boundary conditions also apply:

for $r \rightarrow R_0$ then $\Delta s \rightarrow 0$

for $r \rightarrow r_s$ then $\Delta s \rightarrow H$

in which

K = representative hydraulic conductivity of the rock mass (m/s)

$K_g$ = hydraulic conductivity of the grouted zone (m/s)

t = thickness of grouted zone (m)

$Q_s$ = inflow in steady state conditions ($m^3/s$)

r = radial distance (m)

$r_s$ = shaft radius (m)

$R_0$ = distance to fringe condition (m)

$T_m$ = representative transmissivity of the rock mass ($m^2/s$)

$\Delta s$ = drawdown (m)

H = shaft depth (groundwater assumed at surface level) (m)

$\zeta$ = skin factor (dimensionless)

On calculating inflow to the shaft $r = r_s$, which according to Equation 3-3 implies that Δs is set to H. The drawdown, Δs, is based on the full shaft depth and takes no consideration to stop in the excavation or drilling.

Equation 3-3 can thus be written as Equation 3-4.

$$Q_s = \frac{2 \cdot \pi \cdot T_m \cdot H}{\ln\left[\frac{R_0}{r_s}\right] + \left[\frac{K}{K_g} - 1\right] \ln\left[1 + \frac{t}{r_s}\right] + \zeta} \qquad \text{3-4}$$

For a non-grouted shaft $K_g$ is set to K in Equation 3-4.

## 3.3 Input data and assumptions

The following section presents the input data and the assumptions that have been used in calculating the inflow of water. Input data concerning hydraulic characteristics, K or T, depth below ground level (water pressure), H, and also radius, $r_t$ or $r_s$, for different functional areas, the underground openings and parts of the rock mass are also presented in Appendix B, Tables B1–B4.

### 3.3.1 Hydraulic characteristics

#### *Hydraulic rock mass domains (HRD)*

Values of the hydraulic conductivity, K, for the rock mass between the deformation zones in different hydraulic rock mass domains are from SER /SKB 2008b/. In SER, the "ΣT/L (m/s)" values are presented for every hydraulic rock mass domains and depth intervals.

For depth interval 400–650 m and every hydraulic rock mass domains the cumulative distribution for the sum of the transmissivity along 20 m and 100 m of the tunnel (correlated) has been determined /SKB 2008b, Stigsson 2009/, see Figures in Appendix D. In Figure 3-2 a summary is made of the cumulative distributions for 100 m tunnels and respective hydraulic rock mass domain, based on the Figures in Appendix D.

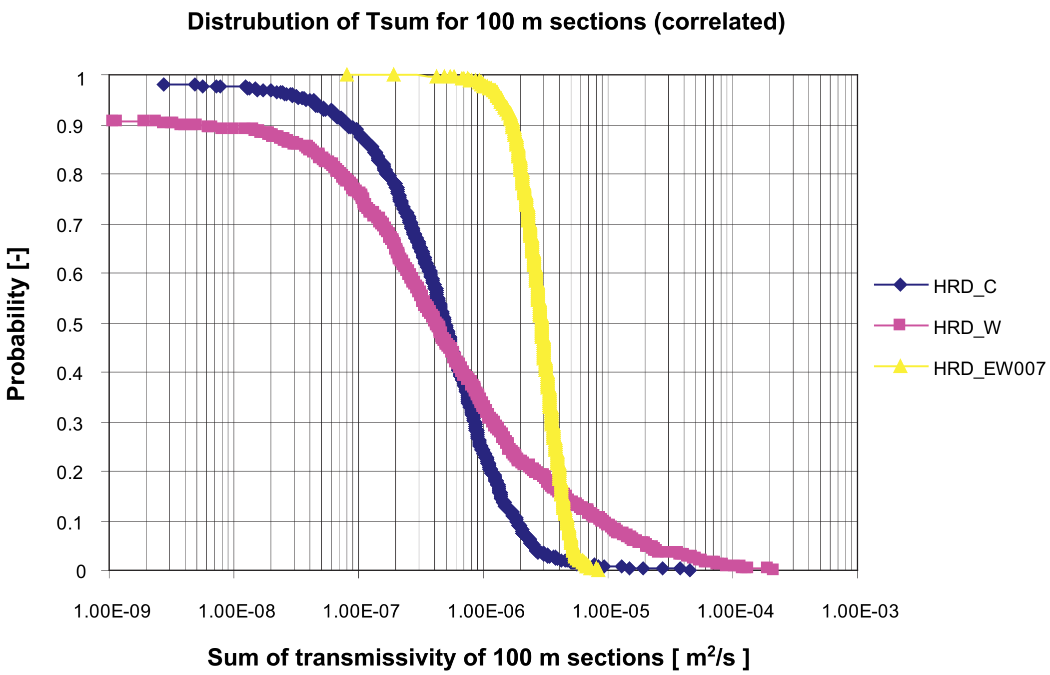


***Figure 3-2.** The cumulative distribution for the sum of transmissivity along 100 m tunnel (correlated) for hydraulic rock mass domains at level 400–650 m, based on /SKB 2008b, Stigsson 2009/.*

For the inflow calculation the values of "ΣT/L (m/s)" have been used for the depth intervals 0–150 m and 150–450 m. For depth 450–600 m the cumulative distribution for the sum of the transmissivity (correlated) has been used.

In Table 3-1 a summary is made of the hydraulic conductivity for different hydraulic rock mass and depth intervals.

Transmissivity values (T) for calculating inflow to the shafts have been calculated as the K value multiplied by the length of the shaft within the depth interval of each hydraulic rock mass domain.

### *Deformation zones*

According to /SKB 2008b/ the deterministically modelled deformation zones are characterised by the following; even though considerable variations occur:

- The transmissivity diminishes with the depth but the span is considerably and constant with the depth.
- The transmissivity increases with increased zone length but differences between individual zones is considerable.
- Deformation zones located E-W are possibly more water-bearing.

In /SKB 2008b/ the thickness and hydraulic conductivity (K) for deterministic deformation zones, located in the deposition area, are present. The transmissivity values (T) has been calculated, with help of thickness and conductivity, for every zone.

Two larger (> 3 km) steeply dipping deformation zones are located in the layout at deposition level, see /Leander et al. 2009/. The zones are NS059A and NE107A and intersect transport tunnels. The calculated transmissivity values for NS059A and NE107A at deposition level are $1.1 \cdot 10^{-5}$ $m^2/s$ and $2.8 \cdot 10^{-6}$ $m^2/s$ respectively and the thickness of the zones are about 50 m and 35 m. Deformation zone NE107A also crosses the ramp at about depth level –75 m and about –180 m with calculated transmissivity values $5.2 \cdot 10^{-5}$ $m^2/s$ and $2.9 \cdot 10^{-5}$ $m^2/s$ respectively.

The shorter deformation zones (<3 km), which are presented in /SKB 2008b/, are 22 in number and a summary of the calculated transmissivity are presented in Table 3-2 as min, median and maximum value.

The shorter deformation zone klx11_dz11 intersects the exhaust ventilation shaft, according to the layout, at about depth level 400 m. The zone has a transmissivity of $7.5 \cdot 10^{-7}$ $m^2/s$ at this depth.

**Table 3-1. Hydraulic conductivity for different hydraulic rock mass domains and depth intervals (based on data given in SER /SKB 2008b/).**

| Hydraulic rock mass domain | Depth (m) | Hydraulic conductivity (m/s) | | |
|---|---|---|---|---|
| HRD_C | –150 | $2.1 \cdot 10^{-7}$ | | |
| | 150–400 | $2.4 \cdot 10^{-8}$ | | |
| | 400–650 | 90-percentile* | 50-percentile* (median) | 10-percentile* |
| | | $2 \cdot 10^{-8}$ | $5 \cdot 10^{-9}$ | $9 \cdot 10^{-10}$ |
| HRD_W | –150 | $2.8 \cdot 10^{-7}$ | | |
| | 150–400 | $2.9 \cdot 10^{-8}$ | | |
| | 400–650 | 90-percentile* | 50-percentile* (median) | 10-percentile* |
| | | $1 \cdot 10^{-7}$ | $4 \cdot 10^{-9}$ | $4 \cdot 10^{-11}$ |
| HRD_EW007 | –150 | $3.1 \cdot 10^{-7}$ | | |
| | 150–400 | $1.2 \cdot 10^{-7}$ | | |
| | 400–650 | 90-percentile* | 50-percentile* (median) | 10-percentile* |
| | | $5 \cdot 10^{-8}$ | $3 \cdot 10^{-8}$ | $2 \cdot 10^{-8}$ |

* based on the cumulative distribution for the sum of the transmissivity along 100 m of the tunnel (correlated) /SKB 2008b/.

**Table 3-2. Hydraulic characteristics of shorter (<3 km) deformation zones located in the deposition area, /SKB 2008b/.**

| | Min value | Median value | Max value |
|---|---|---|---|
| T ($m^2/s$) | $1.7 \cdot 10^{-7}$ | $4.6 \cdot 10^{-7}$ | $4.1 \cdot 10^{-6}$ |

### 3.3.2 Other input data and assumptions

The ground water pressure, H, has been set at the mean water pressure, with the assumption that the groundwater table is at ground level.

The radius, r, for the different underground openings is based on the geometries that are presented in /SKB 2008a/. The radius for tunnels is denominated $r_t$ and the radius for shafts $r_s$.

The distance to the edge of the sink, $R_0$ = 2,500 m is assumed, according to data for design step D1 /SKB 2004/.

The skin factor, $\xi$, varies between 2–5 according to /Emmelin et al. 2007/. In the calculations the skin factor is conservatively set at 2.

In the calculations of inflow into the shafts at the central area, a type shaft with a diameter of 4 m has been assumed. Since two of the smaller shafts are placed close together it is assumed that these two shafts correspond to one type shaft in the calculations.

Due to their size, the rock caverns are assumed to give the major contribution to the inflow to the central area, and the contribution from other adjacent tunnels thus can be neglected. This is because other tunnels and shafts have significantly smaller dimensions and are located adjacent to the large caverns.

## 3.4 Calculation result

A summary of the results of the inflow calculations before grouting is found in Table 3-3 to Table 3-6, see Appendix B for input data. The results correspond to "ground behaviour" in different conditions (hydraulic rock mass domains or deformation zones).

**Table 3-3. Calculated inflow of water before grouting, to underground openings belonging to functional area "accesses". The presentation of three values or only a single value depends on how input data has been presented in SER /SKB 2008b/.**

| Underground opening | Inflow per 100 m, (l/min) |
|---|---|
| **Ramp (depth 0–500 m), which is located in domain HRD_C, according to /Leander et al. 2009/.** | |
| Domain HRD_C (0–150 m) | 100 |
| Domain HRD_C (150–400 m) | 34 |
| Domain HRD_C (400–500 m) | 10-percentile: 2.0<br>Median: 11.0<br>90-percentile: 44 |
| | **Inflow per zone, (l/min)** |
| Zone NE107A (–75 m) | 250 |
| Zone NE107A (–180 m) | 290 |
| **Shafts inside ramp (depth 0–500 m), which are located in domain HRD_C, according to /Leander et al. 2009/.** | **Inflow per 100 m (l/min)** |
| Domain HRD_C (0–150 m) | 65 |
| Domain HRD_C (150–400 m) | 26 |
| Domain HRD_C (400–500 m) | 10-percentile: 1.6<br>Median: 9.0<br>90-percentile: 36 |

**Table 3-4. Calculated inflow of water before grouting, to underground openings belonging to functional area "central area". The presentation of three values or only a single value depends on how input data has been presented in SER /SKB 2008b/.**

| Underground opening | Inflow per 100 m, (l/min) |
|---|---|
| **Rock caverns (depth 500 m), which are located in domain HRD_C, according to /Leander et al. 2009/.** | |
| Domain HRD_C (400–500 m) | 10-percentile: 2.5<br>Median: 14<br>90-percentile: 55 |

**Table 3-5. Calculated inflow of water before grouting, to underground openings belonging to functional area "deposition area" including transport tunnels. The presentation of three values or only a single value depends on how input data has been presented in SER /SKB 2008b/.**

| Underground opening | Inflow per 100 m, (l/min) |
|---|---|
| **Deposition tunnels (depth 500 m)** | |
| Domain HRD_C | 10-percentile: 2.1<br>Median: 12<br>90-percentile: 47 |
| Domain HRD_W | 10-percentile: 0.1<br>Median: 9.4<br>90-percentile: 235 |
| Domain HRD_EW007 | 10-percentile: 47<br>Median: 71<br>90-percentile: 120 |
| | **Inflow per zone, (litre/min)** |
| Zones, < 3 km | Min.: 4.0<br>Median: 11<br>Max.: 95 |
| **Transport/main tunnels (depth 500 m)** | **Inflow per 100 m (litre/min** |
| Domain HRD_C | 10-percentile: 2.2<br>Median: 12<br>90-percentile: 49 |
| Domain HRD_W | 10-percentile: 0.1<br>Median: 9.8<br>90-percentile: 245 |
| Domain HRD_EW007 | 10-percentile: 49<br>Median: 75<br>90-percentile: 125 |
| | **Inflow per zone, (litre/min)** |
| Zone NS059A, passage in transport tunnel | 270 |
| Zone NE107A, passage in transport tunnel | 70 |
| Zones < 3 km | Min.: 4.3<br>Median: 12<br>Max.: 105 |

**Table 3-6. Calculated inflow of water before grouting, to ventilation shafts in functional area "deposition area". The presentation of three values or only a single value depends on how input data has been presented in SER /SKB 2008b/.**

| Underground opening | Inflow per 100 m, (l/min) |
|---|---|
| **Evacuation air shafts, which are located in domain HRD_C och HRD_W, according to /Leander et al. 2009/** | |
| Domain HRD_C (0–150 m) | 65 |
| Domain HRD_C (150–400 m) | 27 |
| Domain HRD_C (400–500 m) | 10-percentile: 1.7<br>Median: 9.3<br>90-percentile: 37 |
| Domain HRD_W (0–150 m) | 87 |
| Domain HRD_W (150–400 m) | 33 |
| Domain HRD_W (400–500 m) | 10-percentile: 0.1<br>Median: 7.4<br>90-percentile: 185 |
| Zone klx11_dz11 (400 m) | 12 |

Values presented in Tables 3-3 to 3-6 give values for the individual underground openings. In practice the inflow of water will vary within the functional area, especially for accesses (ramp and shaft) where a mean groundwater pressure has been assumed for the selected depth interval, see Appendix B.

## 3.5 Conclusions

For the ramp and shafts a considerable inflow of water can be estimated, especially in the uppermost depth interval 0–150 m. Furthermore the inflow of water in the ramp will probably be large where it is intersected twice by deformation zone NE107A.

In tunnels at deposition level and in the hydraulic rock mass domains HRD_C and HRD_W, the median inflow is approximately 10 litre/min, 100 m tunnel. The variation in calculated inflow is however large, especially in HRD_W. In the most water-bearing parts of these two domains high inflow can occur, that is more than 50 litre/min 100 m tunnel.

In most parts of the tunnels at deposition level in hydraulic rock mass domain HRD_EW007, the inflow will be more than 50 litre/min 100 m tunnel.

The two larger deformation zones, NS059A and NE107A, cause considerable inflow of water where they cross the transport tunnels. Considerable inflow of water in tunnels also occurs at maximal transmissivity in shorter deformation zones.

# 4 Basis for estimation of grouting measures

## 4.1 Introduction

According to UDP /SKB 2008a/, design in step D2 can be performed by using analytical calculation methods and/or experience from other grouting work. In this stage it is regarded as motivated to configure the grouting measures based on a combination of experience from other projects and calculations.

Experience from grouting work is described in Appendix A. A summary of the description of experience in Appendix A is made in Chapter 4.2.

In Chapter 4.3 analyses of different aspects regarding the difficulty of grouting have been made. These aspects are the predicted inflow after grouting, aperture and orientations of fractures and the extent of grouting in deposition tunnels and location of deposition holes.

Chapter 4.4 includes a summary and discussion of the analyses in Chapter 4.3.

## 4.2 Summary of grouting experience

This section is a summary of the literature study in Appendix A. As a conclusion from the literature survey cement based grouting is defined, in this report, as “existing/proven technique” and silica sol based grouting define as “new/unproven technique”.

The possibility to succeed with the grouting depends to a great extent on the characteristics of the rock mass and the requirements on tightness that are specified.

Experience of grouting at great depth in tunnels, in shaft sinking and in deep boreholes from the surface indicate that grouting can be carried out for the different grouting scenarios that are expected. However, this does not mean that such grouting is easy to carry out or that the need of complementary sealing work can be excluded. Grouting has not been sufficient in some projects and in some cases freezing combined with lining had to be used instead of grouting.

Grouting in sink shafts has been carried out with good results according to the same principles as when grouting in tunnels. Probe drilling is especially important from the point of view of safety when driving sink shafts, because uncontrolled inflow of water can quickly flood a shaft.

Grouting of raise bored shaft will be done in long boreholes. Experience is available from a number of different drilling procedures for the drilling of long boreholes, e.g. tophammer drilling, down-the-hole drilling, water-powered drilling systems or core drilling. Which drilling procedure is most suitable for Laxemar must be investigated further. In addition, a number of practical aspects must also be considered and checked when grouting in deep boreholes, e.g. handling of grout in transport down the hole, pressurizing of the grout, type of drill tubes, hoses and packers.

Pre-grouting with silica sol, at low or moderate depth (< 70 m), has so far shown good sealing results in superficial conditions, but the grouting procedure and equipment must be developed to achieve a more rational procedure. Moreover, the grouting procedure using silica sol puts other demands on personnel and equipment compared to conventional cement grouting. Good results of post-grouting using silica sol have been achieved as well as results where no sealing effect was achieved, i.e. similar experience to that of post-grouting in general.

For SKB’s ongoing sealing project at great depth (–450 m), see Appendix A, the project purpose is to demonstrate that it is possible to fulfil SKB’s requirement on a maximum inflow of 1 l/min and 60 metre tunnel (i.e. 5 l/min and 300 metre tunnel), at great depth. Both cement-based grout with low pH and silica sol grout have been used in the project, but due to the lack of larger fractures cement-based grout has been used to a relatively small extent. It is stated in that the main targets for the limited stretch of tunnel have been met, implying that the requirement on inflow of 1 l/min and 60 m tunnel at great depth has been fulfilled.

Especially when grouting with silica sol the grouting time is influenced to a great extent by the desired length of penetration in the fractures. In grouting made with silica sol at Äspö HRL /Funehag 2008/ the design was based on a penetration length, in the finest fractures, from the grouting holes of about 1.5 m or 2.5 m. These penetration lengths implied reasonable grouting times and practical control of the grouting.

## 4.3 Assessing the degree of difficulty for grouting

### 4.3.1 Predicted inflow after grouting

The degree of difficulty can be linked to the prediction of inflow after grouting. The inflow after grouting is calculated according to Equation 3-1, 3-2 and 3-4, using estimations of the hydraulic conductivity in the grouted zone.

Based on the experience of grouting performed, see Appendix A, the assessment is that the lowest hydraulic conductivity of the grouted zone, $K_g$, that can probably be achieved in the rock mass outside of the deformation zones is $1\cdot10^{-9}$ m/s, in the 100 meters scale, when using a cement-based grout.

However, when grouting from surface level in raise bored shafts a maximum tightness corresponding to $1\cdot10^{-8}$ m/s has been assumed. This is motivated by the anticipated higher degree of difficulty when grouting from surface level compared to grouting being carried out from the bottom of the shaft sinking (skip shaft) or at tunnel level.

For the deformation zones a corresponding value of $1\cdot10^{-8}$ m/s is estimated as possible. This is motivated in that a higher fracture frequency and more heterogeneous conditions occur in deformation zones.

A guide value, based on grouting experiences, in assessing maximum tightness of the grouted zone is also that the hydraulic conductivity before grouting can be reduced by a maximum of about twice the power of ten, when using a cement-based grout.

In the ongoing SKB sealing project at great depth, see Appendix A, it is reported that a hydraulic conductivity of about $1\cdot10^{-11}$ m/s could be obtained, both in rock mass outside deformation zones and in a deformation zone. When grouting in the deformation zone both cement and silica sol was used. It is also implied that fractures with a hydraulic width down to 10 µm could be sealed. These results must be considered as being at the limit of what is practically achievable with regard to time and other resources. In further analyses in this report a conductivity of $1\cdot10^{-10}$ m/s is considered a more reasonable value and is used both outside and in deformation zones.

The mean thickness of the grouted zone, t, has been set at 5 m. This value is set considering the requirement on limited grout spread.

In Table 4-1 a summary is made regarding maximum achievable tightness, that is the lowest value of the hydraulic conductivity, $K_g$.

The calculated inflow of water after grouting with cement based grouting are presented in Table 4-2, for ramp, shafts, transport tunnels and main tunnels. The calculated inflow of water after grouting for deposition tunnels is presented in Table 4-3.

Using proven technique, i.e. cement-based grouting, the requirements on inflow of water cannot be fulfilled generally in the raised shafts, rock caverns in the central area, main and transport tunnels in domain HRD_EW007 and also passage of the larger deformation zones NE107A and NE059A.

Furthermore, neither can inflow requirements in deposition tunnels be met by using only cement-based grouting. Along certain tunnel sections or individual tunnels the requirements on inflow will be met using cement based grouting, in particular in domain HRD_W.

Calculations have also been made with a lowest hydraulic conductivity of the grouted zone, $K_g$, corresponding to $1\cdot10^{-10}$ m/s, that is silica sol based grouting, for deposition tunnels. The result of these calculations is presented in Table 4-4.

**Table 4-1. Summary regarding hydraulic characteristics, rounded from Section 3.3 (rounded to the nearest integral number), and maximum achievable tightness.**

| Part of rock mass | Hydraulic characteristics, T ($m^2$/s) or K (m/s) | Lowest hydraulic conductivity of grouted zone, $K_g$ (m/s) |
|---|---|---|
| HRD_C<br>(0–150 m) | $K=2\cdot10^{-7}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-9}$ (cement)<br>$K_g=1\cdot10^{-8}$ (refers to raise shaft and cement) |
| HRD_C<br>(150–400 m) | $K=2\cdot10^{-8}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-9}$ (cement)<br>$K_g=1\cdot10^{-8}$ (refers to raise shaft and cement) |
| HRD_C<br>(400–500 m) | $K_{10\text{-percentile}}=9\cdot10^{-10}$<br>$K_{median}=5\cdot10^{-9}$<br>$K_{90\text{-percentile}}=2\cdot10^{-8}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-9}$ (cement) |
| HRD_W<br>(0–150 m) | $K=3\cdot10^{-7}$ | $K_g=1\cdot10^{-8}$ (refers to raise shaft and cement) |
| HRD_W<br>(150–400 m) | $K=3\cdot10^{-8}$ | $K_g=1\cdot10^{-8}$ (refers to shaft and cement) |
| HRD_W<br>(400–500 m) | $K_{10\text{-percentile}}=4\cdot10^{-11}$<br>$K_{median}=4\cdot10^{-9}$<br>$K_{90\text{-percentile}}=1\cdot10^{-7}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-9}$ (cement) |
| HRD_EW007<br>(400–500 m) | $K_{10\text{-percentile}}=2\cdot10^{-8}$<br>$K_{median}=3\cdot10^{-8}$<br>$K_{90\text{-percentile}}=5\cdot10^{-8}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-9}$ (cement) |
| Deformation zones, < 3 km<br>(500 m) | $T_{min}=2\cdot10^{-7}$<br>$T_{median}=5\cdot10^{-7}$<br>$T_{max}=4\cdot10^{-6}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-8}$ (cement) |
| Deformation zone, NS059A<br>(500 m) | $T=1\cdot10^{-5}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-8}$ (cement) |
| Deformation zone, NE107A<br>(at different levels) | $T_{75\ m}=5\cdot10^{-5}$<br>$T_{180\ m}=3\cdot10^{-5}$<br>$T_{500\ m}=3\cdot10^{-6}$ | $K_g=1\cdot10^{-10}$ (silica sol)<br>$K_g=1\cdot10^{-8}$ (cement) |
| Deformation zone klx11_dz11<br>(400 m) | $T=8\cdot10^{-7}$ | $K_g=1\cdot10^{-8}$ (refers to raise shaft and cement) |

**Table 4-2. Calculated inflow of water after cement based grouting for different functional areas with input data according to Appendix B. The presentations of 10-percentile, median and 90-percentile values correspond to input data in Section 3.3.1.**

| Functional areas/ underground openings | Inflow, per 100 m (litre/min) | Maximum permitted inflow per 100 m (litre/min) | Comments |
|---|---|---|---|
| **Accesses** | | | |
| Ramp, depth 0–500 m, HRD_C | Min: 0.9<br>Median: 6.5<br>Max: 28 | 10 | Min correspond to 10-percentile at depth 400–500 m<br>Max include def. zone NE107A at depth 180 m |
| Raise shaft, depth 0–500 m, HRD_C | Min: 1.9<br>Median: 17<br>Max: 31 | 10 | Min correspond to 10-percentile at depth 400–500 m<br>Max correspond to 90-percentile at depth 400–500 m<br>No def. zones in the shafts |
| Sink shaft, depth 0–500 m, HRD_C | Min: 1.7<br>Median: 5.2<br>Max: 10 | 10 | Min correspond to 10-percentile at depth 400–500 m<br>Max correspond to 90-percentile at depth 400–500 m<br>No def. zones in the shafts |
| **Central area** | | | |
| Rock caverns, HRD_C | 10-percentile: 2.5<br>Median: 11<br>90-percentile: 23 | 10 | |
| **Deposition area** | | | |
| Transport/Main tunnels, HRD_C | 10-percentile: 2.2<br>Median: 8.4<br>90-percentile: 15 | 10 | |
| Transport/Main tunnels, HRD_W | 10-percentile: 0.1<br>Median: 7.3<br>90-percentile: 20 | 10 | |
| Transport/Main tunnels, HRD_EW007 | 10-percentile: 15<br>Median: 17<br>90-percentile: 19 | 10 | |
| Deformation zones < 3 km in Transport/Main tunnels | Min: 4.0<br>Median: 9.9<br>Max: 60 | – | Inflow, per zone (l/min), def. zones properties in Table 3-2 |
| Transport tunnels, with deformation zone NE107A, HRD_C | 47 | 10 | Included Median inflow value, for HRD_C, outside def. zone, and the def. zone properties /SKB 2008b/ |
| Transport tunnels, with deformation zone NE107A, HRD_EW007 | 55 | 10 | Included Median inflow value, for HRD_EW007, outside zone, and the def. zone properties /SKB 2008b/ |
| Transport tunnels, with deformation zone NS059A, HRD_W | 86 | 10 | Included Median inflow value, for HRD_W, outside def. zone, and the def. zone properties /SKB 2008b/ |
| Exhaust raise shaft (0–500 m), HRD_C | Min.: 1.9<br>Median: 19<br>Max.: 33 | 10 | No def. zone in the shaft<br>Min correspond to 10-percentile at depth 400–500 m<br>Max correspond to 90-percentile at depth 400–500 m |
| Exhaust raise shaft (0–500 m), HRD_W | Min.: 0.1<br>Type: 20<br>Max.: 92 | 10 | Min correspond to 10-percentile at depth 400–500 m<br>Max include def. zone klx11_dz11 at depth 400 m |

**Table 4-3. Calculated inflow of water after cement based grouting for deposition tunnels. The presentation of 10-percentile, median and 90-percentile values corresponds to input data in Section 3.3.1.**

| Functional areas/ underground openings | Inflow, per 100 m (litre/min) | Maximum permitted inflow per 100 m (litre/min) | Comments |
|---|---|---|---|
| **Deposition area** | | | |
| Deposition tunnels (per tunnel), HRD_C | 10-percentile: 2.2<br>Median: 7.6<br>90-percentile: 13 | 1.7 | $K_g$ $1\cdot10^{-9}$ m/s (cement) Grouted zone: 5 m |
| Deposition tunnels (per tunnel), HRD_W | 10-percentile: 0.1<br>Median: 6.7<br>90-percentile: 16 | 1.7 | $K_g$ $1\cdot10^{-9}$ m/s (cement) Grouted zone: 5 m |
| Deposition tunnels (per tunnel), HRD_EW007 | 10-percentile: 13<br>Median: 14<br>90-percentile: 15 | 1.7 | $K_g$ $1\cdot10^{-9}$ m/s (cement) Grouted zone: 5 m |
| Deformation zones < 3 km in deposition tunnels | Min: 3.7<br>Median: 8.9<br>Max: 49 | – | $K_g$ $1\cdot10^{-8}$ m/s (cement) Inflow, per def. zone (l/min) Grouted zone: 5 m |

**Table 4-4. Calculated inflow of water after grouting with silica sol for deposition tunnels. The presentations of 10-percentile, median and 90-percentile values correspond to input data in Section 3.3.1.**

| Functional areas/ underground openings | Inflow, per 100 m (litre/min) | Maximum permitted inflow per 100 m (litre/min) | Comments |
|---|---|---|---|
| **Deposition area** | | | |
| Deposition tunnels (per tunnel), HRD_C | 10-percentile: 1.0<br>Median: 1.5<br>90-percentile: 1.7 | 1.7 | $K_g$ $1\cdot10^{-10}$ m/s (silica sol) Grouted zone: 5 m |
| Deposition tunnels (per tunnel), HRD_W | 10-percentile: 0.1<br>Median: 1.5<br>90-percentile: 1.7 | 1.7 | $K_g$ $1\cdot10^{-10}$ m/s (silica sol) Grouted zone: 5 m |
| Deposition tunnels (per tunnel), HRD_EW007 | 10-percentile: 1.7<br>Median: 1.7<br>90-percentile: 1.7 | 1.7 | $K_g$ $1\cdot10^{-10}$ m/s (silica sol) Grouted zone: 5 m |
| Deformation zones < 3 km in deposition tunnels | Min: 0.2<br>Median: 0.3<br>Max: 0.9 | – | $K_g$ $1\cdot10^{-10}$ m/s (silica sol) Inflow, per def. zone (l/min) Grouted zone: 5 m |

Grouting based on silica sol and with a sealed zone of about 5 m outside the contour of the tunnel the requirements on inflow of water are met in all hydraulic domains. Similar calculations with silica sol based grouting for rock caverns in the central area, main and transport tunnels give corresponding results confirming that requirements on inflow are met.

Using a smaller thickness of the grouted zone in the calculations, the requirements on inflow will not be met. Accordingly, this implies that thickness of the grouted zone must be at least 5 m, which in turn must be considered in the detailed design of the pre-grouting measures.

### 4.3.2 Fracture statistics

SER /SKB 2008b/ presents the hydraulic fracture statistics, with fracture frequency and transmissivity for each hydraulic domain and for different depth intervals. Table 4-5 presents fracture statistics for relevant hydraulic domains and depth intervals, according to /SKB 2008b/.

Decisive for what tightness can be achieved is how the grout penetrates and spreads in fractures in the rock mass. Grouts have, however different possibilities of penetrating the finer fractures depending on composition of the grout, e.g. mixing procedure and additives. The analyses aiming at a grouting design must therefore result in an assessment of the aperture of fractures that must be sealed.

**Table 4-5. Hydraulic fracture statistics for individual water-bearing fractures in the relevant hydraulic domains HRD_C, HRD_W and HRD_EW007 per depth interval (SER, /SKB 2008b/).**

| Hydraulic domain and depth interval | Transmissivity of individual water-bearing fractures, minimum, average, maximum ($m^2/s$) | |
|---|---|---|
| **HRD_C** | | |
| 50–150 m | Min. = $4 \cdot 10^{-10}$<br>Average = $3.2 \cdot 10^{-8}$<br>Max. = $4.9 \cdot 10^{-5}$ | StDev. = LOG 1.1 |
| 150–400 m | Min. = $4 \cdot 10^{-10}$<br>Average = $1.3 \cdot 10^{-8}$<br>Max. = $3.2 \cdot 10^{-5}$ | StDev. = LOG 1.1 |
| 400–650 m | Min. = $3.2 \cdot 10^{-10}$<br>Average = $7.9 \cdot 10^{-9}$<br>Max. = $1 \cdot 10^{-6}$ | StDev. = LOG 0.9 |
| **HRD_W** | | |
| 50–150 m | Min. = $4 \cdot 10^{-10}$<br>Average = $3.2 \cdot 10^{-8}$<br>Max. = $4.7 \cdot 10^{-5}$ | StDev. = LOG 1.0 |
| 150–400 m | Min. = $1 \cdot 10^{-9}$<br>Average = $1.3 \cdot 10^{-8}$<br>Max. = $7.9 \cdot 10^{-5}$ | StDev. = LOG 1.6 |
| 400–650 m | Min. = $6.3 \cdot 10^{-10}$<br>Average = $3.2 \cdot 10^{-8}$<br>Max. = $1 \cdot 10^{-4}$ | StDev. = LOG 1.4 |
| **HRD_EW007** | | |
| 50–150 m | Min. = $4 \cdot 10^{-10}$<br>Average = $4 \cdot 10^{-8}$<br>Max. = $3.2 \cdot 10^{-5}$ | StDev. = LOG 1.2 |
| 150–400 m | Min. = $3.2 \cdot 10^{-10}$<br>Average = $3.2 \cdot 10^{-8}$<br>Max. = $4 \cdot 10^{-5}$ | StDev. = LOG 0.9 |
| 400–650 m | Min. = $7.9 \cdot 10^{-10}$<br>Average = $2.5 \cdot 10^{-8}$<br>Max. = $2 \cdot 10^{-6}$ | StDev. = LOG 0.7 |

It is not so simple however to determine the aperture of fractures, since the network of fractures by its nature is complicated and must be contemplated with regard to the type of flow-dimensionality that occurs. Simplification of the fracture aperture can be made by the concept of the hydraulic fracture aperture /Snow 1968/ expressed by Equation 4-1:

$$T_s = \frac{b_{hyd}^3 \cdot \rho_w \cdot g}{12 \cdot \mu_w} \qquad 4\text{-}1$$

where

$T_s$ = transmissivity of an individual fracture ($m^2/s$)

$b_{hyd}$ = hydraulic fracture aperture (m)

$\rho_w$ = density of water ($kg/m^3$)

$\mu_w$ = viscosity of water (Pas)

g = acceleration of gravity ($m/s^2$)

Equation 4-1 gives:

$$b_{hyd} \approx 0.01 \cdot \sqrt[3]{T_s} \qquad 4\text{-}2$$

The hydraulic fracture aperture is based on assumptions of simplified relationships, e.g. that the fractures are plane-parallel with a constant fracture aperture. It should be noted that the hydraulic aperture is not equal to the average physical aperture /Eriksson and Stille 2005/. How much smaller is however not distinct.

It is not obvious which fractures that needs to be grouted. By using Equation 4-2 for a specific number of fractures, it can be seen that an equivalent sealing effect can for example be obtained if the fractures with a large aperture are sealed to a great extent or whether all fractures are sealed to a smaller extent. Sealing one fracture can possibly also prevent water inflow from another fracture even if this is not sealed.

In design step D2 cement based grouts are to be used for "larger" hydraulic fracture apertures, that is >0.1 mm, according to /Emmelin et al. 2007/. Using Equation 4-1 it is implied that cement grouting would be suitable if T $>1 \cdot 10^{-6}$ m²/s.

Figure 4-1 presents the estimated distribution of individual water-bearing fractures in all domains at deposition level, i.e. depth 500 m. The distributions of aperture are based on Equation 4-1 and hydraulic fracture statistics in Table 4-5.

Figure 4-1 illustrates how the hydraulic fracture aperture varies from domain to domain; for example in domain HRD_W there are some fractures that can be grouted with cement (>100 µm) while in hydraulic domain HRD_C and HRD_EW007 there are hardly any fractures apertures wider than 100 µm.

For the deformation zones, considerable variations in fracture aperture can be anticipated depending on transmissivity and depth of the deformation zones. Accordingly, hydraulic apertures both bigger and smaller than 100 µm can occur in the deformation zones, implying that both cement and silica sol is needed.

### 4.3.3 Fracture orientation

In /SKB 2008b/ the fracture orientation is also presented, at deposition level. It is worth noticing that the more transmissive fractures are gently dipping in domain HRD_C and HRD_W and the vertical fractures have a main orientation approximately parallel to the deposition tunnels in all domains. This should imply that the degree of difficulty increases, because of the orientation of grouting holes, which are normally relatively parallel to the deposition tunnels. To reduce the degree of difficulty the grouting holes should sprawl out from the tunnel and angle up or down. Thus grouting holes inside the tunnel contour will be unfavourable because they will be nearly horizontal.

As a comment on which orientations of fractures that are the most essential to seal is that the gently dipping fractures at great depth are dependent on the vertical fractures that convey water down to the gently dipping fractures. This would mean in principle that if all vertical fractures and structures are tight then the gently dipping fractures will not conduct any water into the tunnels.

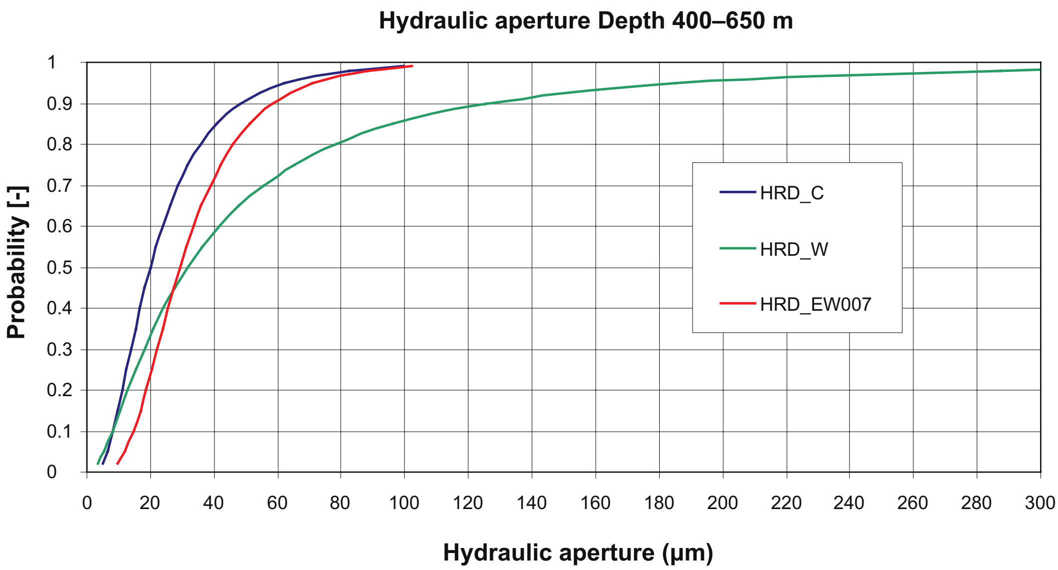


*Figure 4-1.* *Hydraulic fracture aperture distribution in all hydraulic domains at deposition level, that is depth 400 m to 650 m.*

### 4.3.4 Extent of grouting in deposition tunnels

SER /SKB 2008b/ present cumulative distributions for the sum of the transmissivity along 20 and 100 meter sections for deposition tunnels in domain HRD_C, HRD_W and HRD_EW007 at depth 500 m, see Appendix D.

An analysis of transmissivity data along a 20 meter section has been made to obtain an assessment of the extent of necessary grouting for deposition tunnels. A tunnel section of 20 m correspond to the length of a grouting fan.

An estimate of the necessary transmissivity, surrounding a 20 meter deposition tunnel, to avoid grouting can be made using Equation 3-2. The criterion of grouting or not in a 20 meters tunnel is roughly assumed to be a fifth of the requirement on inflow, i.e. 1.7 l/min dived by 5. A transmissivity along a 20 metre section at about $1 \cdot 10^{-8}$ m²/s or lower would indicate that no grouting is necessary to fulfill the requirement.

An assessment has been made of how many of the grouting fans (%) in the deposition tunnels of 20 meters that need to be grouted. Figure 4-2 is based on the distribution of the sum of transmissivity along 20-metre tunnel, see Appendix D, at different intervals.

From Figure 4-2 an approximation can be deduced of how many 20 m sections (grouting fans) in deposition tunnels need to be grouted, i.e relative number (%). The criterion for grouting or not is roughly a transmissivity of about $1 \cdot 10^{-8}$ m²/s for a 20 metre long section of tunnel. Table 4-6 presents how many of all 20 metre long tunnel sections that need grouting.

A rough estimation could also be done of how many of the grouting fans that can be grouted with cement. The limit for cement grouting in one fracture is $1 \cdot 10^{-6}$ m²/s, based on the assumption that cement grouting is only possible in fracture apertures greater than 100 µm, see Section 4.3.2. Since the frequency is low between hydraulic fractures with a transmissivity of $1 \cdot 10^{-6}$ m²/s or higher, according to SER /SKB 2008b/, it could be assumed that one hydraulic fracture, with a transmissivity of $1 \cdot 10^{-6}$ m²/s or higher, contributes the most flow in a 20 m tunnel section.

From Figure 4-2 an approximation can be deduced of how many 20 m sections, relative number of grouting fans (%), in deposition tunnels that could be grouted with cement. Table 4-7 presents the relative amount of 20 m sections that would contain a hydraulic fracture that can be grouted with cement.

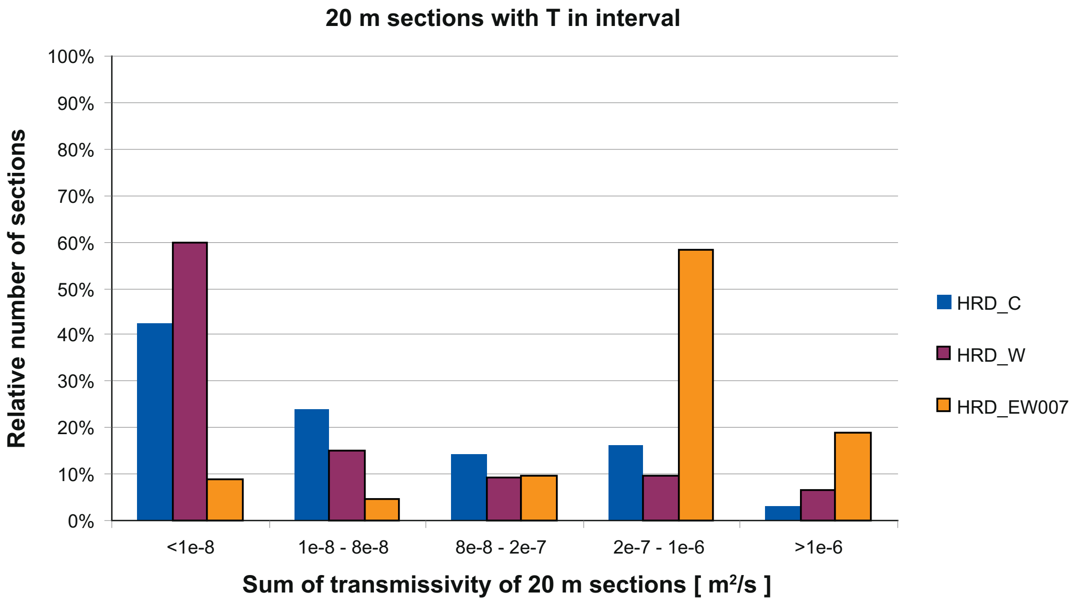


*__Figure 4-2.__ Sum of transmissivity (correlated data) along 20-metre sections in deposition tunnels, based on SER /SKB 2008b/ and Appendix D, with the interval $<1 \cdot 10^{-8}$ m²/s; $1 \cdot 10^{-8}$–$8 \cdot 10^{-8}$ m²/s; $8 \cdot 10^{-8}$–$2 \cdot 10^{-7}$ m²/s; $2 \cdot 10^{-7}$–$1 \cdot 10^{-6}$ m²/s and $>1 \cdot 10^{-6}$ m²/s.*

**Table 4-6. Amounts of 20-metre tunnel sections, that need grouting, that is sections with transmissivity, $T > 1 \cdot 10^{-8}$ m²/s.**

| Hydraulic domain | Amount of 20-metre sections that need grouting |
|---|---|
| HRD_C | About 60%. |
| HRD_W | About 40%. |
| HRD_EW007 | About 90%. |

**Table 4-7. Amounts of 20-metre tunnel sections, that could be grouted with cement, that are sections with transmissivity, $T > 1 \cdot 10^{-6}$ m²/s.**

| Hydraulic domain | Amount of 20-metre sections that can be grouted with cement |
|---|---|
| HRD_C | About 5%. |
| HRD_W | About 5%. |
| HRD_EW007 | About 20%. |

### 4.3.5 Extent of grouting in locations for deposition holes

According to UDP /SKB 2008a/ drilled deposition holes are not to be sealed and deposition holes with leakage >0.1 l/min are to be rejected (see Chapter 2.3). However, grouting in locations for deposition holes may be needed before the drilling of deposition holes in order to meet the requirements on inflow of water to both deposition tunnels and deposition holes.

SER /SKB 2008b/ present cumulative distributions for the sum of the transmissivity along 8 metre sections in vertical direction in domain HRD_C, HRD_W and HRD_EW007 at depth 500 m. A vertical section of 8 m correspond to the length of a deposition hole.

An analysis of transmissivity data along a 8 meter vertical sections has been made to obtain an assessment of the extent of necessary grouting in locations for deposition holes.

For a rough estimation of the inflow to a bore hole, Equation 4-3 can be used /Fransson 2001/. The inflow to a deposition hole, $Q_s$, of 0.1 l/min corresponds to a transmissivity of about $3 \cdot 10^{-9}$ m²/s according to Equation 4-3. The pressure difference, dH, in this case is assumed to be 500 m, which is probably a conservative assessment because of the possible pressure decrease round an excavated tunnel.

$$T = \frac{Q_s}{dH} \qquad \text{4-3}$$

Figure 4-3 is based on the distribution of the sum of transmissivity along 8-metre vertical sections /SKB 2008b/, at different intervals.

From Figure 4-3 an approximation can be deduced of how many 8 m vertical sections (deposition holes) need to be grouted, i.e relative number (%). The criterion for grouting or not is roughly a transmissivity of about $3 \cdot 10^{-9}$ m²/s. Table 4-8 presents how many of all 8 metre vertical sections that need grouting.

In Table 4-8 it is assumed that tightness round the deposition holes is not affected by the pre-grouting round the deposition tunnel. With an assumed thickness of the grouted zone at 5 m, within which the hydraulic conductivity shall be at least $1 \cdot 10^{-10}$ m/s, even many of the water-bearing fractures within 8 m from the tunnel contour will probably be grouted.

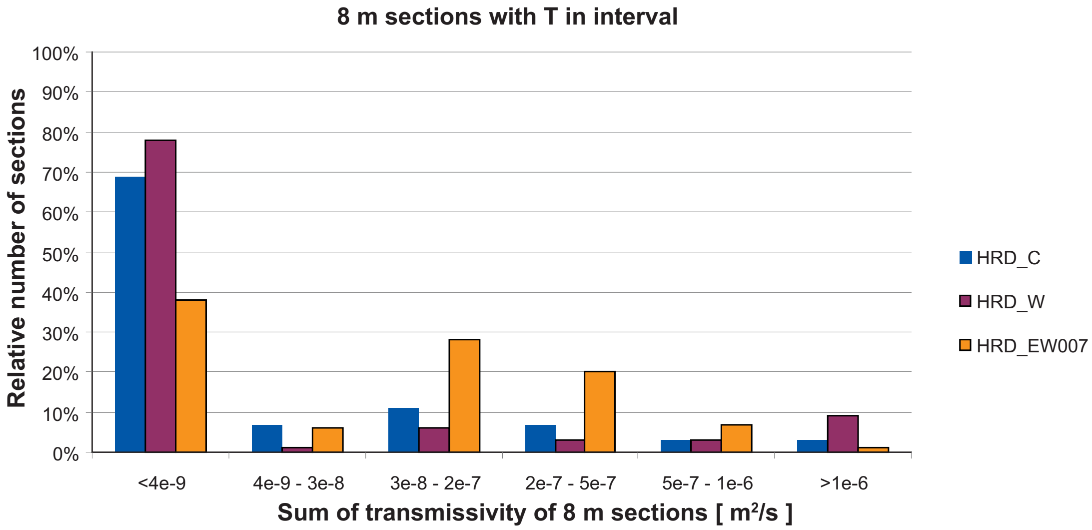


***Figure 4-3.** Sum of transmissivity of 8 metre long sections in direction of the deposition holes in different intervals (correlated data) /SKB 2008b/.*

**Table 4-8. Amounts of 8-metre vertical sections, that need grouting, that is sections with transmissivity, $T > 3 \cdot 10^{-9}$ m²/s.**

| Hydraulic domain | Amount of 8-metre vertical sections that need grouting |
|---|---|
| HRD_C | About 30%. |
| HRD_W | About 20%. |
| HRD_EW007 | About 60%. |

## 4.4 Conclusions

It can be concluded that it is uncertain whether the requirement on inflow will be fulfilled for the following underground openings:

- The requirement on inflow of water using existing and proven technique, i.e. cement based grouting, is not fulfilled for the raise bored shafts. This is probably most likely for the shafts in the deposition area which will be grouted in 500 m long boreholes. The shafts in the central area will be grouted in 100 m stages from niches in the ramp. There is no experience available of grouting with silica sol in long vertical boreholes

- For deposition tunnels in all hydraulic domains and main and transport tunnels in domain HRD_EW007, the requirement on inflow will not be met using proven technique based on cement grouting. If a systematic pre-grouting using the more unproved grouting technique based on silica sol is applied and the grouted zone is at least 5 m the requirements will be fulfilled.

Selective grouting in deposition tunnels is possible in hydraulic domain HRD_C and HRD_W. The grouting requisites are in the two hydraulic domains about 50% (about 60 and 40% according to Table 4-6). To enable this type of grouting probe drilling and hydraulic tests at the tunnel face it is essential to enable a decision as to whether grouting is needed or not. Criteria for whether grouting is needed or not are related to the results from the hydraulic tests. The extent of hydraulic tests and interpretation of the hydraulic results in relation to the grouting requisite are today related to uncertainties.

Systematic pre-grouting is necessary in all tunnels in hydraulic domain HRD_EW007. The grouting requisite in the deposition tunnels, in domain HRD_EW007, is nearly 100% (about 90% according to Table 4-6).

The inflow of water in individual tunnels and rock caverns and whether the requirements on inflow can be fulfilled by cement grouting depends partly on where the tunnel is located in the hydraulic domain and partly on whether any deformation zone is crossed. It could for example mean that several tunnels in an area, especially within domain HRD_W, can be excavated without major grouting. There is also the scenario that significantly different grouting measures may be used in one and the same tunnel, implying that a particular part of the tunnel needs extensive grouting and another part of the tunnel no grouting at all.

Only a small amount of depositions tunnel sections, in 20 meters scale, could be grouted with cement, i.e. have fractures with a transmissivity higher than $1 \cdot 10^{-6}$ $m^2/s$. This fact implies that grouting with silica sol is necessary as the main grout.

Grouting in locations for deposition holes are necessary if the inflow is > 0.1 l/min /SKB 2008a/. However, there is a requirement /SKB 2008a/ that no holes may be drilled so that they intersect locations for deposition holes. The interpretation of this requirement is that grouting holes in the bottom of the tunnel must be drilled inside the tunnel contour. From a grouting point of view, probe holes may also be needed in the location of possible deposition holes. The motive for these holes is that an estimate of inflow to deposition holes and possible grouting should be made before drilling of a deposition hole. The estimated extent of grouting in locations for deposition holes is greatest in domain HRD_EW007, about 60% according to Table 4-8. The extent of grouting may be lower because of the effect by the pre-grouting round the deposition tunnel.

Further more the design of the grouting fans must consider the fracture orientation.

# 5 Grouting measures

## 5.1 Strategy for establishing grouting measures

The following section presents the strategy that has been chosen as the starting point for configuring the grouting measures, including fan geometry, grout, execution, equipment and verifications. The strategy is based on the premises stated in Chapter 2, calculations of inflow before grouting (Chapter 3), experience of performed grouting (Chapter 4.2 and Appendix A) and analyses of degree of difficulty (Chapter 4.3 and 4.4):

- Test drilling and grouting trials from surface level are to be made before starting on ramp and shafts to the central area. This drilling and grouting should be carried out in possible locations for ramp and shafts. In this way, the drilling and grouting measures could be tested and adjusted before large-scale production begins.
- For the raise shafts that are to be drilled sealing by grouting will not be sufficient. It is evident that alternatives to cement grouting must be available before the excavation work begins. An alternative to the drilled raise shafts could be to change from the present driving technique to the sink shaft method in which screen grouting can be carried out in stages. In sink shaft driving it would be possible to seal the shaft by cement and silica sol grouting. Nevertheless, sealing by grouting from the surface in deep boreholes shall be made initially with the purpose to reduce large inflows.
- The shink shaft is grouted mainly from the face of the excavation. The grouting procedure is to begin with curtain grouting from the surface to create better conditions for the shaft sinking. Less time will then be needed for grouting from the bottom of the shaft and a more rational shaft sinking is enabled.
- Preparedness for rapid hardening grout (e.g. with added accelerators) shall be available as well as alternative sealing methods (e.g. freezing and/or lining) when excavating tunnels and rock caverns at greater depth.
- Grouting using cement based grout is considered “proven technique”, in accordance with criteria of /SKB 2008a/, and shall be used where possible. Grouting with silica sol grout at great depth it is not considered “proven technique” but it will be necessary on a large scale to meet the requirements. Grouting trials at considerable depth using silica sol are however in progress at present, under the auspices of SKB, to test execution and sealing result /Funehag 2008/.
- The grouting for the functional areas will be either systematic or selective. The type of grouting that will be used depends on the hydraulic conditions and the requirements on inflow of water. Moreover, post-grouting will be necessary to seal larger point leakage following blasting of rock mass. Post-grouting to seal minor leakage is difficult and the success is uncertain.
- Systematic pre-investigation and probe drillings will be essential to ensure favourable sealing results by grouting and to enable decisions on grouting types. Pre-investigation drilling is made to verify rock-mass prognosis with locations of deformation zones and probe drilling is made to enable decisions on detailed grouting procedure for implementation.
- For deposition tunnels the strategy is as follows:
  - Pre-grouting in accordance with principles used in SKB’s fine sealing project with silica sol and cement at Äspö HRL, see /Funehag 2008/
  - Selective grouting in HRD_C and HRD_W.
  - Systematic grouting will be made in HRD_EW007.
  - Probe holes are to be done in positions for deposition holes.
  - Post-grouting is to be done if point leakage in the tunnel is >1 l/min.

## 5.2 General principles

### 5.2.1 Grouting types

The grouting measures should according to SER /SKB 2008b/ be grouped into three so called grouting types. The grouting types include different measures for pre-grouting, such as type of grout, fan geometry and execution. In the following descriptions of grouting measures, these grouting types have been defined as:

- Grouting type 1 (GrT1): Selective grouting
- Grouting type 2 (GrT2): Systematic grouting
- Grouting type 3 (GrT3): Systematic grouting including special measures

### 5.2.2 Grouts

The main principle is that the grout is selected in relation to the hydraulic fracture aperture that has been estimated. Cement-based grouts are to be used for "major fractures" and silica sol for "minor fractures". In this case according to /Emmelin et al. 2007/ "major fractures" refer to fractures with a hydraulic fracture width ≥100 µm. Recipes and characteristics for cement based grouts and silica sol have been provided by SKB, see Appendix C. It may be necessary to adjust the recipe to obtain the desired properties depending on the properties of individual fractures.

For design step D2 the assessments is that the different available grouts are sufficient for the grouting that can be anticipated. "Plug grout" is normally used as hole-filling grout but can also be used in extreme situations, as a blocking grout to make a temporary stop in grouting. The "stop grout" can for example be used in the first round in more fractured rock at greater depth and also be applied to limit the grout spread. Lastly, the "injection grout" can be used as the main grout when cement is to be used for sealing in all hydraulic rock domains/zones and depths. Silica sol is used as main grout in deposition tunnels and as supplement in other tunnels and rock caverns and also for post-grouting of point leakage.

### 5.2.3 Grouting fan

All grouting types and related procedures are based on base grouting fans which in detail depend on geometry of the individual openings of the underground facility. Figure 5-1 illustrates the base grouting fans for the different geometries in the deposition area. It should be noted that holes in the bottom of the deposition tunnels could be drilled outside tunnel countor, when the tunnel is passing a deformation zone. Furthermore, the bottom holes shall be adapted so that the greatest possible angle is achieved in relation to the dominating fracture system.

It shall also be noted that the grouting fans include tunnel-front holes, i.e. holes located at the face of the tunnel. In grouting types 2 and 3 control holes will be drilled. In the control holes hydraulic tests should be made with the aim of checking the sealing result. In grouting type 1 some control holes will also be drilled, depending on results from probe holes and grouting work.

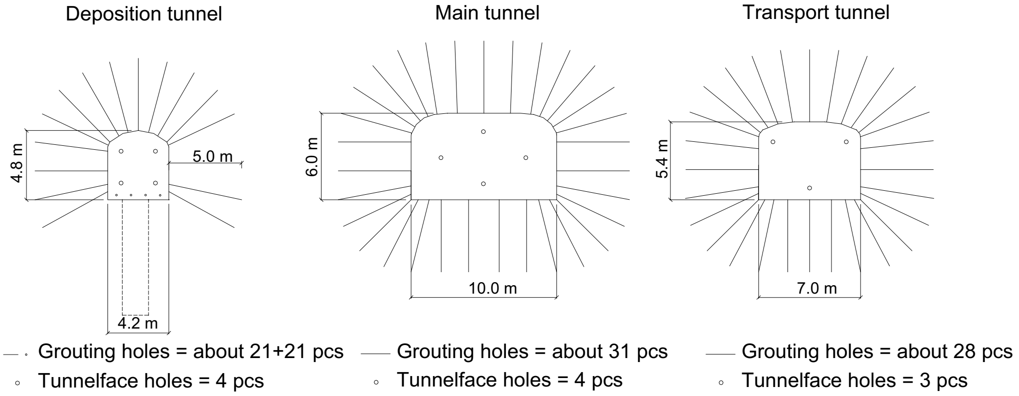


***Figure 5-1.*** *Principle execution of base pre-grouting fans in the deposition area.*

### 5.2.4 Execution and equipment

The grouting pressure ($p_g$), i.e. total pressure, is mainly based on the groundwater pressure ($p_w$) /Eriksson and Stille 2005/ and the risk of jacking /Fransson and Gustafson 2006, Fransson 2008/, i.e. rock mass load ($\rho_b gd$), and could be expressed as:

$$3\rho_b gd - 2p_w \geq p_g \geq 2p_w \qquad \text{5-1}$$

This means that the grouting pressure should be at least twice the groundwater pressure /Eriksson and Stille 2005/. This yields relatively large grouting pressure at full groundwater pressure at repository depth, compared to conventional grouting in superficial conditions. The reason for recommending the relatively large overpressure is partly to avoid reverse flow when grouting stops and partly to prevent erosion of the grout before it reaches sufficiently high strength. Further, the erosion risk could be reduced if the cement-based grout has a certain yield value /Axelsson 2009/. With a grouting time for silica sol at 4/5 of the gel time the risk for erosion can be reduced somewhat even for silica sol grouting /Funehag 2008/.

For practical and production-adapted grouting the grout injection time should be an important control parameter. Control of the grout injection time will enable a better control of the grout spread, which for example may be needed when grouting near ground surface. According to /Gustafson and Stille 2005/ and /Funehag 2007/ the grout injection time can be adjusted in detail on the basis of theoretical relationships, in which necessary penetration length, estimated hydraulic fracture aperture, selected pressure and hole spacing, and also tested properties of the grout are input data. For silica sol grout injection time must also be based on the selected gel induction time.

The experience that exists with regard to mixing cement-based grouting media of low pH is that the performance and quality of the mixing equipment is of great significance to ensure repeatable properties of different mixing occasions /Ranta-Korpi et al. 2007/. This also signifies that checking of equipment, cleaning and mixing times must be regular.

An adapted mixing and pumping procedure is necessary when grouting with silica sol. In the present procedure one batch is prepared for each separate grouting hole and when reaching the pre-defined grout injection time the grouting of the hole is stopped. The remaining grout in the equipment is emptied (from mixer to packer hose connection) and the equipment is cleaned before starting on the next grouting hole. This procedure requires more planning, logistics and time than normal, even if two holes could be grouted simultaneously. Accordingly, there is need of development with regard to equipment that could enable more efficient grouting using silica sol. Figure 5-2 shows a conventional grouting equipment, with mixer, agitator and pump for cement base grout or silica sol grout and also the control panel.

***Figure 5-2.*** *A grouting equipment, with mixer, agitator and pump for cement base grout or silica sol grout and also the control pane. Foto from SKB:s sealing project at great depth.*

All equipment, e.g. hoses and couplings, must be designed for the high total pressures that apply at the repository depth.

In addition, a venting system/equipment must be provided to vent the grouting holes before grouting begins.

## 5.3 Choice of preliminary grouting measures in different functional areas

### 5.3.1 Summary of preliminary grouting measures

Table 5-1 summarises each functional area with regard to grouting type, main grout and also aspects regarding execution of grouting, such as the need of special equipment, several grouting rounds or checks. The term "cement" refers to one or several of the cement-based grouts provided by SKB. In connection with detailed design the composition of grouts may need to be adjusted. For example, particular grout properties may be required when grouting deep boreholes from the surface.

### 5.3.2 Accesses

#### *Ramp*

The main grouting in the ramp will be made down to a depth of 400 m, and especially between 0–150 m, see Figure 4-1. The grouting type are selective grouting, GrT 1, and systematic grouting, GrT 2. When passing high conductive deformation zones, the grouting could be extensive and with special measure GrT 3.

The grouting fans, including the tunnel-front holes, should follow the principles in Figure 5-1, with holes outside the tunnel counter.

#### *Shafts*

**General**

The shafts will be done in two different ways, either by shaft sinking (skip shaft) or by expanding the shafts using raise-drilling technique (lift and ventilation shafts).

With regard to the uncertainty concerning fulfilment of requirements on inflow to the drilled shafts, methods for post-grouting in shafts need to be developed to reduce the inflow of water to an acceptable level. Certain development of equipment and accessories may therefore be needed because of cramped conditions in the shafts. The possibility of using the shaft sinking technique for these shafts should also be further studied.

**Skip shaft**

The skip shaft is to be excavated from the top by drilling and blasting. The grouting can be carried out in a conventional manner in connection with the shaft sinking. In principle this means that the same grouting fans that apply for tunnels in the respective grouting types can be used although drilled vertically instead of horizontally, see Figure 5-3. This type of grouting is sometimes denoted as "cover grouting". It is suggested that some of the curtain grouting holes are extended in the sink shaft down to 500 m to reduce the risk of serious and uncontrolled leakage of water in the shaft sinking, according to general experiences and recommendation (see Appendix A).

**Lift and ventilation shafts through the central area**

As pointed out earlier sealing by grouting will not be sufficient to cope with requirements on inflow of water to lift and ventilation shafts. However, initial sealing will begin by grouting deep holes to reduce the considerable inflow.

The grouting of these shafts will be carried out before starting the raise drilling. The grouting is carried out in long, vertical boreholes which are drilled in a ring outside the contour of the shafts.

**Table 5-1. Summary of selected grouting types, GrT, and principles for grouting in different functional areas.**

| Functional area /underground opening | Choice of grouting type, GrT | Grout | Execution aspects |
|---|---|---|---|
| **Accesses** | | | |
| **Ramp/shaft** | | | |
| HRD_C (0–150 m) | 2 | Cement, compl. with silica sol | Systematic pre-grouting. Extensive pre-grouting through def. zones.<br>Grouting in shafts made from the surface and from niches in ramp. Special measures for grouting in vertical boreholes. |
| HRD_C (150–500 m) | 1 and preparedness for 2 (3 in def. zone) | Cement, compl. with silica sol | Selective pre-grouting or systematic pre-grouting in some sections. Extensive pre-grouting through def. zones.<br>Grouting in shafts made from the surface and from niches in ramp. Special measures for grouting in vertical borehole. |
| **Central area** | | | |
| **Rock caverns** | | | |
| HRD_C (–500 m) | 1 and preparedness for 2 | Cement, compl. with silica sol | Selective pre-grouting and systematic pre-grouting in some sections. |
| **Deposition area** | | | |
| **Deposition tunnels** | | | |
| HRD_C and HRD_W with deformation zones (– 500 m) | 1 and preparedness for 2 and 3 | Silica sol, compl. with cement | Selective grouting and extensive pre-grouting through def. zones. Equipment for high pressure and flows, in def. zones. |
| HRD_EW007 with deformation zones (–500 m) | 2 and preparedness for 3 | Silica sol, compl. with cement | Systematic pre-grouting and extensive pre-grouting through def. zones. Equipment for high pressure and flows, in def. zones.<br>Systematic probe drilling in locations for deposition holes. |
| **Deposition area** | | | |
| **Transport tunnels, main tunnels** | | | |
| HRD_C and HRD_W with deformation zones (–500 m) | 1 and preparedness for 3 | Cement, possible compl. with silica sol | Selective pre-grouting.<br>Extensive systematic pre-grouting through def. zones and equipment for high pressure and flows, in def. zones.<br>Possible long-hole grouting with special equipment through def. zones. |
| HRD_EW007 with deformation zones (–500 m) | 2 and preparedness for 3 | Silica sol, possible compl. with cement | Systematic pre-grouting.<br>Extensive systematic pre-grouting through def. zones and equipment for high pressure and flows, in def. zones.<br>Possible grouting with special equipment through def. zones. |
| **Exhaust shaft (0–500 m)** | | | |
| HRD_C and HRD_W (0–500 m) | 2 | Cement | Preliminary curtain grouting from surface level. Systematic grouting.<br>Special measures for grouting in vertical boreholes. |

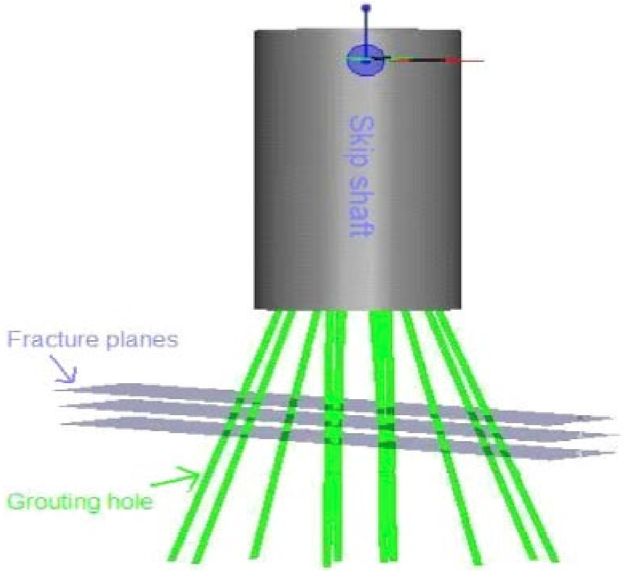


*Figure 5-3. Principle of grouting in skip shaft.*

Furthermore, the shafts down to the central area will be accessible from the ramp every 100 m, which is an advantage with regard to grouting because the work can be done in 100-m stages.

The principle of grouting lift and ventilation shafts is that the grouting holes are drilled about 25 m deep and with hole spacing depending on the detail design, i.e. grouting round 1. Hydraulic tests are then made, followed by grouting with cement-based grouts and renewed drilling of grouted holes and subsequent hydraulic tests, in grouting round 1. If a desired sealing effect is achieved, a new stage of about 25 m is drilled; otherwise the grouting procedure is repeated, i.e. grouting round 2. The boreholes in grouting round 2 are located between the holes in round 1 and processed in a similar way as for grouting round 1 in stages of 25 metres.

Figure 5-4 presents the principle for grouting the lift and ventilation shafts in the central area.

A drilling deviation of about 1% is considered a reasonable tolerance in relation to drill length, hole spacing and conventional drilling equipment. Diameter of the borehole depends on the selected method of drilling.

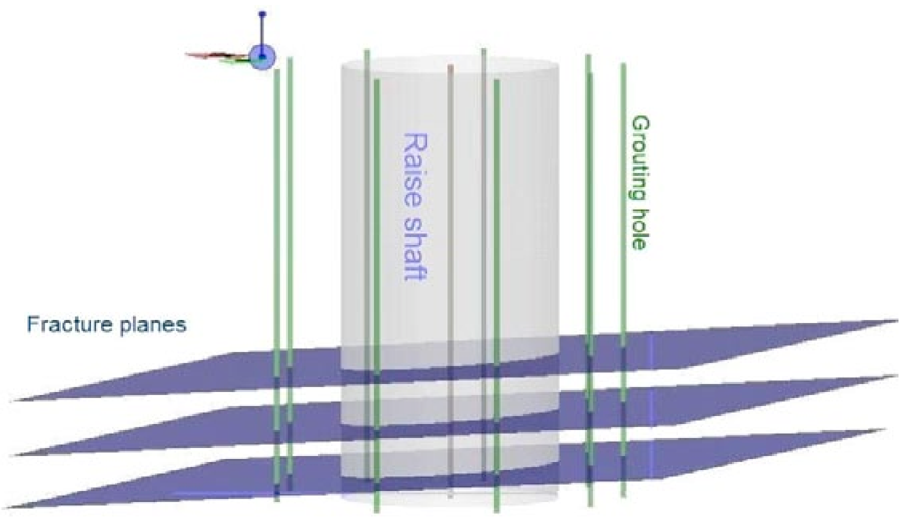


*Figure 5-4. Principle for grouting in boreholes around lift and ventilation shafts.*

### 5.3.3 Central area

The central area consists of a number of different tunnels and rock caverns; see Figure 2-4 in Chapter 2. The unique geometries in the central area compared to other functional areas are the large rock caverns. In the rest of this description the focus is on the rock caverns. Refer to the descriptions of ramp and tunnels in the deposition area concerning the other underground openings.

The size of the rock caverns, from about 95 to 255 $m^2$ cross section, and sequence of excavation, e.g. whole section/divided stope/gallery and bench, influence considerably the geometry of the grouting fan but it should be possible to follow the guidelines of the respective grouting type.

### 5.3.4 Deposition area

***Deposition tunnels***

Design criteria are that thickness of the grouted zone shall be 5 m thick and attain a tightness of $1 \cdot 10^{-10}$ m/s. However, penetration lengths of 5 m out from the grouting hole in fracture apertures smaller than 20 µm are not verified in SKB's fine sealing project.

The appearance of the grouting fan in principle follows that of a conventional fan, with length of holes about 20 m and a hole point from the tunnel contour of about 5 m. The biggest difference is that tunnel face holes are drilled as standard and that grouting holes in the bottom are to be drilled inside the tunnel contour except in deformation zones were holes outside the contour are allowed. How the grouting holes in wall and roof are to be placed in details depends on the detail grouting design and also to the orientation of the fractures, see Figure 5-5.

With holes inside the contour the total number of needed holes is reduced but at the same time the requirement on penetration length is increased (>5 m). Grouting holes inside the contour also reduces the possibility of adapting the direction of the borehole to the fracture orientation. Figure 5-6 shows a performed grouting fan with hole inside tunnel contour.

For deposition tunnels in domain HRD_EW007 a systematic pre-grouting with silica sol will be needed to almost 100%. In domain HRD_C and HRD_W about half of the probe sections, corresponding to a grouting fan, will need grouting with silica sol. However, it will be possible to grout a smaller proportion of the fractures using cement, in the few larger fractures.

For domain HRD_EW007 a systematic probe drilling in locations of possible deposition holes should be done. Grouting of these holes will in some cases be necessary, and criteria for grouting must be established in the detailed design.

The ongoing SKB project "Sealing at great depth" are focus on grouting in deposition tunnels with the purpose to fulfill the requirement on inflow of 1 l/min and 60 m tunnel, i.e. 1.7 l/min and 100 m tunnel. This requirement has been fulfil, with silica sol grouting. A briefly description of the methodology and experience will be done below, more details is present in Appendix A.

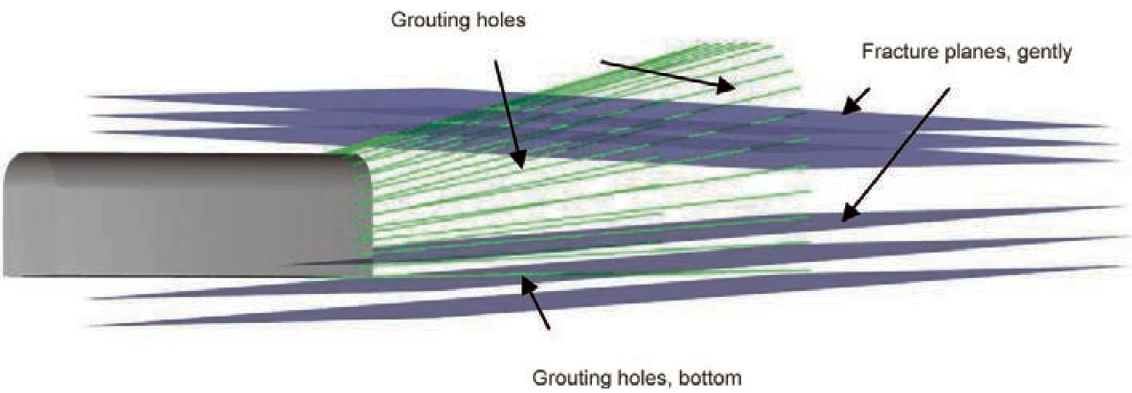


***Figure 5-5.*** *Illustration grouting fan in deposition tunnel and gently fracture planes.*

***Figure 5-6.*** *A performed grouting fan inside tunnel contour, photo from SKB:s sealing project at great depth.*

The ongoing SKB project "Sealing at great depth" is focused on grouting in deposition tunnels with the purpose to fulfil the requirement on inflow of 1 l/min and 60 m tunnel, i.e. 1.7 l/min and 100 m tunnel. This requirement has been fulfilled, with silica sol grouting. A briefly description of the methodology and experience will be done below, more details is present in Appendix A.

A complete grouting fan includes, in addition to at least two rounds of drilling and grouting, an extensive programme with several tests and analyses:

1. Drilling and installation of packers.
2. Different types of hydraulic tests in all holes.
3. Analysis of the results from the hydraulic tests. Based on these results, the execution of the grouting for each individual borehole is decided.
4. Grouting of the first rounds of boreholes, with silica sol or cement grout.
5. Drilling the second rounds of boreholes.
6. Three hydraulic tests in boreholes of the second grouting rounds.
7. Analysis of results with same procedure as in item 3 above.
8. Grouting in all boreholes according to item 4 above.
9. Possible drilling of the third run of boreholes, if inflow in the boreholes is greater than a critical value.
10. Hydraulic tests in the boreholes of the third round.
11. Grouting in all boreholes in the third and final round.

When grouting with silica sol only one hole can be grouted at a time, so-called batch grouting in one hole. This is due to the fact that when the batch is mixed, the ageing starts and the time to the gel induction point has started with no possibility to change the time. This means that one batch of grout is prepared for every single grout hole. However, two or more holes may be grouted at the same time if several batches are prepared and if it is possible to use several pumps.

A design requirement for deposition tunnels is that the sealed zone shall be at least 5 m thick around the tunnel, see Section 4.4. For grouting holes outside the tunnel contour this is achieved when the penetration ($I_{max,\ 2\text{-}D}$) in a critical hydraulic fracture aperture (b), i.e. 20 µm, is 2.5 m /Funehag 2008/, in accordance with Equation 5-2. The distance between tunnel contour and hole bottom is 5 m, see Figure 5-1, and also that there is overlap between grouting fans:

$$I_{\max,2-D} = 0.45 \cdot b \cdot \sqrt{\frac{\Delta p t_G}{6\mu_0}} \qquad \text{5-2}$$

where $t_G$ is the gel induction time, $\mu_0$ is the initial viscosity of silica sol and Δp is the grouting pressure.

The grouting holes at the bottom of the tunnel, which must be located inside the tunnel contour, shall be designed in a corresponding way but for a penetration in the critical fracture aperture at 5 m.

For the grouting holes in the bottom of the tunnel, which must be located inside the tunnel contour, they shall be desugned in a corresponding way but for a penetration in the critical fracture aperture at 5 m.

To illustrate the difference in grouting time that is achieved between grouting holes outside and inside the tunnel contour the Equation 5-2 has been used to calculate the gel induction time. In the calculations all the other parameters were held constant. The grouting time per hole has then been determined according to the principles of Equation 5-2. Table 5-2 presents the results of the calculations.

### Exhaust shafts in the deposition area

The exhaust shafts in the deposition area will be raise-drilled, in the same way as for the lift and ventilation shafts in the central area.

Grouting in these shafts will be carried out before raise drilling begins. The grouting is carried out in vertical boreholes which are drilled in a ring round the shafts, similar as the lift and ventilation shafts through the central area. The grouting procedure is also the same, i.e. drilling and grouting in stages of 25 m. The total length of the vertical grouting holes will be about 500 m, which puts strict demands on drilling equipment and handling of grout. Drill deviation should not be greater than 0.3 to 0.5% for a 500 m deep hole in order to avoid severe spreading of holes at that depth. It may also be necessary to use drilling techniques with smaller deviations, for example directional drilling.

In addition, there are the practical aspects concerning the handling of grout, transport down the hole, filling/applying packers and also the actual grouting which are critical in achieving success when grouting in boreholes deeper than 100 m. A detailed specification of requirements and working plan for each item and equipment details must be compiled and verified by testing.

In general the principles of grouting are the same for these shafts as for the shafts to the central area except that all the work is carried out from the surface and not at 100-metre levels.

**Table 5-2. Calculated gel induction time and grouting time for holes outside and inside the tunnel contour respectively.**

| Location of grouting hole | Design penetration length | Gel induction time | Practical grouting time/hole |
|---|---|---|---|
| Outside contour | 2.5 m | 10 min | 30 min |
| Inside contour | 5.0 m | 40 min | 100 min |

## 5.4 Choice of grouting measures during construction

Different types of investigation holes can be drilled to obtain data for decisions on the choice of grouting type and adjustment of measures in the respective type. In the following text differentiation has been made between pre-investigation holes and probe holes.

Pre-investigation holes refer to holes, which are drilled mainly with the purpose to identify large deformation zones of high transmissivity, but also to give an initial indication of design and the extent of grouting. For successful grouting in a high transmissive deformation zone at great depth, it is important to have preparedness for necessary grouting measures such as drilling program, tests, equipment and grouts. For the grouting programme, one pre-investigation hole shall always be drilled before tunnel excavation, and the hole shall have a length of minimum 100 m or be equal to the deposition tunnel length. Pre-investigation holes are to be drilled inside the tunnel contour. In the hole the following hydraulic investigation should be performed: water loss measurements in sections; continuous hydraulic logging along the hole; and outflow measuring. The results from the pre-investigation holes shall provide information about thickness and transmissivity of discovered deformation zones. The pre-investigation hole should be a core drilled hole, in which mapping of the orientation of fracture sets could be done.

Probe holes are specific holes for grouting, which are drilled ahead of the tunnel face in connection with tunnel excavation and grouting. In the systematic pre-grouting some holes in the grouting fan are used as probe holes. In the probe holes possible fracture or deformation zones are recorded and hydraulic tests, i.e. water loss measurement and outflow measurement, are carried out to determine hydraulic fracture aperture and groundwater pressure.

For selective pre-grouting the extent of probe holes is chosen depending on results from pre-investigation holes, prediction of deformation zones or requirements concerning inflow of water. The necessity and extent of grouting can be determined based on results from probe holes.

A preliminary assessment is that three probe holes, about 20 m long for every 15 meters of tunnel, always are drilled in all underground openings except in deposition tunnels and transport- and main tunnels in hydraulic domain HRD_EW007. Domain HRD_EW007 should always be systematic pre-grouted and, accordingly, the grouting holes will be probe holes before grouting.

The probe hole could after the hydraulic tests be included in the grouting fan as tunnel face holes, see Figure 5-1. If the fracture planes, for example gently dipping fractures (see Figure 5-4), or deformation zones are parallel to the tunnel axis or otherwise special geometrical conditions are encountered, probe holes outside of the contour will be needed.

Prior to probe drilling, a procedure or method is needed to determine whether or not grouting is required. A theory for this was developed in connection with the fine sealing project at Äspö HRL, but has not been tested practical in this project. On the other hand, the method was tested on performed hydraulic tests in probe holes in a follow-up project /Persson et al. 2009/ with good results. The decision methodology is based on results from hydraulic tests in probe holes, from which it should be determined statistically whether grouting is needed or not. For several probe holes this is done by evaluating the probability for the median value from the hydraulic tests being lower than the critical transmissivity, where the critical transmissivity among other items is dependent on the inflow requirement.

All of the individual water-bearing fractures and possibly also deformation zones with low transmissivity are considered not reasonably identifiable by probe drilling or pre-investigation hole. Some fractures will probably be detected in the first case as point leakage after excavation of the tunnel. In the deposition tunnels post-grouting of point-leakage should be anticipated and accordingly planned for.

A summary of a possible process for choosing grouting types and adjustment of grouting measures based on pre-investigation holes and probe holes is presented in Table 5-3.

**Table 5-3. Summary of a possible process for choosing grouting types and adjusting grouting measures.**

| Functional area | Investigation | Decision | Basis for decision |
|---|---|---|---|
| **Accesses** | | | |
| Before excavation of longer tunnel sections | One pre-investigation hole (core drilling) | Prediction on deformation zones. Number of probe holes. Need for special equipment, adjustment of grouting measures (GrT3). | Geological mapping of drill core, hydraulic tests |
| Before grouting | Probe holes in the grouting fan, i.e outside tunnel contour | Need for grouting (Grt1 or GrT2). Adjustment of grouting measures. | Hydraulic tests |
| **Deposition area in HRD_EW007** | | | |
| Before excavation of each tunnel | One pre-investigation hole (core drilling) | Prediction on deformation zones. Need for special equipment in deformation zones (GrT3). | Geological mapping of drill core, hydraulic tests |
| Before grouting | The grouting holes (GrT2) should be probe holes | Adjustment of grouting measures. | Hydraulic tests |
| **Deposition area in HRD_C and HRD_W, central area** | | | |
| Before excavation of each tunnel | One pre-investigation hole (core drilling) | Prediction on deformation zones. Need for special equipment in deformation zones (GrT3). | Geological mapping of drill core, hydraulic tests |
| Before grouting | Probe holes inside the tunnel contour | Need for grouting (GrT1 or GrT2). Adjustment of grouting measures. | Hydraulic tests |

## 5.5 Checks

### 5.5.1 General

SKB is at present engaged in activities to produce programmes for checking of underground excavation works. The aim of these programmes is to specify methods for verification and checking of different aspects related to construction of the final repository. Checking regarding grouting are included in a programme for rock engineering checks, which in turn is part of a programme for technical systems. The checking programmes should include:

- What is to be inspected (checking parameters)?
- How is it to be checked (method)?
- How often should cheching be made (frequency)?
- Conditions for approval (acceptance criteria)?

On the basis of the first item of the observational method concerning acceptable behaviour, the checks with regard to grouting according to /Emmelin et al. 2007/ can be divided into four different parts. The purpose of the checks is partly to assess the status of the ungrouted rock mass, result of grouting and partly to check how the specified requirement on acceptable behaviour in terms of water inflow is fulfilled. The need of checks before grouting, during grouting and after grouting is summarised in Table 5-4.

The following Chapter presents which methods can be used to check different parameters.

### 5.5.2 Checks before grouting

The purpose, with checks in bore holes before grouting, is to estimate the hydraulic characteristics, as transmissivity, groundwater pressure and fracture frequency. These parameters give the distributions of the hydraulic fracture aperture.

The hydraulic characteristics could be determined with different types of hydraulic tests in bore holes, for example inflow test or pressure test (water loss measurement). Further more the pressure test can be made with one or two packers, different packer spacing, number of pressure levels and testing time. These hydraulic tests are separated from the drilling work, i.e. can only be done after drilling of the holes.

**Table 5-4. Actity before, during and after grouting. The table is based on /Emmelin et al. 2007/.**

| When | Check | Requirements | Observation, criteria | Action |
|---|---|---|---|---|
| Before grouting | Is grouting necessary?<br>Water inflow before grouting "ground behaviour". | Critical value for grouting.<br>Inflow values within limits for respective grouting type. | Hydraulic characteristics based on investigation/ probe holes. | Grouting or not.<br>Choice or change of grouting type.<br>Adjust design in respective grouting type.<br>Alternative sealing measures and equipment. |
| During grouting | Grout spread in the rock mass. | Values of pressure, flow, volume, time and properties of the grout. | Pressure, flow, volume, time, properties of the grout. | Adjust design in respective grouting type.<br>Practical quality aspects. |
| After grouting, before rock excavation | Achieved tightness around the tunnel, "system behaviour". | Tightness of grouted zone. | Water inflow or hydraulic characteristics in control holes. | Another grouting round. |
| After grouting, after rock excavation | Water inflow after grouting "system behaviour". | Permitted inflow of water to the different underground openings. | Water inflow in measuring weirs, drop mapping. | Post-grouting. |

The determination of the hydraulic characteristics, at tunnel site, should verify or not the predictions of ungrouted rock mass conditions.

For identification of deformation zones drilling could automatic recording be used. Modern drilling rigs normally feature automatic recording of drill parameters integrated with the rig. These parameters give no detailed information on individual hydraulic fracture characteristics but could facilitate identification of deformation zones.

### 5.5.3 Checks during grouting

Pressure, flow, volume, time and properties of the grout should be recorded continuously during the grouting work. The degree of accuracy, depend on the grouting type is used.

The aim of the continual checks is mainly to test that the intended design parameters are achieved. For grouts that have low pH the mixing procedure is especially critical /Ranta-Korpi et al. 2007/. Proper working of the grouting equipment should be checked continually during the process.

Various test methods for cement-based grouts are described for example in /Eriksson and Stille 2005/. Tests are normally made of the rheological properties (viscosity, yield value), penetrability, curing (gain in strength), change of volume, and specific weight. Figure 5-7 shows a "Mud balance", in which the specific weight could be checked.

Test methods for silica sol are described for example in /Axelsson 2009, Funehag 2008/.

Which tests, both for cement and silica sol based grouts, are to be made depending on which grout properties are determined in the design, for respective grouting type and hydraulic characteristics. Development of test methods are done in research projects at Chalmers (e.g. /Axelsson 2009/) and KTH (e.g. /Draganovic 2005/).

### 5.5.4 Checks after grouting, before rock excavation

Hydraulic tests should be made in control holes, drilled after a first grouting round, to check grouting results. Based on the results of these tests subsequent decisions can be made regarding another grouting round. Hydraulic tests are carried out in the same way as those before grouting.

### 5.5.5 Checks after grouting, after rock excavation

Inflow in different underground openings can be checked by measuring the inflow in measuring weirs (see Figure 5-8), located at suitable places in the underground opening (see for example /Almén and Stenberg 2005, Funehag 2008/). To obtain accurate measurements it is necessary that the

***Figure 5-7.*** *A "Mud balance" to check the specific weight.*

***Figure 5-8.*** *Measuring the inflow in measuring weirs, photo from SKB:s sealing project at great depth /Funehag 2008/.*

amount of process water can be measured or is known, which can be difficult in individual stretches of tunnel. Inflow can also be checked by collecting the inflow in pump sumps and measuring the total pumped out volume per unit of time.

Point leakage can also be checked, e.g. by drop mapping. Drop mapping should be carried out in connection with rock mapping and can subsequently be repeated as required. Furthermore, the location of drop can be determined with the aid of photographs and/or laser scanning.

Considerable uncertainties are generally related to the results of inflow measurements in tunnels, especially together with the hard requirements regarding inflow. This applies especially to results of measurements in the construction stage. A general opinion is that there is a need of development in methods for checking that specified requirements on tightness are fulfilled. Particular emphasis should be put on developing a method for measuring point leakage in deposition tunnels and deposition holes.

## 5.6 Specific of grouting measures for different grouting types, GrT

### 5.6.1 Grouting type 1

#### *General*

Grouting type 1 implies that grouting is either not needed or that only one grouting fan is needed with either a limited number of grouting holes or as a complete grouting fan, according to Figure 5-1. The fact that a limited number of grouting holes should be sufficient is motivated by experience that high transmissivity discrete fractures, which are perpendicular to the tunnel axis, are "easily" grouted. It also presumes that the requirement of inflow is moderate. For grouting type 1 no further grouting, i.e. only one round, is anticipated.

The functional areas and hydraulic rock domains/deformation zones that are anticipated to be handled as GrT1 are:

- Ramp and shafts in central area between the depth of 150 to 500 m.
- Some part of rock caverns in the Central area (–500 m).
- Some part of transport and main tunnels in domain HRD_C and HRD_W.

#### *Fan geometry*

The grouting procedure begins by drilling probe holes. Based on results from probe holes and structure of the rock mass, it is decided if grouting is necessary and if so, which holes are to be drilled and grouted.

In a selective grouting fan, which is base on results from the probe holes, there are greater possibilities to adapt the angles of holes in relation to the water-bearing fractures than in a complete pre-grouting fan, which is drilled more systematic.

From the above way of reasoning, there are different base grouting fans for GrT1 depending on geometry of the individual underground openings.

#### *Grout*

For GrT1 mainly the cement based grout called "injection grout" will be used whereas silica sol will be used as a complement grout. Regarding grout name, see Appendix C.

#### *Execution and equipment*

Hydraulic tests are made in the probe holes to determine fracture transmissivity, aperture for GrT1 and groundwater pressure. Based on the results of these tests decisions are made regarding type of grout, pressure and stop criteria.

For GrT1, the assumption is that no specific checks of sealing results such as control holes are normally necessary. This is motivated by the fact that this grouting type should be used in rock mass with good hydraulic condition and moderate requirement of inflow. But control holes could be used if the results or observations from the grouting work are unexpected in any way.

### 5.6.2 Grouting type 2

#### *General*

Grouting type 2 means that an initial pre-grouting with one or possibly two grouting rounds is carried out. At least one complete grouting fan is drilled and grouted, after which control holes are made to facilitate decision on possible new grouting rounds.

The functional areas and hydraulic rock domains/deformation zones that are anticipated to be grouted with GrT2 are:

- Ramp and shaft: between the depth of 0 to 150 m.
- Some parts of rock cavern in the Central area.
- Some parts of transport and main tunnels in domain HRD_C and HRD_W.
- All types of tunnels in domain HRD_EW007 and deposition tunnels in domain HRD_C and HRD_W.

#### *Fan geometry*

Based on the results from probe holes or prognosis, a complete grouting fan is drilled for GrT2, see Figure 5-1.

The grouting is made mainly at great depth and high pressure gradients. Erosion can occur in the grout at these high gradients /Emmelin et al. 2007/. To reduce the gradients and thus the risk of erosion, the grouting fan should include a number of grouting holes at the tunnel front and also have a clear overlap between two adjacent grouting fans, corresponding to one tunnel diameter.

Geometry of the fan is to be adapted to the part of the facility and if possible to the fracture structure.

#### *Grout*

For GrT2 in the ramp tunnel, rock caverns and main and transport tunnels in domain HRD_C and HRD_W, the main choice is a cement-based grout for the first grouting round. In a possible second grouting round, a silica sol grout may be chosen depending on the result from control holes after the first grouting round.

For all tunnels in domain HRD_EW007 and deposition tunnels in domain HRD_C and HRD_W the main choice is a silica sol grout in all rounds, see Section 4.4. A cement base grout is chosen in “larger” fractures.

For GrT2, mainly the cement based grouts called “injection grout” and “stop grout” will probably be used. The ration between silica sol and the salt in the silica sol grout depends on the detail design.

#### *Execution and equipment*

Hydraulic tests are made in each grouting hole, mainly to enable assessment of the transmissity and hydraulic aperture of the fractures and also the groundwater pressure. This hydraulic data, together with the detail design, yields a foundation to decision on which of the different SKB recipes should be selected. The data also yields a foundation on which grouting pressure should be chosen. Furthermore, identification of other execution aspects, such as batch volume and holes with communication, can be made. This implies that cement-based grouts and silica sol recipes can occur within the same grouting fan, depending on fracture hydraulic aperture. The combination of grouts has been used with good results in grouting trials, see Appendix A.

After grouting, a check of the result should be made using the control holes. Based on the result from the control holes a possible complementary grouting is then carried out in a second grouting round.

The grouting pump shall cope with both low flows and extremely high flows at high total pressure, which current equipment does not manage with one and the same pump. This implies that a pumping system including several pumps with different capacities must be connected. The recording equipment must cope with the extreme measuring intervals that can be anticipated. To cover the extreme values and measuring accuracy possibly two parallel systems can be required.

Through larger deformation zones at repository depth it may be necessary with separate special grouting such as long-hole grouting, which may require special equipment (see description of grouting type 3).

### 5.6.3 Grouting type 3

#### *General*

Grouting type 3 means that grouting is carried out as systematic and extensive pre-grouting in several rounds. Moreover special equipment may be necessary. Supplementary sealing measures such as freezing may also be needed as a complement to grouting.

The GrT3 is focused to underground openings that cross deformation zones of high water pressure and flows.

The grouting measures for GrT3 are similar to the principles as for GrT2, but are anticipated as being more extensive and may require special equipment and rapid-hardening grouts.

It should be possible to seal remaining inflow by post-grouting after openings have passed the deformation zone. A tight lining may be necessary if major inflow remains after the grouting. Accordingly, the underground openings need to be further enlarged (stoped/blasted) compared to the geometry presented in UDP /SKB 2008a/.

#### *Fan geometry*

A first grouting fan is made with double hole spacing, compared to the grouting fan in GrT2, and all tunnel-front holes. A second round of grouting holes is then drilled between the holes of the first fan plus additional tunnel-front holes. Depending on results from control holes it may be necessary to selectively drill a third round of grouting holes. This implies that almost twice as many holes will be made in GrT3 compared to GrT2. In the event of large flows of water in probe holes it may be advantageous to drill and grout one hole at a time.

#### *Grout*

In the first and second grouting round a cement-based grout is selected and in the third grouting round silica sol is chosen if necessary. For GrT3 all of the cement based grouts, “injection grout”, “stop grout” and “plug grout”, according to Appendix C, will probably be used. Silica sol will be used for complementary grouting.

A grout with focus on rapid hardening must also be tested and approved before grouting in GrT3 can begin. This type of grout may be needed in the event of large flows of water. For this purpose grouts based on polyurethane may be necessary.

#### *Execution and equipment*

For GrT3, time must be allowed for detailed analyses of results from pre-investigation holes, probe holes and control holes, and possibly of grouting work done earlier.

In the case of large inflows from bore holes, grouting round 1 is grouted at once without any hydraulic tests. The aim is to inject large amounts of grout, i.e. the design criteria are large amounts of grout and/or long grout injection times. Hydraulic tests are made in each grouting hole before grouting round 2.

After completed grouting in round two, control holes are made in which hydraulic tests are performed to check the tightness achieved. Based on the result from the control holes a possible complementary grouting is then carried out, i.e. a third grouting round. In the third grouting round it is assumed that a cement based grout or silica sol will be used. The grouting pressure, recipe, procedure and equipment for silica sol are the same as for GrT1.

Some special equipment and measures may be necessary in GrT3, especially at great depth. Preparedness should be present for example for drilling extra-long grouting holes /Chang et al. 2005/ and also for grouting with rapid-hardening grout. In the event of anticipated large flows of water, drilling and grouting through Blow-Out-Preventors (BOP) should be considered whereby the flow of water from the boreholes can be controlled /Chang et al. 2005/ (see Figure 5-9). Furthermore, the large flows of water and the pressures in grouting type 3 require substantial pump capacities. In some cases, for GrT3, freezing may be necessary as complement to grouting.

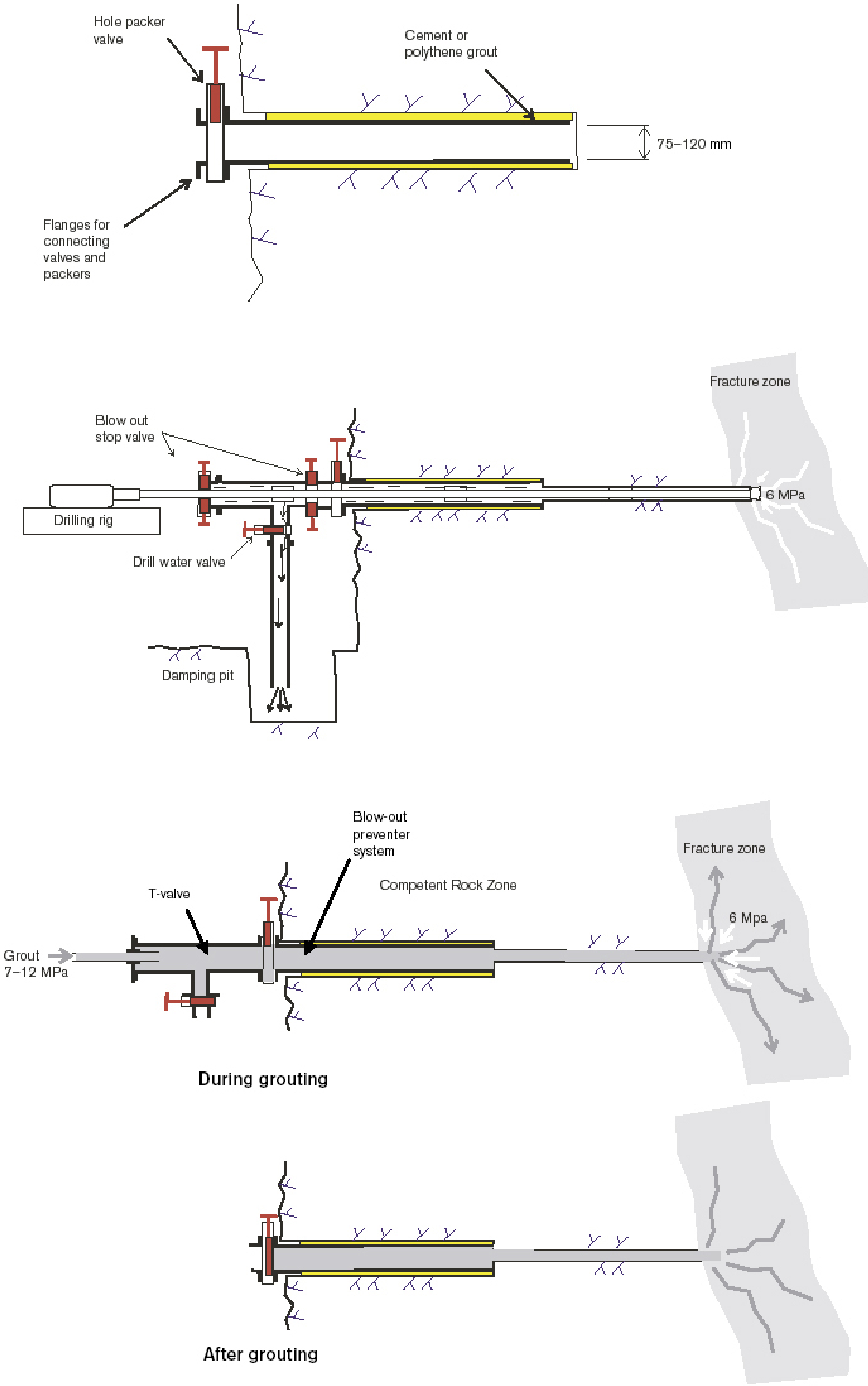


*Figure 5-9. Principles for drilling and grouting when using Blow-Out-Preventors /Chang et al. 2005/. It should be noted that the stated water pressure and grouting pressure relate to the criteria described in /Chang et al. 2005/.*

## 5.7 Curtain grouting

### General

The curtain grouting shall cover the shafts. Curtain grouting is made from the surface before excavation of the shafts, see Figure 5-10.

### Fan geometry

The holes are drilled in a systematic pattern around the shafts. The spacing of holes is then to be halved in one to two additional rounds (split-spacing-technique). The holes are drilled in stages of about 25 m.

### Grout

For the curtain grouting a cement-based grout is selected and tested that is suitable for transport down in deep holes and that has a low pH. An important factor in enabling success when grouting in deep boreholes is finding a grout that is robust in withstanding the effect of dilution, and which hardens in a controlled manner.

***Figure 5-10.** Grouting from surface, before sink shaft excavation /Ahlbrecht 2005/.*

***Execution and equipment***

The principle is that grouting holes in about 25-m stages are drilled, hydraulic tests are made, and then the holes are grouted. Renewed drilling is subsequently made in the grouted holes, with new hydraulic tests to check the result. If the grouting has given a desired effect, the same holes are drilled down in a new stage of about 25 m, otherwise re-grouting is carried out. When the grouting holes are completed down to a depth of 100 m (100 m stage inside the access area) or 500 m (in the deposition area), a new round of grouting holes are done with a "split-spacing" pattern. That means, the new holes are drilled and grouted between the holes of the first round. Additional holes are drilled to make an even closer pattern to enable checking of the grouting result and a possible decision to drill more grouting holes.

Drilling down to 100 m is assumed possible using conventional equipment, because requirements on drilling accuracy are not critical in curtain grouting. Drilling equipment of greater accuracy will be required for holes deeper than 100 m. Drilling may in these cases be executed using core-drilling, down-the-hole technique or special equipment for controlling the position of the drill bit.

Grouting is made by inserting the grout hose to the bottom of the hole and then filling the hole with grout to the upper level of the stage (25 m), where a packer is secured. Grouting of the stage is then started from the surface. For depths greater than 100 m the grout is inserted down the hole through a casing tube and a packer around the tube is extended at the upper level of the actual stage.

The grouting pressure and also time or volume criteria for curtain grouting are determined after the introductory grouting trials.

The execution of grouting and the handling of grout in deep boreholes have been shown to be complex with many components that must work practically without malfunctioning or taking too long time. It is therefore recommended that a detailed specification of requirements and working plan is compiled for the various items with regard to grouting in deep boreholes.

## 5.8 Post-grouting

A certain amount of inflow and some point leakage will always remain after completed pre-grouting. Accordingly, post-grouting will be necessary in some tunnel sections. Systematic post-grouting fans are recommended in preference to pinpoint measures with grouting holes, see Appendix A.

There are no generally established grouting methods/strategies for post-grouting, which is stated in an ongoing research project /Fransson and Gustafson 2006/. A few guidelines for post-grouting are presented below, based on this research project and also /Butron et al. 2008 Granberg and Knutsson 2008/.

Design of post-grouting must be created with regard to penetration of the grout, jacking effect, risk of surface leakage and also the pressure gradient and size/appearance of tunnel contour /Fransson and Gustafson 2006/. The gradient is considered first and foremost to prevent the grout from flowing back to the tunnel or that erosion of the grout occurs before it hardens. The execution and results of the pre-grouting is of considerable significance for design of the post-grouting.

When designing a post-grouting fan it is important to have knowledge of dip and strike direction of the water bearing fractures, based on tunnel mapping. This will facilitate the drilling of grouting holes into the fractures as accurately as possible at large angles. In describing fractures in connection with post-grouting design, consideration should be taken to whether the observed fractures are an effect of the tunnel blasting. Furthermore, probe holes are needed to assess fracture aperture and groundwater pressure before confirming the design.

In a case where the leakage comes from fractures that are not sealed due to geometry of the pre-grouting fan, a different angle should be used than that in the pre-grouting. This implies that the post-grouting fan should generally be orientated towards the tunnel driving and at relatively right angles to the tunnel contour. The reason for this is that fractures that have not been sealed are often fairly gently dipping. Some overlapping between the holes should also be strived for, see Figure 5-11. In this scenario the grouting can be made in two grouting rounds, a first round with a cement-based grout and a second round with silica sol.

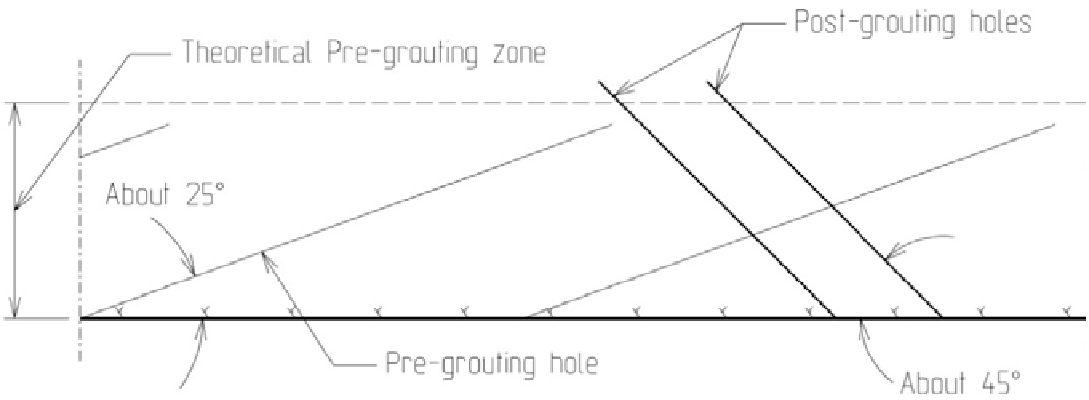


***Figure 5-11.*** *Principles for post-grouting, in the scenario were the pre-grouting fan does not cross the water bearing fractures.*

If fractures are not sealed because the grout does not have sufficient penetrability the fan geometry of the pre-grouting should be retained (unless gently dipping fractures dominate) and a different grout should be selected.

As with pre-grouting, the best penetration result is achieved by high-pressure, but particular attention must be paid in post-grouting to proximity of the tunnel contur. If the pressure used is too high there is a risk that wedges/blocks and possibly bolt reinforcement and shotcrete reinforcement will be damaged.

The earlier guideline has been that grouting holes for post-grouting should not be drilled further out than the sealed area of the pre-grouting. According to this guideline, the grouting has no effect outside this area. On the other hand, if a grout with good penetrability is used, e.g. silica sol, longer post-grouting holes can be used to achieve better sealing results /Granberg and Knutsson 2008/. This is mainly because the pressure gradient often results in considerable surface leakage and a grout such as silica sol will flow out into the tunnel with less effect in the rock. The new recommendation is that the post-grouting should be drilled outside the pre-grouted zone. Outside the pre-grouted zone the gradient is much smaller and the grout can penetrate further and has time to gel/harden and not flow out into the tunnel.

As in the case of pre-grouting, when grouting with silica sol it is essential that the grout is pumped until the accelerated gelling process begins /Granberg and Knutsson 2008/.

# 6 System behaviour – assessment of water ingress after grouting

## 6.1 Introduction

The assessment of inflow after grouting (in UDP /SKB 2008a/ called "system behaviour") is to be made for different functional areas based on calculations using analytical methods and/or experience from earlier grouting. A comparison between assessed inflow after grouting and the requirements that are prescribed with regard to permitted inflow is also to be made.

Calculations of possible inflow after grouting are already presented in Chapter 4.3 in connection with the assessment of the degree of difficulty for the grouting. These are also presented in Chapter 6.3 below, together with a comparison of calculated inflow before pre-grouting. A comparison between calculation results and experience of grouting is presented in Chapter 6.4.

## 6.2 Calculation methods

A description of the calculation methods is presented in Chapter 3.2, input data before grouting is described in Chapter 3.3, and input data concerning the grouted zone is presented in Table 4-1.

## 6.3 Calculation result

The calculation of inflow after grouting, base on cement, is presented in Table 6-1 and inflow after grouting, base on silica sol, is presented in Table 6-2. Table 6-1 and Table 6-2 also presents inflow before grouting calculated according to Chapter 3.

**Table 6-1. Calculated inflow of water for different functional areas before grouting, after cement grouting and also maximum permitted inflow according to UDP /SKB 2008a/.**

| Functional areas/ underground openings | Inflow before grouting per 100 m (litre/min) | Inflow after cement grouting per 100 m (litre/min) | Maximum permitted inflow per 100 m (litre/min) |
|---|---|---|---|
| **Accesses** | | | |
| Ramp, depth 0–500 m | Min.: 2.0<br>Median: 49<br>Max.: 315 | Min: 0.9<br>Median: 6.5<br>Max: 28 | 10 |
| Raise shaft, depth 0–500 m | Min.: 1.6<br>Median: 34<br>Max.: 120 | Min: 1.9<br>Median: 17<br>Max: 31 | 10 |
| Sink shaft, depth 0–500 m | Min.: 1.7<br>Median: 35<br>Max.: 125 | Min: 1.7<br>Median: 5.2<br>Max: 10 | 10 |
| **Central area** | | | |
| Rock caverns | 10-percentile: 2.5<br>Median: 14<br>90-percentile: 55 | 10-percentile: 2.5<br>Median: 11<br>90-percentile: 23 | 10 |
| **Deposition area** | | | |
| Transport/main tunnels, HRD_C | 10-percentile: 2.2<br>Median: 12<br>90-percentile: 49 | 10-percentile: 2.2<br>Median: 8.4<br>90-percentile: 15 | 10 |
| Transport/main tunnels, HRD_W | 10-percentile: 0.1<br>Median: 9.8<br>90-percentile: 245 | 10-percentile: 0.1<br>Median: 7.3<br>90-percentile: 20 | 10 |
| Transport/main tunnels, HRD_EW007 | 10-percentile: 49<br>Median: 75<br>90-percentile: 125 | 10-percentile: 15<br>Median: 17<br>90-percentile: 19 | 10 |
| Deformation zones < 3 km in Transport/Main tunnels | Min.: 4.3 /zone<br>Median: 12 /zone<br>Max.: 105 /zone | Min: 4.0 /zone<br>Median: 9.9 /zone<br>Max: 60 /zone | – |
| Transport tunnel, with deformation zone NE107a, HRD_C | Median: 80 | Median: 47 | 10 |
| Transport tunnel, with deformation zone NE107a, HRD_EW007 | Median: 135 | Median: 55 | 10 |
| Transport tunnel, with deformation zone NS059a, HRD_W | Median: 280 | Median: 86 | 10 |
| Exhaust rasie shaft (0–500 m), HRD_C | Min.: 1.9<br>Median: 35<br>Max.: 125 | Min: 1.9<br>Median: 19<br>Max: 33 | 10 |
| Exhaust raise shaft (0–500 m), HRD_W | Min.: 0.1<br>Median: 44<br>Max.: 165 | Min: 0.1<br>Median: 20<br>Max: 92 | 10 |

**Table 6-2. Calculated inflow of water for deposition areas before grouting, after silica sol grouting and also maximum permitted inflow according to UDP /SKB 2008a/.**

| Deposition area | Inflow before grouting per 100 m (litre/min) | Inflow after silica sol grouting per 100 m (litre/min) | Maximum permitted inflow per 100 m (litre/min) |
|---|---|---|---|
| Deposition tunnels (per tunnel), HRD_C | 10-percentile: 2.1<br>Median: 12<br>90-percentile: 47 | 10-percentile: 1.0<br>Median: 1.5<br>90-percentile: 1.7 | 1.7 |
| Deposition tunnels (per tunnel), HRD_W | 10-percentile: 0.1<br>Median: 9.4<br>90-percentile: 235 | 10-percentile: 0.1<br>Median: 1.5<br>90-percentile: 1.7 | 1.7 |
| Deposition tunnels (per tunnel), HRD_EW007 | 10-percentile: 47<br>Median: 71<br>90-percentile: 120 | 10-percentile: 1.7<br>Median: 1.7<br>90-percentile: 1.7 | 1.7 |
| Deformation zones < 3 km in Deposition tunnels | Min.: 4.0 /zone<br>Median: 11 /zone<br>Max.: 95 /zone | Min: 0.2 /zone<br>Median: 0.3 /zone<br>Max: 0.9 /zone | – |

## 6.4 Comparison between calculation results and experience of grouting

Comparisons between calculations of water inflow after grouting and measured water inflow in other constructed underground facilities are associated with many uncertainties. Differences can exist for example in hydrogeological characteristics and groundwater pressure, geometry of the tunnels, requirements on tightness and also grouting measures. Moreover, there are uncertainties with regard to accuracy of the calculation method.

Experience of grouting carried out, which should be compared to the inflow referred to in Table 6-1 is available from the excavation of ramp, tunnels, shafts and rock caverns for Clab 1 and 2 and also from ramp and tunnels for Äspö HRL /Carlsson and Christiansson 2007/. These functional areas border on the Laxemar area and have similar geological conditions as Laxemar. The rock mass at Clab 1 and 2 (depth about 30 m) are however much tighter than the rock mass in Laxemar at the corresponding depth. Measurements from adjacent cored boreholes indicate conductivity at about $2 \cdot 10^{-10}$ m/s for Clab 1 and 2 /Rhen et al. 2006/, which can be compared to conductivity at about $1 \cdot 10^{-7}$ m/s for Laxemar. For this reason, measurements of inflow, after grouting, from Clab 1 and 2 are not comparable with corresponding prognosis of inflow in Laxemar. Furthermore, there are differences in requirements on inflow between the final repository and Clab 1 and 2 and thus accordingly on the objectives of the grouting.

Limited grouting trials using cement-based grouts have also been carried out at Äspö HRL with the objective of achieving the best possible grouting in connection with tunnel production /Emmelin et al. 2004/. The trials with cement-based grouting were made in two fans at a depth of –450 m. Hydraulic tests in grouting holes prior to grouting indicated conductivity in the range of about $1 \cdot 10^{-7}$ m/s. With an adapted grouting design, especially with regard to the grout, the,sealing effect based on the hydraulic tests in boreholes, was calculated in the range of about 95–97% according to /Emmelin et al. 2004/, i.e. a conductivity in the grouted zone of about $1 \cdot 10^{-9}$ m/s. No direct measurements of inflow to the tunnel, i.e. in measuring weirs, have been made. It should be noted that the grouting design and prognosis were changed according to the stepwise investigations, from cored bore holes and long probe holes to grouting holes.

Furthermore, grouting trials have been carried out using silica sol /Funehag 2008/. However, these grouting trials have been used as references, together with other references, when choosing values of the tightness of the grouted zone, $K_g$.

## 6.5 Conclusions

Based on the calculations made and the comparisons with practical grouting carried out in the Oskarshamn area, the following conclusions can be drawn:

- In unfavourable conditions the inflow of water after grouting can exceed the requirement of maximum permitted inflow to ramp, shafts and transport/main tunnels.
- Especially the difficulty of meeting the requirement for the raise shafts in the deposition area should be considered, since grouting can only be carried out from the surface.
- Based on analytic calculations; it can be concluded that the inflow to deposition tunnels in all hydraulic domains, after grouting with silica sol and a sealed zone of about 5 m outside the contour of the tunnel (see 4.3.1), will meet the requirements. However, the calculated inflow is equal to or just below the requirement. The success to fulfil the requirement is thus an extensive grouting work and in some deposition tunnels the requirement may not be met.

# 7 Compilation of materials and other resources

## 7.1 Introduction

In UDP /SKB 2008a/ it is stated that the amount of grouting refers to different ingredient materials in respective proposed grout together with the number of boreholes. The amounts are to be given by $m^3$ and tonne. Amounts are to be presented for accesses (ramp and shafts), central area and deposition area. In design step D2 it is to be assumed that the grouts provided by SKB can be used.

The assessment of the amounts of grout is presented in Chapter 7.2.

Even other resources, for example equipment, are to be summarised according to UDP /SKB 2008a/ (see Chapter 7.3). These resources and the application are also described in Chapter 5 in conjunction with the description of grouting types.

Lengths of different underground openings in the underground facility through hydraulic rock domains and deformation zones are based on the presented layout /Leander et al. 2009/.

## 7.2 Amounts of grout

In UDP /SKB 2008a/ it is stated that the calculation of the amount is to be based on the assumption that the porosity in the rock mass is filled with grout a certain length outside the tunnel periphery. The porosity is to be based on the hydrogeological properties that are presented in SER /SKB 2008b/ and the grout spread around the tunnel periphery is to be assumed as corresponding to the thickness of the grouted zone.

### 7.2.1 Calculation methods

Calculations of the volumes in the different underground openings have been made using Equation 7-1 (/Eriksson and Stille 2005/). Assessments of amounts based on this equation can according to /Eriksson and Stille 2005/ be adequate in a calculation phase. However, the importance of utilising experience from earlier grouting is emphasised.

To calculate the amount of grout remaining in the rock mass after blasting, the Equation 7-1 is modified according to Equation 7-2.

$$V = n \cdot \pi \cdot (t + r_t)^2 \qquad \text{7-1}$$

$$V = n \cdot \pi \cdot (t^2 + 2 \cdot t \cdot r_t) \qquad \text{7-2}$$

in which

V = injected volume ($m^3/m$)

t = thickness of grouted zone (m)

$r_t$ = tunnel radius (m)

n = porosity (dimensionless)

The porosity, n, can be calculated using different equations, which describe the relationship between the porosity and the hydraulic properties of the rock mass. A common equation is the one according to /Brotzen 1990/ (Equation 7-3). This equation was prescribed and used, for example, in design step D1 at both Laxemar and Forsmark. A conclusion from design step D1 was that the equation resulted in reasonable and comparable amounts of cement based grout (/Janson et al. 2006, Brantberger et al. 2006/) as long as the conductivity value was not too low. An assessment was that the equation gave less reliable results at conductivity values $<10^{-9}$ m/s. With silica sol based grout, the corresponding conductivity value should be lower.

Another relationship between hydraulic conductivity and porosity is also found according to /Emmelin et al. 2007/ in /Dershowitz et al. 2003/. This relationship has however not been used in earlier designs.

Other ways of assessing the porosity is to use information about fracture frequency and hydraulic fracture apertures according to /Snow 1968/. The frequency with regard to water-bearing fractures can normally be determined by hydraulic tests but the appearance of fracture distribution is more uncertain.

For calculation of amounts in design step D2 it is regarded, in brief, that Equation 7-3 according to /Brotzen 1990/ gives sufficient accuracy.

$$\log n = 0.17 \cdot \log K \ - 1.7 \pm 0{,}3 \qquad \text{7-3}$$

in which

n = porosity (dimensionless)

K = conductivity of the rock mass (m/s)

K = T/L (/Eriksson and Stille, 2005/), where L=length (m)

### 7.2.2 Input data and assumptions

Based on the conductivity values that are presented in Chapter 3.3.1, Table 7-1 shows which values that have been used for the porosity according to Equation 7-3. The calculations are only made for the median hydraulic properties. The thickness of the grouted zone, t, is 5 m.

All other input data are presented in Table 7-2.

**Table 7-1. Hydraulic conductivity (rounded to the nearest integral number), and porosity in various hydraulic domains in intervals of depth and zones.**

| Part of rock mass | Hydraulic characteristic, K (m/s) or T (m²/s) | Porosity, n (‰) min/average/max. |
|---|---|---|
| HRD_C (0–150 m) | $K=2 \cdot 10^{-7}$ | 0.6/1.3/2.6 |
| HRD_C (150–400 m) | $K=2 \cdot 10^{-8}$ | 0.5/1.0/2.0 |
| HRD_C (400–500 m) | $T^*_{20,\ median}=1 \cdot 10^{-7}$ | 0.4/0.8/1.5 |
| HRD_W (0–150 m) | $K=3 \cdot 10^{-7}$ | 0.8/1.6/3.1 |
| HRD_W (150–400 m) | $K=3 \cdot 10^{-8}$ | 0.5/1.0/2.1 |
| HRD_W (400–500 m) | $T^*_{20,\ median}=1 \cdot 10^{-7}$ | 0.4/0.8/1.5 |
| HRD_EW007 (400–500 m) | $T^*_{20,\ median}=6 \cdot 10^{-7}$ | 0.5/1.0/2.1 |
| Zones, < 3 km (500 m) | $K_{median}=3 \cdot 10^{-8}$ | 0.5/1.0/2.1 |
| Zone, NS059A (500 m) | $K=2 \cdot 10^{-7}$ | 0.7/1.4/2.9 |
| Zone, NE107A (at different level) | $K_{75\ m}=1 \cdot 10^{-6}$<br>$K_{180\ m}=8 \cdot 10^{-7}$<br>$K_{500\ m}=8 \cdot 10^{-8}$ | 0.6/1.2/2.5<br>0.9/1.8/3.7<br>0.6/1.2/2.5 |
| Zone klx11_dz11 (400 m) | $K=4 \cdot 10^{-8}$ | 0.6/1.1/2.2 |

* Based on Figures "$T_{sum}$ *for 20 m sections*" in /SKB 2008b/. $T^*_{20,\ median}$ is the median value between $T=1 \cdot 10^{-8}$ and maximum T in the Figures.

**Table 7-2. Input data for calculating grouting amounts.**

| | Ramp, deposition tunnels, main and transport tunnels | Rock caverns | Shafts |
|---|---|---|---|
| Total number of holes including re-grouting and tunnel-face holes | 40* pcs (25 to 60 pcs depending on geometry and GrT) | 50 pcs | 25 pcs |
| Hole length | 20 m (shaft in deposition area 500 m, shafts in central area 100 m) | | |
| Overlap | 5 m | | |

* as an average value for the different tunnel geometry (deposition tunnels and main/transport tunnels) and grouting type.

Systematic grouting, i.e. 100% pre grouting, is anticipated at depth 0–150 m for all tunnels and shafts, all tunnels in domain HRD_EW007 and also in all deformation zones that are passed. Selective grouting is anticipated for ramp and shafts between depth 150 to 500 m, rock caverns in the central area and also for all tunnels in domain HRD_C and HRD_W. Selective grouting is anticipated at 50% of the tunnel stretches.

### 7.2.3 Calculation results

Based on calculations made, Table 7-3 presents a summarised assessment of amounts for different functional areas. The amount of grout presented refers to the total amount of grout including grouting of probing holes, tunnel-front grouting and post-grouting. The amount of hole-filling in tight holes is not included in the calculation. The amount of grout that remains in the rock mass after blasting is presented in Table 7-4. The amounts presented have been rounded off to the nearest 10 $m^3$.

**Table 7-3. Summary of total amounts of grout injected before blasting for different functional areas.**

| Functional areas/ underground openings | Drilling, number/drilled metre (no./m) | Volume of grout Min.–max. ($m^3$) | Proportion "plug grout"/ "stop grout"/"injection grout"/silica sol (%)* |
|---|---|---|---|
| **Accesses (0 to –500 m)** | | | |
| Ramp | 8,370/167,360 | 370–1,460 | 10/10/50/30 |
| Shaft (4 shafts) | 540/10,800<br>Curtain grouting: 75/37,500 | 30–125 | 10/20/50/20 |
| **Central area (–500 m)** | | | |
| Rock caverns | 5,100/102,000 | 140–410 | –/10/60/30 |
| **Deposition area (–500 m)** | | | |
| Deposition tunnels, HRD_C (including zones) | 57,730/1154,670 | 1,500–6,250 | –/10/10/80 |
| Deposition tunnels, HRD_W (including zones) | 33,910/678,130 | 850–3,550 | –/10/10/80 |
| Deposition tunnels, HRD_ EW007 (including zones) | 66,330/1326,670 | 2,300–9,200 | –/10/10/80 |
| Main tunnels, HRD_C (including zones) | 5,450/109,070 | 200–850 | –/10/60/30 |
| Main tunnels, HRD_W (including zones) | 3,030/60,530 | 100–450 | –/10/60/30 |
| Main tunnels, HRD_EW007 (including zones) | 5,390/107,730 | 250–1,050 | –/10/10/80 |
| Transport tunnels (including zones < 3 km, NE107A and NS059A) | 12,720/254,370 | 500–2,000 | 10/10/40/40 |
| Exhaust shaft SA01 | Curtain grouting: 25/12,500 | 25–95 | 10/20/70/– |
| Exhaust shaft SA02 (including klx11_dz11) | Curtain grouting: 25/12,500 | 40–155 | 10/20/70/– |

* Definition according to previous page

**Table 7-4. Summary of amounts of grout remaining in the rock mass after blasting for different functional areas.**

| Functional areas/ underground openings | Drilling, number/drilled metre (no./m) | Volume of grout Min.–max. ($m^3$) | Proportion "plug grout"/"stop grout"/"injection grout"/silica sol (%)* |
|---|---|---|---|
| **Accesses (0 to –500 m)** | | | |
| Ramp | 8,370/167,360 | 320–1,255 | 10/10/50/30 |
| Shaft (4 shafts) | 540/10,800 | 25–115 | 10/20/50/20 |
| | Curtain grouting: 75/37,500 | | |
| **Central area (–500 m)** | | | |
| Rock caverns | 5,100/102,000 | 120–350 | –/10/60/30 |
| **Deposition area (–500 m)** | | | |
| Deposition tunnels, HRD_C (including zones) | 57,730/1,154,670 | 1,350–5,550 | –/10/10/80 |
| Deposition tunnels, HRD_W (including zones) | 33,910/678,130 | 750–3,150 | –/10/10/80 |
| Deposition tunnels, HRD_EW007 (including zones) | 66,330/1,326,670 | 2,050–8,200 | –/10/10/80 |
| Main tunnels, HRD_C (including zones) | 5,450/109,070 | 150–700 | –/10/60/30 |
| Main tunnels, HRD_W (including zones) | 3,030/60,530 | 90–350 | –/10/60/30 |
| Main tunnels, HRD_EW007 (including zones) | 5,390/107,730 | 200–850 | –/10/10/80 |
| Transport tunnels (including zones < 3 km, NE107A and NS059A) | 12,720/254,370 | 400–1,700 | 10/10/40/40 |
| Exhaust shaft SA01 | Curtain grouting: 25/12,500 | 20–85 | 10/20/70/– |
| Exhaust shaft SA02 (including klx11_dz11) | Curtain grouting: 25/12,500 | 35–135 | 10/20/70/– |

* Definition according to previous pages.

The proportions of different grouts are assessed based on the following presumptions:

- "Plug grout" is used for grouting of large fractures, which is anticipated in deformation zones and superficial rock. For less permeable rock the amount of grout is judged to be smaller.
- "Stop grout" is anticipated for grouting, e.g. a first grouting round in rock mass of high hydraulic conductivity.
- "Injection grout" is the main cement grout.
- Silica sol is used primarily in deposition tunnels and for complementary grouting in the other grouting types and also for post-grouting.

Based on the assessed proportion of the different grouts, the amount of grout materials included was calculated based on recipes of the individual grouts, see Appendix C.

Table 7-5 present the estimated tunnel lengths with and without grouting, in respective functional area, and also the grout take, based on Table 7-3.

Based on Table 7-4 and recipes in Appendix C the amount of grout materials are estimated for ramp/shaft, central area and deposition area.

**Table 7-5. Estimated lengths with and without grouting and grout take.**

| Functional areas/ underground openings | Tunnel/shaft length (m) | Grout take ($m^3$/m tunnel/shaft) |
|---|---|---|
| **Accesses** | | |
| Ramp, 0–150 m, grouting | 1,500 | 0.15–0.50 |
| Ramp, 150–400 m, 50% grouting | 2,500 | 0.10–0.35 |
| Ramp, 400–500 m, 50% grouting | 1,000 | 0.08–0.30 |
| Shafts, 0–150 m, grouting | 150 | 0.10–0.35 |
| Shafts, 150–400 m, 50% grouting | 250 | 0.03–0.15 |
| Shafts, 400–500 m, 50% grouting | 100 | 0.03–0.10 |
| **Deposition area** | | |
| Deposition tunnels, grouting | 59,300 | 0.08–0.35 |
| Deposition tunnels, no grouting | 30,000 | – |
| Transport tunnels, grouting | 4,800 | 0.10–0.45 |
| Transport tunnels, no grouting | 1,950 | – |
| Main tunnels, grouting | 5,200 | 0.08–0.3 |
| Main tunnels, no grouting | 2,700 | – |

**Table 7-6. Estimated quantities of grout materials and drilling that remain in the rock mass after excavation of the different underground openings.**

| Element | Material | Ramp/Shafts [ton] | | Central area [ton] | | Deposition area [ton] | |
|---|---|---|---|---|---|---|---|
| | | min | max | min | max | min | max |
| Cement grouting | Water | 120 | 470 | 40 | 120 | 620 | 2,500 |
| | Portland [1)] | 90 | 360 | 30 | 85 | 540 | 2,200 |
| | Silica Fume [2)] | 120 | 490 | 40 | 120 | 740 | 3,000 |
| | Super Plasticiser [3)] | 6 | 25 | 2 | 6 | 40 | 150 |
| Solution grout | Silica | 110 | 420 | 40 | 110 | 4,000 | 16,000 |
| | NaCl solution | 20 | 80 | 8 | 20 | 790 | 3,200 |
| Volume of grout [$m^3$] | | 350 | 1,400 | 120 | 350 | 5,000 | 20,500 |
| Drilling | Number of holes | 8,900 pcs | | 5,100 pcs | | 180,000 pcs | |
| | Drilling meter | 220 km | | 100 km | | 3,700 km | |

1) Sulphate resistant Ordinary Portland cement with $d_{95}$ on 16 µm, type Ultrafin 16 or equivalent, see Appendix C.

2) Dispersed silica fume, microsilica with $d_{90}$=1 µm type GroutAid or equivalent. The density is to be between 1,350–1,410 kg/$m^3$ and 50% ±2% of the solution is to consist of solid particles, see Appendix C.

3) Super plasticiser, naphthalene-sulphonate based, density about 120 kg/$m^3$, type SIKA Melcrete, see Appendix C.

### 7.2.4 Comparison between calculated amounts and experience of grouting

To verify the calculated amounts of grout comparisons have been made between estimated amounts and used amounts in underground projects already constructed. However, the comparisons involve considerable uncertainties. Differences can exist for example in the hydrogeological characteristics and groundwater pressure, geometry of the tunnels, requirements on tightness and also on grouting measures.

Experience from grouting, which could be compared with the calculated amounts in Table 7-3 is available, for example from the construction of Clab 2 and Äspö HRL /Carlsson and Christiansson 2007/.

The rock excavation started in the winter 1999 with Clab 2 and was completed by the end of 2000. Systematic pre-grouting was made in the rock cavern, and selective pre-grouting was made in transport tunnels. Grouting of the rock cavern was made in three stages; the initial stage in conjunction with blasting of the gallery, afterwards in connection with blasting of the intermediate bench and lastly in connection with the bottom bench. A total of 19.6 $m^3$ cement-based grout was injected in 20-metre long boreholes, according to /Carlsson and Christiansson, 2007/. The rock cavern is about

115 m long and has a cross section of 560 m$^2$. In Clab 2 about 0.17 m$^3$/m rock cavern was grouted and for the largest rock caverns in Laxemar a corresponding mean value of about 0.35 m$^3$/m was obtained. The difference between the result for Clab 2 and prognosis for rock caverns in the central area is about a factor of two, corresponding to porosity differences between 0.4 ($2 \cdot 10^{-10}$ m/s, see Section 6.4) for Clab and 0.8 ($5 \cdot 10^{-9}$ m/s, see Section 3.3) for Laxemar.

The access tunnel to Äspö HRL is about 3,600 m long and the rock excavation was started at the end of 1990 and was completed in the summer of 1995, according to /Carlsson and Christiansson 2007/. Grouting was made as selective pre-grouting using cement-based grout in about 20 long boreholes.

Two large deformation zones, NE-3 and NE-1, were passed with extensive grouting in Äspö HRL. In deformation zone NE-3 the consumption was about 1.6 m$^3$/m and in zone NE-1 about 3.8 m$^3$/m. Outside the two larger deformation zones and between tunnel section 0 to 1,400 m the consumption was about 0.24 m$^3$/m grouted tunnel /Stille et al. 1993/. For tunnel section 1,340 to 2,565 the corresponding consumption was 0.35 m$^3$/m /Stille et al. 1994/.

These results from Äspö HRL have been compared with the corresponding prognosis values in the ramp and deformation zone NS107A in the ramp. For the ramp outside deformation zone NS107A a mean value of about 0.26 m$^3$/m was obtained, for section 0 to 1,500 m, and about 0.20 m$^3$/m, for section 1,500 to 4,000 m. For deformation zone NS107A a mean value of about 0.38 m$^3$/m was obtained.

For the upper tunnel sections, that is 0 to 1,400/1,500 m, a good comparison is obtained between Äspö HRL and Laxemar whereas for the lower tunnel sections, that is 1,400/1,500 to 2,665/4,000 m, some difference is obtained. The difference in the lower tunnel sections is probably because the tunnel section in Äspö HRL contains a number of smaller deformation zones, see /Carlsson and Christiansson 2007/, which are not separated in the grouting report /Stille et al. 1994/. Considerable difference is obtained in comparison between the zones, which is mainly explained by the fact that the hydraulic characteristics of the zones have greater differences in values, where NE-1 has a mean conductivity of about $2 \cdot 10^{-5}$ m/s compared to about $1 \cdot 10^{-6}$ m/s for NS107A.

In the cement-based grouting trial made in Äspö HRL during tunnel production in the spring–summer of 2003, about 0.13 m$^3$/m tunnel was used in the grouted tunnel, according to data in /Emmelin et al. 2004/. This can be compared with the prognosis of about 0.18 m$^3$/m, for deposition tunnels in domain HRD_EW007 that has a hydraulic conductivity in the same range as the rock mass in the grouting trial.

### 7.2.5 Conclusions

From Table 7-3 it can be concluded that extensive grouting, using large amounts of grout can be anticipated in large parts of Laxemar's underground openings and especially in domain HRD_EW007.

The difference between estimated maximum and minimum amounts is however considerable. This reflects the uncertainty in conditions that will be met in tunnel excavation and grouting. This implies that test grouting should be carried out in a preliminary phase and an updating of grouting measures and estimations of amounts should be done as the tunnel excavation and grouting progresses.

Moreover, it should be observed that the calculation methods that have been used include considerable uncertainty. For example, the relationship between hydraulic conductivity and porosity is much discussed. It is also difficult to assess, if at all possible, how much of the porosity around the rock mass is filled with grout and also the actual grout spread.

In brief, the opinion is that the estimated amounts of grout are in the correct range of magnitude in comparison with groutings performed earlier.

## 7.3 Equipment summary

Grouting in the final repository is anticipated in a variety of conditions and with different requirements on tightness. Grouting will for example be carried out at great depth and at potentially high water pressure, a number of water-bearing fracture zones will probably be passed, grouting must be carried out from the surface down to several hundred metres depth and relatively unproven grouts will be used. These different grouting scenarios impose requirements on adapted equipment and skill in its use. Procurement of these resources must be made in good time before the construction starts since access to them can be limited.

In Chapter 5.6 the need of different equipment is described to a varying degree in connection with the description of grouting measures. The following list has been compiled of the special equipment that is anticipated for the grouting work.

On the basis of the grouting design work, the need of equipment required is listed below.

- Drilling equipment for the drilling of boreholes from the surfaced down to 500 m depth (maximum borehole deviation 0.5%).
- Grouting equipment adapted for grouting in deep boreholes (e.g. packers, casing tubes, device for pressing the grouting to the bottom of the hole).
- Grouting equipment adapted for grouts based on silica sol and for more than one hole grouting.
- Grouting equipments for cement-based grouts with low pH, which gave uniform properties of the grout independent of invaluable equipment.
- Grouting pumps for both low flows and extremely high flows at high-pressure.
- Mixing equipment of high capacity.
- Recording equipment for both low flows and extremely high flows.
- Equipment that enables venting of grouting holes.
- Equipment for rapid-hardening grout.
- Equipment for measuring/confirmation of tightness conditions in the rock mass at about $1 \cdot 10^{-10}$ m/s.
- Packers, hoses and connections for high pressure.

# 8 Overall judgement of feasibility and uncertainty

## 8.1 General

In the purpose of the report (see Chapter 1.2) it is expressed that the design with regard to grouting shall:

- Show that technique is available which, in anticipated conditions at the relevant site, can satisfy stipulated requirements.
- Estimate the amounts of grout and other resources that are needed.

Earlier in the report, these two items have been discussed with regard to feasibility and uncertainties. On the basis of assessments concerning feasibility and uncertainties a risk list has been compiled in parallel with the grouting design work. The risk list constitutes a basis for the technical risk assessment, which is made in a separate design activity and is presented in a separate report according to UDP /SKB 2008a/.

The uncertainties that have appeared during the grouting design work, and which are linked to the two items listed above, are summarised as follows:

- Fulfilment of tightness requirement and whether the requirement is to be interpreted per functional area or a random 100 m length of an individual underground opening.
- Method for evaluating the need of grouting.
- Pre-grouting with silica sol in deposition tunnels.
- Grouting in location for deposition holes.
- Grouting in deep boreholes, that is deeper than 100 m.
- SKB's grouts (cement-based grouts of low pH and silica sol).
- Robustness of control measures for decision between grouting types.
- Preparedness for unexpected events.
- Preparedness for alternative sealing measures – lining and freezing.
- Grouting measures and events that require special skills/equipment.
- Post-grouting, sealing of point leakage.
- Predicition of inflow.
- Equipment for Blow-Out-Preventors (BOP).
- Quantity of grout (Amounts of grout).

The above items are divided into two groups, those that are more linked to grouting measures and those that are linked to calculations.

## 8.2 Grouting measures

In assessing plausibility with regard to the proposed grouting measures it should be observed according to Chapter 1.3 that:

- The grouting measures are to be realistic in relation to current know-how and experience.
- The grouting measures are to be robust in relation to anticipated variations in characteristics of the rock mass.
- A process for handling prevailing uncertainties should be presented.

As a concluded in Chapter 4.2, cement based grouting is defined as "existing/proven technique" and silica sol based grouting defined as "new/unproven technique".

Based on the analyses made in Chapter 4, it is not considered realistic to carry out grouting in large parts of the underground facility with what is known as proven and well-known technique to fulfil prescribed requirements on the inflow of water. This applies in particular to deposition tunnels for which the requirements on inflow are strict, and for all tunnels in hydraulic domain HRD_EW007, where conductivity of the rock mass is relatively high and also in the drilled shafts. In ramp, central area, sink shafts, and also main and transport tunnels in domain HRD_C and HRD_W it is judged possible to carry out grouting using proven and well-known technique to meet the requirements on inflow.

SKB has taken part in developing the grouting method based on silica sol in order to cope with the strict requirements on inflow in deposition tunnels. Based on analyses made in Chapter 4 it is concluded that the requirements can be fulfilled if using this grouting method. This relatively unproven method has also been tested to a limited degree in conditions similar to Laxemar and results demonstrate that the requirements on inflow can be fulfilled. However, grouting of fine fractures is time consuming and there are several practical aspects related to silica sol grouting that remain to be investigated and verified. Also, there is a need for development of equipment when grouting with silica sol (see Chapter 5.2.4).

It should be noted that systematic grouting will be needed to a large extent in deposition tunnels, especially in the hydraulic domain HRD_EW007 (see Chapter 4.3.4). However, systematic grouting should, if possible, be avoided according to UDP /SKB 2008a/. Thus, from a grouting point of view other hydraulic domains than HRD_EW007 should if possible be chosen for deposition.

With regard to the aim of robust measures, cement-based grouting, which is regarded as a proven technique, can be used for some of the grouting work. The three cement-based grouts that are provided by SKB are assessed as adequate for the different conditions that can be anticipated. In the compilation of grouting measures a large amount of experience from earlier conducted grouting has been studied to support the choice of grouting measures. This has also been done to verify that the grouting measures are realistic and feasible.

With regard to grouting in deposition tunnels it will be done mainly using grout based on silica sol, which is a material that is less proven compared to cement-based grout (see Chapter 4.2). However, it should be noted that, even many similarities with conventional cement based grouts, the low pH-grouts that are intended for the cement based grouting also are relatively unproven. The main issue regarding the low PH-grouts in that the quality of mixing is of great importance (see Chapter 5.2.4).

Silica sol grouting is also considered for use in complementary pre-grouting and post-grouting.

Thus, it is concluded that grouting measures are available for different grouting scenarios.

A process for handling prevailing uncertainties has been described. The process includes principles for choosing and adjusting grouting measures together with checks and possible measures for different stages during construction (see Chapter 5.4 and 5.5).

Aspects concerning feasibility and uncertainties are described below in more detail with regard to the grouting measures.

- Fulfilment of tightness requirement and whether the requirement is to be interpreted per functional area or a random 100 m length of an individual underground opening.

Concerning the shafts grouting will probably not fulfil the requirements on inflow. Alternatives to grouting as a sealing method must thus be considered. A further alternative for the drilled shafts could be to change excavation technique to the shaft sinking method in which grouting can be carried out as pre-grouting at the face of the shaft.

In order to verify the fulfilment of the tightness requirements different checks are to be made. One of these checks includes hydraulic tests in control holes. However, there is uncertainty in the measuring accuracy for normal measuring methods at $1 \cdot 10^{-10}$ m/s. However, special measuring equipment is available according to /Funehag 2008/.

The definition of the inflow requirement, that is whether the requirement applies to an entire functional area or for a random 100 metre stretch, affects the probability of fulfilling the requirement and the degree of difficulty for grouting. Due to the heterogeneity of the rock mass differences there will most likely be differences in the grouting result along the different tunnels.

- Robustness of control measures for evaluating the need of grouting and decision between grouting types.

As concluded in Chapter 4.3.4 the extent of grouting may be significant in deposition tunnels. However, the outcome of the grouting (sealing result and resources) might be very different depending on the decided level for “high enough certainty” when choosing between grouting types. In order to minimise the use of systematic grouting a robust method for choosing between grouting types must be used.

One uncertainty in this is judging a reasonable number of investigation and probing holes and also by which methods water-bearing deformation zones and fractures of various transmissivity values is identifiable.

The total number of probing holes has been assessed preliminarily to at least three to five and that the deformation zones are identified mainly by drill cores from the investigation holes and hydraulic tests in all the holes. The exact number of holes and what methods are to be used must be verified in the introductory grouting and possibly adjusted depending on results and site experience.

Various statistical methods may also be used to derive the optimum number of holes and to evaluate the need of grouting. The systematic control of grouting by such a statistical approach is yet not known. Accordingly further tests are needed concerning this method. Instead of use of statistical method is to make systematic grouting in all tunnels, especially in the deposition tunnels.

- Grouting with silica sol in deposition tunnels.

To cope with the requirements on inflow in deposition tunnels it is necessary, as noted above, that grouting is carried out using a relatively new and unproven method based on silica sol (see /Funehag 2008/). This grouting method has been tested at great depth with favourable results and in similar conditions to those at Laxemar, however, at present it is not considered to be a method for large scale grouting. Especially in the hydraulic domain HRD_EW007, where probably systematic pre-grouting will be required in all tunnels. Furthermore, there are several practical aspects that remain, such as handling of mixtures remaining after the grouting to be investigated and handled..

To fulfil the requirement on tightness it is also necessary that a 5 m thick grouted zone round the tunnel periphery is achieved (see Chapter 4.3.1). Because grouting holes must be drilled in the bottom inside the tunnel contour, the grouting must be carried out so that penetration length is 5 m radially from the borehole. Compared to grouting carried out at Äspö HRL the grouting must be done with longer gel induction time and/or higher pressure. A limitation of pressure must however exist with regard to capacity of the equipment and stability of the tunnel front. Consequently, the grouting in deposition tunnels will be time consuming and the attainable result is uncertain, especially in the tunnel bottom.

- Grouting in location for deposition hole.

Drilled deposition holes shall not be sealed according to the design premises, UDP /SKB 2008a/. However, probe holes should be drilled in the locations of possible deposition holes in hydraulic domain HRD_EW007. Based on analyses presented in Chapter 4.3.5, grouting may be needed in order to fulfil the requirements on inflow to deposition tunnels and/or to enable the location of a deposition hole. The extent of this grouting depends, for example, on the range of fractures that become sealed in connection with grouting round the deposition tunnel. Grouting of the deposition holes can be made through vertical holes that are drilled inside the theoretical contour of the deposition holes. This grouting will in principle be similar to post-grouting, which involves special difficulties with regard to pressure gradients, for example. Consequently, it is difficult to predict the result of grouting in the location for deposition holes.

- Grouting in deep boreholes, that is deeper than 100 m.

Grouting in long bore holes, deeper than 100 m, is to be carried out in the shafts in the deposition area (see Chapter 5.3.2). The difficulty and uncertainties involved with grouting in deep boreholes concerns mainly practical problems such as transporting the grout down, how it is to be injected, and also when boring up can begin. Test drillings and grouting trials should also be carried out.

If possible silica sol grouting in long vertical boreholes should be developed in order to achieve a lower hydraulic conductivity in the grouted zone around the shaft.

- SKB's grouts (cement-based grouts and silica sol).

The different grouts that SKB provided for design step D2, that is three cement-based grouts of low pH and one silica sol based, are all relatively unproven (see Chapter 4.2 and 5.2.4). Grouting using cement-based grouts has been carried out a long time both in Sweden and abroad. Grouting with low pH-grouts is however not commonly practiced even though there are many similarities with conventional cement based grouts. Grouting is at present in progress in Finland using these grouts, where following up is carried out by Posiva.

With regard to silica sol the uncertainties refer to execution and results at great depth, how a rational procedure can be performed and its long-term durability, >5 years.

- Preparedness for unexpected events.

It is generally difficult to know what unexpected events can be anticipated during the construction stage. Moreover, individual interpretations of what is an unexpected event are very varied. An unexpected event during the construction stage can for example be flooding in sink shaft, large consumption of grout, hardening of grout in equipment and unexpected leakage paths. In preparation for the grouting work an initial preparedness plan, including decision process and organisation, should be drawn up for unexpected events.

Preparedness should be available, for example, for more time-consuming and extensive grouting, several grouting rounds and special equipment, e.g. BOP (Blow-Out-Preventor). Various measures and criteria for these should be formulated according to the principles for the observational method.

- Preparedness for alternative sealing measures – lining and freezing.

If the inflow of water cannot be accepted, a technical solution should be available to build a tight lining in the most water-bearing sections. This measure implies considerable cost and delays and also requires specific technical skills. Furthermore, the linings can have different appearances and be made in different ways depending on whether ramp or shafts are involved. A lining implies new questions, such as criteria for construction of a lining, extent of lining, technical aspects, geometries, tunnel excavation, costs and time for construction. It is therefore recommended that a separate survey is made concerning this issue.

- Grouting measures and events that require special equipment/know-how.

Some grouting measures require special equipment and know-how. Examples of such equipment are equipment for drilling of long bore holes from the surface, various types of packers, equipment for grouting in deep boreholes and also pump for varying pressure and flows. The access to this type of equipment is probably limited in the Swedish market. Furthermore, particular skills are needed for large-scale grouting based on silica sol (see Chapter 4.2 and 5.4.2). An important part of planning for the construction stage is therefore to allow time for planning, procurement and training regarding various special equipment that are considered necessary.

- Post-grouting, sealing of point leakage.

Post-grouting is a measure that is necessary if the requirements are not fulfilled by pre-grouting. As concluded in Chapter 5.8, it is well known that succeeding with post-grouting is difficult and that the work demands both planning and thorough execution and also time. There are no established and reliable strategies for post-grouting. There is great need for development and several development projects are in progress both within and outside the SKB organisation.

## 8.3 Calculations

- Prediction of inflow

Predictions of inflow water include many sources of error (see Chapter 3). The calculation models that have been used in design step D2 are well known and accepted but they imply considerable simplification of reality. Furthermore, there may be uncertainties in input data from SER /SKB 2008b/, such as interpretations of measurement results, sampling tests within a wide range and calculation models.

The strategy should be to make more predictions that are based on different calculation models and then make a total appraisal of the different predictions together with engineering assessments. Before the construction stage begins a program for measuring the inflow should be compiled in which the prediction is verified and updated in steps with new knowledge about details.

- Quantity of grout

Calculations of grouting amounts are also based on substantial simplifications (see Chapter 7). Furthermore, input data is based on the hydrological properties which also include many simplifications and uncertainties. Better calculation models exist (see for example /Eriksson and Stille 2005, Funehag 2007, Stille and Andersson 2008/), but these require knowledge of details and also that analysis of grouting is made on site. Despite an increased knowledge of details with the more refined calculation methods, even here there is a need of adjustment of models based on results from grouting.

# 9 Continued design

According to UDP /SKB 2008a/ design step D2 is the last design step in connection with the site investigations. After the site investigation stage a detailed design will follow according to /Emmelin et al. 2007/.

The strategy for detailed design should be a step-wise design, which is also evident in /Emmelin et al. 2007/. No detail solutions should be confirmed before more experience is available from grouting trials and from actual grouting in earlier excavated areas. The following work procedure is recommended:

1. Update the site conditions if more detailed investigations have been carried out.
2. Detailed planning, strategy, execution including documentation and also analysis of test grouting from the surface.
3. Design and implementation of grouting in ramp and shafts, based on experience and analyses of trial grouting.
4. With experience from a large part of the ramp, the grouting of the central area and the various underground openings in the deposition area is to be planned and implemented.

With this work procedure, more detailed criteria can be successively confirmed regarding grouting measures, such as criteria for selection of grouting type, adjustment of measures in a grouting type, when a second grouting round is to be made, and so on.

Parallel with the continued design, enquiries should also be made as to the need of development concerning available grouts and equipment, strategies to optimize and estimate grout take, methods for post-grouting and also the extent and implementation of a possible lining or freezing.

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# Appendix A

## Experience of grouting

Engineering assessments must be used in configuring the grouting measures since theoretical associations cannot fully explain the relationship between characteristics of the rock mass and the result of grouting. These assessments are based to a large extent on experience from performed grouting. The following sections present experience from several different types of grouting that are expected to be of interest in the construction of the underground facility at Laxemar. Experience and principles of grouting with silica sol are presented especially, since this type of grouting is to be used in "minor fractures" according to UDP, /SKB 2008a/.

### Experience of grouting in tunnels and rock caverns in general

In order to make an assessment of need of grouting and relevant grouting measures, the corresponding tightness of the grouted zone may be expressed in terms of hydraulic conductivity. In configuring the grouting measures in more detail, the characteristics of individual fractures are of more importance, but in a larger scale it is sufficient to use the hydraulic conductivity in the analyses.

The following section presents experience from some projects with focus on the tightness that it is deemed possible to achieve by grouting in fractured, hard rock.

The tightness that can be achieved in terms of hydraulic conductivity in the grouted zone is not fully clear. Based on experience from grouting in hard fractured rock the assessment is normally that the lowest hydraulic conductivity, that can normally be achieved by cement grouting is in the range of $10^{-8}$ m/s.

The National Swedish Road Administration directions /Vägverket 1993/ state a limit of $0.5\cdot10^{-7}$ m/s for normal grouting with cement-based grouts. Using other grouts than those that were available at the time the directions were established can possibly achieve greater tightness due to better penetrability of the grout.

Grouting trials, using cement grout under production conditions, in the Stockholm Södra Länken tunnels, demonstrated that grouting could be made at a tightness level corresponding to a hydraulic conductivity in the grouted zone of about $2\cdot10^{-8}$ m/s regardless of the type of cement /Dalmalm et al. 2000/. However, in water-loss measurements with regard to production the appraisal is that the lowest water loss that can be measured corresponds to a conductivity of about $1\cdot10^{-8}$ m/s /Dalmalm et al. 2000/. Thus it is possible that a better sealing result have been obtained.

Lower conductivity values, about $2\cdot10^{-9}$–$3\cdot10^{-10}$ m/s, have however been reported from project "APSE Grouting", which was carried out at Äspö HRL /Emmelin et al. 2004/. It should be observed that the experience described above is from grouting in more homogeneous rock with few fractures. Accordingly, these experience values should be used with caution.

### Experience of grouting at great depth

#### General

The experiences in the present section focus on grouting in water-bearing zones at high pressure, which can be anticipated mainly in passing deformations zones at repository depth. Grouting at great depth in general is also referred to.

The possibility of grouting and the sealing result depends on interaction between properties of the rock mass, grout and execution. In a similar manner the accuracy of drilling is influenced by the drilling method and characteristics of the rock mass.

Using special equipment (gyro and controlled drilling) the drilling of a vertical borehole can achieve a drill deviation of only 0.0025% of the length of the hole /Bäckblom et al. 2004/. Without special equipment a typical deviation is about 2%. Different drilling techniques are suggested to achieve straight holes, depending on the drill supplier. The drill supplier Atlas Copco recommends down-the-hole-drilling in which energy is transmitted direct at the bottom of the hole. The supplier Wassara recommends the use of guided drill tubes in combination with water drilling technique which minimises wear on guide ribs of the tubes, thus providing better guiding of the drill tubes.

Experience from drilling and grouting in water-bearing zones at greater depth is available from several projects both in Sweden and abroad. In those cases where documentation is available it is not fully comprehensive and not always totally clear, which makes it difficult to come to extensive conclusions with regard to suitable grouting measures. Comparisons with other projects must also be made with some caution since the geological and hydrogeological condition, tightness requirements and also grouting measures are often different. Furthermore, some experience is 10–20 years old and considerable technical development can have been made. One should also be aware that know-how from failed grouting work is not probably presented.

A brief description is given in the following section of experience from some identified grouting projects at great depth.

The description is divided into three main groups:

Grouting of water-bearing zones in tunnels:

- Sub-horizontal fracture zone and Singö deformation zone in the construction of SFR (Slutförvar För Reaktoravfall), i.e. final repository for reactor waste, at Forsmark, Sweden /Carlsson et al. 1987, Carlsson and Christiansson 2007a/
- Vertical deformation zones in the construction of Äspö HRL, Sweden /Stille et al. 1993, 1994, Chang et al. 2005, Carlsson and Christiansson 2007b/
- "Case histories" from different countries presented in /Chang et al. 2005/

Grouting in sink shaft:

- Transport shaft at Sedrun, Switzerland /Rehbock-Sander and Meier 2000/
- Transport shaft at Konradsberg, Germany /Ahlbrecht 2005/

Grouting in deep boreholes from surface level:

- Investigation drilling, sealing around casing in borehole KFM01A at about 100 m depth, Forsmark, Sweden /Claesson and Nilsson 2004/
- Grouting of shaft, Garpenberg, Sweden
- Grouting of shaft, LKAB , Sweden
- Grouting of 80 m deep holes around shaft, Dounreay, Scotland
- Mines in China, grouting and freezing /Chunlai and Zongmin 2005/
- Mines in South Africa, grouting in deep boreholes /Heinz 1988, 1993, Kipo et al. 1984/ and /Dierz 1982/
- Curtain grouting down to 130 m, between existing gas storage and excavation of new rock cavern, Sweden
- SKB investigation of grouting in deep boreholes
- Vertical shaft Äspö HRL, Sweden /Bäckblom et al. 2004/

Grouting with silica sol:

- General description of silica sol and grouting trials from Hallandsås and the Törnskog tunnel, Sweden /Funehag 2007/
- Grouting trials from the Törnskog tunnel, Sweden /Ellison 2007/
- Grouting trials from the Öxnered tunnel and the Nygård tunnel, Sweden /Edrud and Svensson 2007, Granberg and Knutsson 2008, Butron et al. 2008/
- Grouting trials from TASS-tunneln in Äspö HRL, at depth –450 m /Funehag 2008/

### Grouting of water-bearing zones in tunnels

The tunnels will pass deformations zones at repository depth. These zones must be sealed by pre-grouting before the zones can be passed. The following section presents some experience of grouting in water-bearing zones at high water pressure.

**SFR, Sweden:**

Most of the grouting work in the construction of SFR was carried out according to /Christiansson and Carlsson 2007a/ in connection with the passage of a gently dipping fracture zone and also a larger steeply dipping deformation zone, called the Singö zone. The grouting work is described in detail in /Carlsson et al. 1987/ and also in brief in /Carlsson and Christiansson 2007a/.

The grouting work in connection with the gently dipping water-bearing zone, designated H2, was carried out in the lower building tunnel at SFR. The depth below surface level in this area was about 150 m. The average hydraulic conductivity was about $1–2\cdot10^{-6}$ m/s (/Carlsson et al. 1987/).

The Singö zone was passed by two tunnels at about 55 m depth. The passages through this zone were slightly more than 100 m long in the respective tunnel, but the grouting work was also carried out in connection to the zone. Conductivity values of $2\cdot10^{-8}–1\cdot10^{-6}$ m/s are stated in /Carlsson et al. 1987/ for parts of the zone.

When sealing these zones a conventional technique with cement-based grouting was used. A brief summary of execution, result and conclusions is given below.

The grouting was carried out in principle as follows:

1. One or several probing holes were drilled and the inflow of water and rock quality was noted.
2. 10–30 grouting holes (10–20 m long) were drilled around the tunnel periphery (single or double grouting fans). Double grouting fans refer to a shorter overlapping length between the grouting fans.
3. Grouting was made using a grout based on grouting cement (to begin with even rapid-hardening cement), water cement ratio 1–3, and with a final pressure of 10–20 bar.
4. Complementary grouting was made if necessary. No data about possible control holes has been found in the references.

The result of the grouting according to /Carlsson et al. 1987/ was that conductivity in the water-bearing zone H2 was reduced by about one power, from an average conductivity of $1–2\cdot10^{-6}$ m/s to about $2\cdot10^{-7}$ m/s. The grouting in connection with the Singö zone showed that the inflow of water fell by about 70% after the grouting.

/Carlsson et al. 1987/ even presented the conclusion that probe drilling in the tunnel excavation is the most important success factor for the passage of water-bearing fracture zones. If the grouting is made too close to the water-bearing zone, or if the zone has already been penetrated, it will be very difficult to carry out the grouting.

**Äspö HRL, Sweden:**

Several water-bearing zones were passed when driving the access tunnel to Äspö HRL /Carlsson och Christiansson 2007b/. One of the most water-bearing zones, NE1, was passed at about 200 m depth. This zone consisted of severely fractured and crushed rock that was more or less transformed into clay. The grouting was carried out according to a conventional procedure of grouting fans that were injected using a cement-based grout (water-cement ratio was mostly around 1.0). Even other types of grouts were tested but failed due to being flushed away. Experience from this grouting is, for example, that the drilling work was difficult due to the high water pressure and severe flowing of water, making it necessary to use sealing tubes with valves in the opening of the borehole. With regard to the grouting work the conclusions were, for example, that a rapid hardening cement grout was favourable (calcium chloride was used preferably as accelerator) and that the limit for the maximum volume of grout that is allowed to be pumped into a grouting hole should not be too small. The grouting work took a long time, a large number of grouting fans were made, a large amount of grout was pumped into the rock and a relatively large amount of remaining inflow of water resulted. The tunnel excavation could however be completed without major problems. Geological and hydrogeological criteria and experience from grouting at Äspö HRL are presented for example by /Rhén and Stanfors 1993, Stille et al. 1993, 1994, Markström and Erlström 1996, Chang et al. 2005/.

/Stille et al. 1993/ also presents inflow of water to the tunnel measured after completion of the grouting. A large amount (55%) of the measured inflow of water is judged by /Stille et al. 1993/, to come from the two larger fracture zones NE1 and NE3, which were grouted at about 200 m depth. Based on the inflow of water presented in /Stille et al. 1993/ and an assumed zone width of 10 m, an inflow of about 35 l/min, m has been calculated. The conductivity of the zones before grouting is assumed to have been in the range of $10^{-4}$ m/s, which in turn has been assessed from the transmissivity values presented in /Markström and Erlström 1996/. On the basis of these criteria a calculation has been made of conductivity in the grouted zone, to which the calculated inflow through the zones corresponds. The calculation has been made according to Chapter 4.2 and resulted in conductivity in the grouted zone of about $5 \cdot 10^{-7}$ m/s.

Furthermore, there is a connection in /Stille et al. 1994/ and /Hermansson 1995/ between fan geometry, orientation of discrete water-bearing fractures and the orientation of the major principal stress. When the ramp crossed the water-bearing fractures at perpendicular angles, large amounts of grout were applied in a few grouting holes with good sealing results. When the ramp met the water-bearing fractures obliquely to parallel no large amounts were injected and less favourable sealing was achieved. The water-bearing fracture zones were almost parallel to the major principal stress.

**Other tunnels, i.e. "Case histories" from /Chang et al. 2005/:**

In the report /Chang et al. 2005/ a number of "case histories" are summarised concerning problems and measures in driving tunnels through water-bearing fracture zones at greater depth. In the following section an account is given of the summary of these "case histories".

Both the problems and grouting measures in the different projects are mainly site specific. Important success factors are generally considered to be investigation drilling to determine location, orientation and properties of the fracture zones and that the work is carefully planned before the tunnel is excavated through the zone. Furthermore, pump capacity must be available in the case of large inflows of water.

In most projects grouting has been carried out to enable tunnel excavation through weak zones. Problems with drilling have been dealt with in some projects by grouting in levels through steel tubes. However, in two projects, the Oslo fjords tunnel (Norway) and the Jonkershoek tunnel (South Africa), grouting was not an adequate measure due to poor rock conditions and high water pressure. The result was that freezing had to be used to enable tunnel excavation to continue in these tunnel sections. In the Oslo fjord tunnel, where the water pressure was up to 1.2 MPa, the sealing effect was judged to be uncertain above all in the more earth-like conditions of a weak zone. On the other hand, the sealing effect was judged to be favourable in the part of the zone that consisted of crushed rock. In some parts of the tunnels in Jonkershoek, the rock cover was over 1,000 m and a number of zones with a variable degree of poor rock conditions were passed. One of these zones could be sealed by grouting while freezing was carried out in another zone. In some projects grouting has even been combined with freezing.

In /Chang et al. 2005/ the need for so called Blow-Out-Preventors is pointed out. Blow-Out-Preventors can be used to facilitate and increase reliability when drilling and grouting at high water pressure and high flows of water. When using Blow-Out-Preventors the water flow from the boreholes can be controlled.

### Grouting in sink shaft

The skip shaft will be constructed according to UDP /SKB 2008a/ by shaft sinking, i.e. gradual rock excavation from above by drilling and blasting. Sink shafts have been constructed in a number of projects around the world. Grouting is normally carried out in these shafts in connection with the shaft sinking (i.e. cover grouting) but grouting can also be made from the surface. The following section presents some experience of grouting in shafts when shaft sinking.

**Sedrun, Switzerland:**

The shaft in Sedrun is an 800 m long vertical transport shaft that was constructed in connection with the Gotthard Base Tunnel in Switzerland. In this shaft, 40 m long drill holes were injected with cement grout at up to 12.0 MPa injection pressure. After grouting, the total inflow of water into the shaft was less than 30 l/min (i.e. approximately 4 l/min, 100 m).

**Konradsberg, Germany:**

Another example of a completed sink shaft is the Konradsberg shaft. The shaft is 240 m deep and has a diameter of 6 m. The shaft passed several strongly water-bearing gently dipping deformation zones. 35 metre long grouting holes were drilled in a ring around the shaft in stages as the shaft sinking progressed. After scaling of the shaft however, a watertight concrete lining was applied to the most water-bearing and fractured section.

### Grouting in deep boreholes

Lift and ventilation shafts will be built according to UDP /SKB 2008a/ using the raise-drilling technique. Sealing around these shafts can be made from the surface and/or in stages from niches in the ramp. The following section presents some experience of grouting in deep boreholes.

**Investigation holes Forsmark, Sweden:**

In the investigation boreholes that have been drilled within the proposed Forsmark area, sealing has been performed by means of grout injection between the outer casing and the wall of the borehole. The grout injection was performed in two different ways, either by a packer at the bottom of the borehole or with a hose inserted in the underground opening between the wall of the borehole and the casing. The cement grout was injected by gravity or by applied pressure. The main purpose of the grouting was to seal the underground opening between the wall of the borehole and the casing but at the same time sealing was also made of the fractures that were penetrated by the borehole. The water-cement ratio for the grout was about 0.5 and the final pressure 0.5–2.0 MPa /Claesson and Nilsson 2004/.

In the grouting of KFM01A, with a consumption of 2,500 kg, a strongly water-bearing zone was also injected which was passed at about 40–50 m depth (the casing was at about 100 m depth). The inflow of water through this zone was about 800 l/min. Based on the stated dimensions of the borehole and casing tube, and if the loss of grout out from the borehole can be neglected, the amount of grout injected in the water-bearing zone was about 500 kg cement.

**Garpenberg, Sweden:**

Grouting was carried out from the surface before the drilling of a ventilation shaft, diameter 4.5 m and depth about 300 m, at the Boliden mine in Garpenberg. The shaft passed a number of water-bearing fracture zones. The rock mass between the fracture zones was of good quality. Boreholes were drilled and grouted from the surface in stages of about 2×120 m long using core drilling equipment. After grout injection of the first stage re-drilling was made and the second stage was drilled and grouted. Due to instability in sections of the hole, preparedness was available to stabilise the boreholes with cement slurry. The boreholes were located in a ring about 0.5 m outside the wall of the shaft. Drilling and grouting was made in two rounds, a first round with a drill spacing of about 4.8 m and a second round where the holes were located between those of the first round.

In each stage the drilling, water-loss measurement and grouting were carried out before the next borehole was started. Boreholes of the second round were also used to facilitate investigation of grouting in the first round. The grout injection was made at low pressure using a stable cement grout, water cement ratio 0.7, which was thickened somewhat after a definite time if final pressure had not been reached. The final pressure was set at 1.5 MPa overpressure. To prevent cementing of the packers they were released after 45 minutes injection work and moved up slightly. The grouting result was judged successful and adequate tightness was achieved in the rock mass around the shaft. Both drilling and grouting were carried out without serious practical problems.

**LKAB, Sweden:**

Before the drilling of a mine shaft, grouting was carried out from the surface in about 150 m long percussion drilled holes. The quality of the rock was generally poor. The grouting was carried out from the bottom up using a cement based grout. Both drilling and grouting was carried out without any practical problems.

After completed grouting, pilot holes were drilled without preceding control holes. When the pilot hole was ready it was found that more water than anticipated had leaked into the borehole. Control holes were drilled from below to enable location of the leakage. Cement grout ran out from these holes while drilling. High content of sulphate in the water was judged to be the reason why the grout had not hardened.

**Dounreay, Scotland:**

Grouting has been carried out in Scotland around a 65 m deep shaft belonging to the Dounreay Nuclear Power Establishment. The grouting was done to reduce the inflow of water into the shaft in connection with radioactive waste being moved from the shaft. The grouting was carried out in about 80 m deep boreholes in two grouting rounds. The first round comprised an inner ring, which was grouted at low pressure (Blocker injection) and the second round comprised an outer ring was grouted at a higher pressure. The purpose of the inner ring was to create a screen between the outer ring and the existing shaft. The grout in both of the rings consisted of cement, water, plasticizer and silica slurry. The holes were drilled using core drilling equipment and the grouting was made in stages from the top down. Hydraulic tests showed that a reduction of conductivity in the fracture zones was achieved by up to a power of three (from about $10^{-5}$ m/s to $10^{-7}$–$10^{-8}$ m/s).

**Mine shafts, South Africa and China:**

Published experience with regard to grouting in deep boreholes is also available from the mining industry. In South Africa and China, for example, grouting in deep boreholes around shafts has been carried out since the 1950s. In these shafts, which have been constructed by shaft sinking, grouting has been done in boreholes down to a depth of more than 1,000 m. In these groutings the grout has been based on cement-bentonite, cement-bentonite-fly ash or micro cement and silica. Both grouts with high and low water cement ratio and grouting from the top down and from the bottom up, respectively, have been practised. Grouting from the surface is commonly recommended even when grouting is to be made in connection with shaft sinking. In this way a more reliable and faster shaft sinking is achieved since grouting in the shaft can be reduced compared to if no grouting had been made from the surface. Grouting from the surface has been described as successful, although it is not made clear what requirements on tightness applied. /Heinz 1993/ points out some aspects that must be observed when grouting in deep boreholes.

- At greater depths the temperature of the rock mass can be higher than at the surface, resulting in faster hydration of the cement.
- Packing of cement grains occurs at high pressure which can result in elastic deformation in the surrounding rock mass.
- If water is forced out from the grout an incomplete hydration can occur before re-drilling with the risk of hydration during drilling when water is added.

**Curtain grouting between gas storage rock caverns, Sweden:**

Curtain grouting was carried out close to one existing gas storage. The purpose of the grouting was to prevent leakage from the gas storage, which was in operation, to the adjacent planned rock cavern, especially during the period for the rock works. Grouting holes were drilled down to about 130 m after which injection of cement based grout was made in 20-metre stages, without water-loss measuring, from the bottom up. The drilling was carried out using down-the-hole drilling technique at a diameter of 115 mm. The curtain grouting was done using the split-spacing method, i.e. drilling and grouting of holes between the previous holes. The first grouting round was made with a spacing of 16 m between the holes, which was halved in two further rounds down to 4 m when the grouting was considered adequate. The assessment of sealing result was based on comparison between results of the different grouting rounds, i.e. no water loss measurements, or similar, were carried out. No leakage from the existing gas storage was detected during rock work on the new rock cavern.

**SKB investigation of grouting of deep boreholes:**

The investigation was initiated because of SKB's negative experience of performed grouting or cementing of deep investigation holes at greater depth. The study is presented in an internal SKB report. A number of factors were identified as possible reasons for the negative experience. The factors that were identified as possible were; malfunctioning of the grout on the way down (sedimentation and possible mixing with borehole water), malfunctioning in the pressing out phase (dilution), the effect of high pressure and also the influence of salt intermixture at great depth. The various factors were studied mainly by tests in the laboratory.

Of the factors that were considered to have the greatest influence were the effect of dilution and its relevance to the hardening phase of the injection grout. The conclusion was that the grout should be applied to the rock mass as quickly as possible to minimise malfunctioning of the grout during its bonding phase. The execution of grout injection and the handling of grout in deep boreholes were shown to be complex with many components that must work practically without taking too long. It was therefore recommended that a detailed requirements specification and working plan should be compiled for the various items with regard to grouting in deep boreholes.

Among the other factors even the content of salt could have some effect on the grouting result, while the effect of pressure on the grout was not shown to have any great significance.

**Shaft Äspö HRL, Sweden:**

The about 400 m deep vertical shafts in Äspö HRL were built using the raise-drilling technique. Grouting was made in three rounds at depths of about 100 or 200 m. The first grouting round, from the surface, was about 200 m deep and was pre-grouted through the pilot hole. The two following rounds were each about 100 m deep and were pre-grouted through core boreholes that were drilled around the envisaged shafts /Bäckblom et al. 2004/. The pre-grouting in the boreholes was made in stages from the bottom up, using cement based grouts (water cement ratio 1 and 2). The consumption of grout in the rock mass was marginal and some of the water leakage remained in the completed shafts.

## Grouting with silica sol

### General

Grouting with silica sol in Sweden is relatively new and has been used as a grout in rock grouting since 2002. The grout has been used abroad mainly for earth reinforcement, an application where the grout is more well-tried.

The grout silica sol is a colloid solution containing extremely fine silicate particles of silicon dioxide, $SiO_2$, suspended in water (see Figure A-1). Colloids are defined as a mixture of non-soluble particles bigger than molecules but sufficiently small to remain suspended in a fluid, without sedimentation.

The silica sol that is used in grouting has a particle size between 3 and 100 nanometre (i.e. one thousandth the size of a grain of cement). The silica sol is delivered as a fluid in which the concentration of silicate is about 40 percent by weight. An accelerator in the form of a salt solution is used to enable the grout to gel and finally to harden, e.g. NaCl or $CaCl_2$. The amount of accelerator in the fluid influences the gel time of the silica sol.

Silica sol has a viscosity but no yield value. It is thus similar to a Newtonian fluid. The penetration decreases markedly when the initial viscosity has doubled and the penetration ceases shortly /Funehag 2007/. This point is the gel induction time and is one third of the gel time. This relationship between penetration, gel induction time and gel time is tested and verified by /Funehag 2007/.

For silica sol to give fully satisfactory sealing results, the injection front must be in contact with water because silica sol shrinks in dry conditions /Funehag 2007/. The durability of silica sol is not fully verified. Ongoing analyses regarding the durability of silica sol after gelling show however that the chemical structure is stable, which indicates good durability.

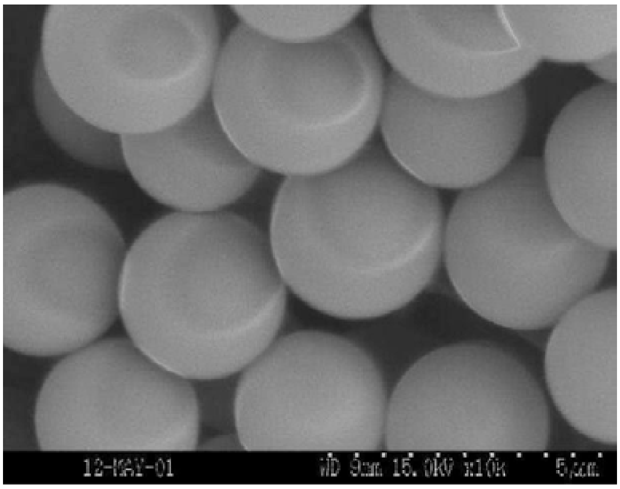


***Figure A-1.*** *Silicate particles suspended in a fluid /Edrud and Svensson 2007/.*

### Experience of grouting using silica sol

A number of different grouting trials using silica sol have been carried out in Sweden. These trials can be divided into pre-grouting trials (see /Funehag 2007, Ellison 2007, Butron et al. 2008/) and post-grouting trials (see /Funehag 2007, Edrud and Svensson 2007, Granberg and Knutsson 2008/). These trials have been made as limited grouting in major tunnel projects and in shallow depth, i.e. down to –50 m. Furthermore, a project is at present in progress under the auspices of SKB /Funehag 2008/, in which silica sol and its grouting technique will be tested and developed according to SKB's criteria (greater depth, i.e. –450 m) and requirements.

The grouting measures in the different trials have been based on the necessary penetration of silica sol, which among other things is dependent on the gel time. This implies that a mixture with a specific gel time is made for each grouting hole or for a couple of holes with about same conditions. Furthermore, dosing of the accelerator was made by hand to achieve exactly the correct mixing ratio. On reaching the pre set grout injection time, the grout injection into the hole was stopped and the remaining mixture in the equipment was emptied (from mixer to hose connection) and the equipment was cleaned before starting on the next grouting hole. The above described mode of work implies that a lot of material and time were used in the process. Grouting using silica sol also required more resources than for cement grouting. The principle was that one person was responsible for mixing and checking gel times, another was responsible for the grout injection, including checking of flow of grout and grouting pressure, while a third person was stationed at the tunnel face to deal with hoses, fittings and cleaning.

The two pre-grouting trials, which were carried out in the Törnskog tunnel (road tunnel) and the Nygård tunnel (railway tunnel), were made in limited stretches in connection with conventional tunnel excavation and grouting in superficial conditions (rock cover 20 to 50 m).

#### Törnskog road tunnel, pre-grouting

The grouting trials in the Törnskog tunnel were carried out in two different steps. The first step was more research inclined /Funehag 2007/ with adapted grouting fan and pressure. The subsequent step was more production inclined /Ellison 2007/ and based to a large extent on the original grouting design (fan and pressure) and combined with cement grouting. A total of about 400 m tunnel was grouted with silica sol. The inflow requirement of 2 l/min, 100 m in combination with the site criteria indicated theoretically that cracks down to a width of 0.014 mm needed to be grouted. Based on this crack width a separate grouting programme was made with a complete grouting fan /Funehag 2007/. Normal equipment and personnel were used but before the trials everyone was subject to training. The results show that the inflow requirement was met and that the residual inflow in the trial section was less than in other parts of the tunnel /Funehag 2007/.

## Nygård railway tunnel, pre-grouting

In the Nygård tunnel a total of about 100 m was grouted with silica sol. From the prescribed inflow requirement, 5 l/min, 100 m, and the site criteria it was judged that the requirement could be met by conventional cement grouting. The grouting trials were therefore focused on sealing the tunnel roof with silica sol. The normal grouting fan, i.e. bottom holes and wall holes, were grouted with cement and the roof holes with silica sol. The result demonstrates good tightness with a reduced amount of residual inflow compared to other parts of the tunnel /Butron et al. 2008/. It should be noted that individual grouting fans were mainly dry before grouting started, i.e. no loss of water occurred in probing holes.

## SKB's sealing project at great depth

For SKB's ongoing sealing project at great depth /Funehag 2008/ an approximately 100 m long tunnel at 450 m depth is to be constructed at the SKB rock laboratory Äspö HRL on the island of Äspö. The purpose of the project is to demonstrate that it is possible to fulfil SKB's requirement on ingress of 1 l/min and 60 metre tunnel (i.e. 5 l/min and 300 metre tunnel), at great depth.

Execution and some results are presented in the report /Funehag 2008/ from five grouting fans. The five grouting fans have been drilled and grouted in three rounds. Three of the fans were made with boreholes outside the tunnel contour, and one of the fans penetrates a zone with high flow of water. The other two fans were drilled inside the tunnel contour and in relatively dense rock. The grouting fans also contained so-called tunnel-face holes that are placed straight ahead in the tunnel face.

Figure A-2 below shows grouting fan 2, including holes outside tunnel contour and through a deformation zone.

Figure A-3 below shows grouting fan 5 inside tunnel contour and in relatively dense rock.

Both cement-based grouts with low pH and silica sol grout, with a composition according to Appendix C, have been used in the project. However cement-based grouts have been used to a relatively small extent. The ingress requirement implies that fractures with a hydraulic width down to 10 µm should be sealed.

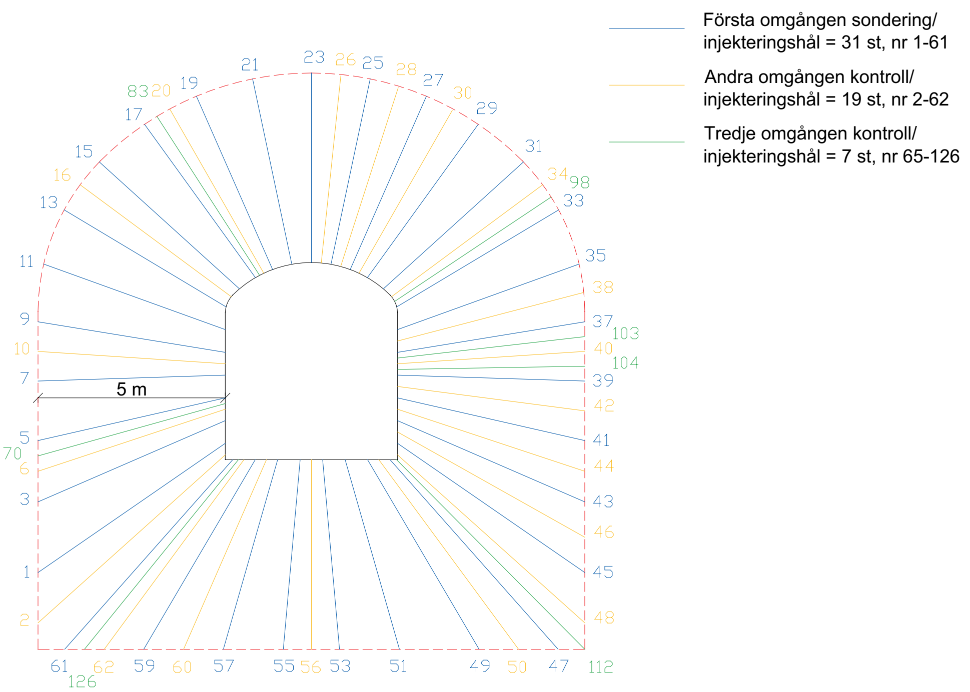


***Figure A-2.** Borehole layout for fan 2 from /Funehag 2008/, not including 3 tunnel-face holes. Blue: first rounds of hole (nos 1–61); red: second rounds of holes (nos 2–62) and green: third rounds of holes (nos 65–126).*

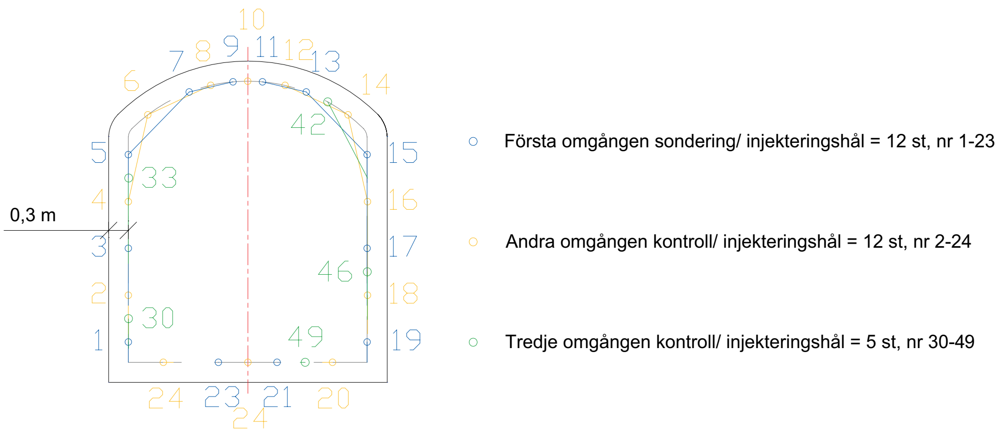


***Figure A-3.*** *Borehole layout for fan 5, from /Funehag 2008/, not including 4 tunnel-face holes. Blue: first rounds of hole (nos 1–23); red: second rounds of holes (nos 2–24) and green: third rounds of holes (nos 30–49).*

The choice of grout in a grouting hole has been made according to the following principle:

- Hydraulic fracture aperture <130 µm, silica sol with long gel time (about 40 to 90 min) and a grouting time per hole of about 35 to 75 minutes.
- Hydraulic fracture aperture between 130 and 150 µm, silica sol with shorter gel time (about 20 to 45 min) and a grouting time per hole of about 20 to 75 minutes.
- Hydraulic fracture aperture >150 µm, low-pH cement with a grouting time per grouting hole of about 45 minutes.

A complete grouting fan including, in addition to three rounds of drilling and grouting, an extensive programme with several tests and analyses followed the general grouting cycle, see /Funehag 2008/:

1. Drilling and installation of packers
2. Hydraulic tests are to be made as full hole testing, in all holes:
   a. Groundwater pressure tests
   b. Measurement of inflow from bore holes
   c. Water injection tests (water loss tests)
3. Analysis of the results from the hydraulic tests is to be made. Execution of the grouting is to be decided for each individual borehole.
4. Grouting of the first round of boreholes, group A. The silica sol is allowed to harden for at least 1 hour after grouting and cement grout for at least 6 hours.
5. Drilling the second rounds of boreholes, group B. Number and location is based on preset criteria.
6. Hydraulic tests are to be carried out in borehole group B; same tests as in item 2.
7. Analysis of results is to be done according to item 3.
8. Grouting is to be carried out in all boreholes according to item 4.
9. Possible drilling of a third round of boreholes, group C, if ingress in the boreholes of group B is greater than 0.1 l/min per hole.
10. Hydraulic tests are to be carried out in the boreholes in group C; same tests as in item 2.
11. Grouting is to be carried out in all boreholes in group C.
12. Reporting and quality control of data.

When carrying out grouting only one hole can be grouted with silica sol grout at a time, so-called batch grouting is done. Using the principle of single-hole grouting and the extensive test programme according to the above, it has taken about 140 to 170 hours to complete one grouting fan.

The result of the grouting is checked partly by hydraulic tests in inspection holes, before, between and also after the grouting, and partly on tests in measuring weirs.

Table A-1 presents calculated median conductivity before and after grouting of fan 1, 2, 3, 4 and 5. The calculated conductivities are based on the results from inspection holes in the respective fan /Funehag 2008/.

It can be noted in Table A-1 that the applied grouting concept, that is grouting with silica sol and with complementary cement grouting, achieves about the same median conductivity after grouting regardless of whether a water-bearing zone or a relatively dense rock mass is grouted.

The report /Funehag 2008/ also presents measurements of the flows in the measuring weirs. The measured flows in the measuring weirs are below the maximum permitted flows, implying that the requirement regarding inflow has been fulfilled.

### Hallandsås, Öxnered and Nygård, post-grouting

The post-grouting trials in Hallandsås, the Öxnered tunnel and the Nygård tunnel, have also been carried out in connection with the completing of railway tunnels. The conditions between the three projects have been very varied, involving everything from geology to execution. For Hallandsås a post-grouting was made at the tunnel face in an older pre-grouting fan. This is not like a normal post-injection situation with surrounding pressure gradients and possible flow paths. The results of these trials showed that the rock could be sealed by an additional factor 10 compared to tightness achieved in the previous pre-grouting. Continual problems arose in the post-grouting trials in the Öxnered tunnel in controlling the surrounding surface leakages of grout in the tunnel despite the long post-grouting holes that were drilled with the aim of reaching beyond the pre-injection fan. A reduced surface leakage could be stated after the trials but no reduction of total inflow to the tunnel could be measured. Even in trials in the Nygård tunnel there were problems with surface leakage of grout in the tunnel, but this was reduced when the grouting holes were made longer with the aim of creating a "cape" around the existing pre-grouted zone. The total inflow was reduced by about 80% after post-grouting with silica sol.

**Table A-1. Calculated median conductivities from presented results in /Funehag 2008/.**

| Grouting fan | Median conductivity, before grouting (m/s) | Median conductivity, after grouting (m/s) | Notes |
|---|---|---|---|
| 1 | $2 \cdot 10^{-9}$ | $2 \cdot 10^{-11}$ | Rock with less water |
| 2 | $20 \cdot 10^{-9}$ | $2 \cdot 10^{-11}$ | Water-bearing zone |
| 3 | $0.2 \cdot 10^{-9}$ | $0.6 \cdot 10^{-11}$ | Rock with less water |
| 4 | $0.02 \cdot 10^{-9}$ | $0.4 \cdot 10^{-11}$ | "Dense" rock |
| 5 | $0.02 \cdot 10^{-9}$ | $0.2 \cdot 10^{-11}$ | "Dense" rock |

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## Appendix B

# Input data for calculating inflow of water

Tables B1–B4 presents the input data, from /SKB 2008b/, that has been used concerning hydraulic characteristics, K or T, depth below surface level (water pressure), H, and also radius, $r_t$ or $r_s$, for different functional areas, underground openings and parts of the rock mass. The data are rounded to the nearest integral number.

**Table B1. Input data for calculating the inflow to functional area "accesses".**

| Underground opening/part of rock mass/depth | H (m) | T (m²/s) or K (m/s) | r ($r_t$, $r_s$) (m) |
|---|---|---|---|
| **Ramp (depth 0–500 m)** | | | |
| HRD_C (0–150 m) | 75 | $K=2 \cdot 10^{-7}$ | $r_t$:3.0 |
| HRD_C (150–400 m) | 275 | $K=2 \cdot 10^{-8}$ | $r_t$:3.0 |
| HRD_C (400–500 m) | 450 | $K_{10\text{-}perc}=9 \cdot 10^{-10}$<br>$K_{median}=5 \cdot 10^{-9}$<br>$K_{90\text{-}perc}=2 \cdot 10^{-8}$ | $r_t$:3.0 |
| Deformation zone NE107A (0–100 m) | 75 | $T=5 \cdot 10^{-5}$ | $r_t$:3.0 |
| Deformation zone NE107A (100–200 m) | 180 | $T=3 \cdot 10^{-5}$ | $r_t$:3.0 |
| **Shaft (from central area to surface level)** | | | |
| HRD_C (0–150 m) | 75 | $K=2 \cdot 10^{-7}$ | $r_s$:2.0 |
| HRD_C (150–400 m) | 275 | $K=2 \cdot 10^{-8}$ | $r_s$:2.0 |
| HRD_C (400–500 m) | 450 | $K_{10\text{-}perc}=9 \cdot 10^{-10}$<br>$K_{median}=5 \cdot 10^{-9}$<br>$K_{90\text{-}perc}=2 \cdot 10^{-8}$ | $r_s$:2.0 |

**Table B2. Input data for calculating the inflow to functional area "central area".**

| Underground opening/part of rock mass/depth | H (m) | T (m²/s) or K (m/s) | r ($r_t$, $r_s$) (m) |
|---|---|---|---|
| **Rock caverns (6) (depth 500 m)** | | | |
| HRD_C | 500 | $K_{10\text{-}perc}=9 \cdot 10^{-10}$<br>$K_{median}=5 \cdot 10^{-9}$<br>$K_{90\text{-}perc}=2 \cdot 10^{-8}$ | $r_t$:8.0 |

**Table B3. Input data for calculating the inflow to functional area "deposition area" and transport tunnels.**

| Underground openings/part of rock mass/depth | H (m) | T ($m^2$/s) or K (m/s) | r ($r_t$, $r_s$) (m) |
|---|---|---|---|
| **Deposition tunnels (per tunnel) (depth 500 m)** | | | |
| HRD_C | 500 | $K_{10\text{-}perc}=9\cdot10^{-10}$<br>$K_{median}=5\cdot10^{-9}$<br>$K_{90\text{-}perc}=2\cdot10^{-8}$ | $r_t$:2.5 |
| HRD_W | 500 | $K_{10\text{-}perc}=4\cdot10^{-11}$<br>$K_{median}=4\cdot10^{-9}$<br>$K_{90\text{-}perc}=1\cdot10^{-7}$ | $r_t$:2.5 |
| HRD_EW007 | 500 | $K_{10\text{-}perc}=2\cdot10^{-8}$<br>$K_{median}=3\cdot10^{-8}$<br>$K_{90\text{-}perc}=5\cdot10^{-8}$ | $r_t$:2.5 |
| Deformation zones, < 3 km | 500 | $T_{min}=2\cdot10^{-7}$<br>$T_{median}=5\cdot10^{-7}$<br>$T_{max}=4\cdot10^{-6}$ | $r_t$:2.5 |
| **Transport and main tunnels (depth 500 m)** | | | |
| HRD_C | 500 | $K_{10\text{-}perc}=9\cdot10^{-10}$<br>$K_{median}=5\cdot10^{-9}$<br>$K_{90\text{-}perc}=2\cdot10^{-8}$ | $r_t$:3.5/4.0 |
| HRD_W | 500 | $K_{10\text{-}perc}=4\cdot10^{-11}$<br>$K_{median}=4\cdot10^{-9}$<br>$K_{90\text{-}perc}=1\cdot10^{-7}$ | $r_t$:3.5/4.0 |
| HRD_EW007 | 500 | $K_{10\text{-}perc}=2\cdot10^{-8}$<br>$K_{median}=3\cdot10^{-8}$<br>$K_{90\text{-}perc}=5\cdot10^{-8}$ | $r_t$:3.5/4.0 |
| Deformation zones, < 3 km | 500 | $T_{min}=2\cdot10^{-7}$<br>$T_{median}=5\cdot10^{-7}$<br>$T_{max}=4\cdot10^{-6}$ | $r_t$:3.5/4.0 |
| Deformation zone NS059A | 500 | $T=1\cdot10^{-5}$ | $r_t$: 3.5 |
| Deformation zone NE107A | 500 | $T=3\cdot10^{-6}$ | $r_t$: 3.5 |

**Table B4. Input data for calculating the inflow to functional area "deposition area".**

| Underground opening/part of rock mass/depth | H (m) | T ($m^2$/s) or K (m/s) | r ($r_t$, $r_s$) (m) |
|---|---|---|---|
| **Exhaust shaft SA01 (0–500 m)** | | | |
| HRD_C (0–150 m) | 75 | $K=2\cdot10^{-7}$ | $r_t$:1.5 |
| HRD_C (150–400 m) | 275 | $K=2\cdot10^{-8}$ | $r_t$:1.5 |
| HRD_C (400–500 m) | 450 | $K_{10\text{-}perc}=9\cdot10^{-10}$<br>$K_{median}=5\cdot10^{-9}$<br>$K_{90\text{-}perc}=2\cdot10^{-8}$ | $r_t$:1.5 |
| **Exhaust shaft SA02 (0–500 m)** | | | |
| HRD_W (0–150 m) | 75 | $K=2\cdot10^{-7}$ | $r_t$:1.5 |
| HRD_W (150–400 m) | 275 | $K=3\cdot10^{-8}$ | $r_t$:1.5 |
| HRD_W (400–500 m) | 450 | $K_{10\text{-}perc}=4\cdot10^{-11}$<br>$K_{median}=4\cdot10^{-9}$<br>$K_{90\text{-}perc}=1\cdot10^{-7}$ | $r_s$:1.5 |
| Deformation zone klx11_dz11 | 400 | $T=8\cdot10^{-7}$ | $r_t$:1.5 |

## References – Appendix B

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# Appendix C

## Grout recipes

### Memo

### Grout for final depository, Design D2

This memo is provided by SKB, and presents the grout recipes that are supplied for final repository design D2.

The products, compositions and properties that are presented here are the same as those used, or that have resulted from, the fine sealing project at Äspö (SU32516). This means that SKB has its own experience of the presented compositions at 450 m depth – although to a relatively small extent – except for the plug grout which was not used in the project.

#### Formal handling of grout choice

According to the nuclear fuel project the choice of grout needs to be motivated and formulated in a technical decision; this will subsequently be done by SKB.

#### Silica sol

The name of the silica sol product is Meyco MP320 and it has a dry content of 40%. The fluid is named sol because it is a colloidal solution, i.e. fine particles of silica suspended in water (not sedimentary).

Meyco MP320 has a density of 1.3 at 20 degrees C.

A salt, sodium chloride or calcium chloride, is added to control the gelling. It has also been demonstrated in the field that it is easier to mix the silica sol with sodium chloride than with calcium chloride.

The sodium chloride solution contains 10 per cent by weight sodium chloride and has a density of 1.0 at 20 degrees C.

The mixing ratio controls the gelling time, which is one of the variables in the design. Normal mixing ratios can be 4–6 parts silica sol to 1 part sodium chloride solution, but this may also vary further depending on how one wish to choose borehole spacing and grouting pressure.

**Examples of gelling times:**

| | | | |
|---|---|---|---|
| Weight ratio: silica sol/ NaCl solution | 4.5:1 | 5:1 | 5.5:1 |
| Gelling time at 15ºC [min] | 21 | 35 | 59 |

#### Cement-based grout

The cement-based grout has been developed in cooperation by Posiva and SKB and subsequently further developed by Posiva. Due to national product differences the proportion of active substance in the super plasticiser is somewhat smaller in the fine sealing project at Äspö than in the super plasticiser of the original composition.

In addition to the injection grout, Posiva also tested a plug grout for filling tight holes.

In the fine sealing project at Äspö, SKB mainly has used the originally composed grout, but also a thicker grout (lower water/dry material ratio) referred to below as Stop-grout from the fine sealing project.

**Composition, Injection grout:**

| | Weight ratio | Material in the fine sealing project |
|---|---|---|
| Water | 1.68 | |
| Portland cement * | 1.00 | Ultrafin 16 |
| Silica fume** | 1.37 | Grout Aid |
| Super plasticiser*** | 0.07 | SIKA Melcrete |
| Water/dry material (W/DM) | 1.4 | |

*Sulphate resistant Ordinary Portland cement with $d_{95}$ on 16 µm, type Ultrafin 16 or equivalent

** Dispersed silica fume, microsilica with $d_{90}$= 1 µm type GroutAid or equivalent. The density is to be between 1,350–1,410 kg/m$^3$ and 50%±2% of the solution is to consist of solid particles.

*** Super plasticiser, naphthalene-sulphonate based, density about 1,200 kg/m$^3$, type Melcrete.

**Properties, Injection grout:**

The following properties have been measured in the field when testing injection grout in the fine sealing project:

| | Mean value |
|---|---|
| Density [kg/m$^3$] | 1,330 |
| Marsh-cone time [s] | 43 |
| Shear limit [Pa] | 15 |
| Viscosity [mPas] | 22 |
| Shear strength 6 h [kPa] | 1.5 |
| Separation 2 h [%] | 0 |

The following properties of injection grout have been measured in Posiva laboratory tests:

| | |
|---|---|
| $b_{min}$ [µm] | 40 |
| $b_{crit}$ [µm] | 88 |

**Composition, Plug grout**

| | Weight ratio | Comments |
|---|---|---|
| Water | 0.80 | |
| Portland cement | 1.00 | Ultrafin 16 |
| Silica fume | 1.38 | Grout Aid |
| Super plasticiser | 0.07 | SIKA Melcrete |
| Water/dry material (W/DM) | 0.9 | |

**Properties, Plug grout**

Measured properties from the laboratory of Posiva’s plug grout direct after mixing; values from Posiva:

| Property | Mean value |
|---|---|
| Density [kg/m$^3$] | 1,490 |
| Marsh-cone time [s] | >100 |
| Shear limit [Pa] | 114 |
| Viscosity [mPas] | 90 |
| Shear strength 6 h [kPa] | 1.3 |

**Composition, Stop grout from the fine sealing project:**

| | Weight ratio | Comments |
|---|---|---|
| Water | 0.64 | |
| Portland cement | 1.00 | Injecting 30 |
| Silica fume | 1.37 | Grout Aid |
| Super plasticiser | 0.07 | SIKA Melcrete |
| Water/dry material (W/DM) | 0.82 | |

**Composition, Stop grout from the fine sealing project:**

Measured properties from the field of stop grout in the fine sealing project:

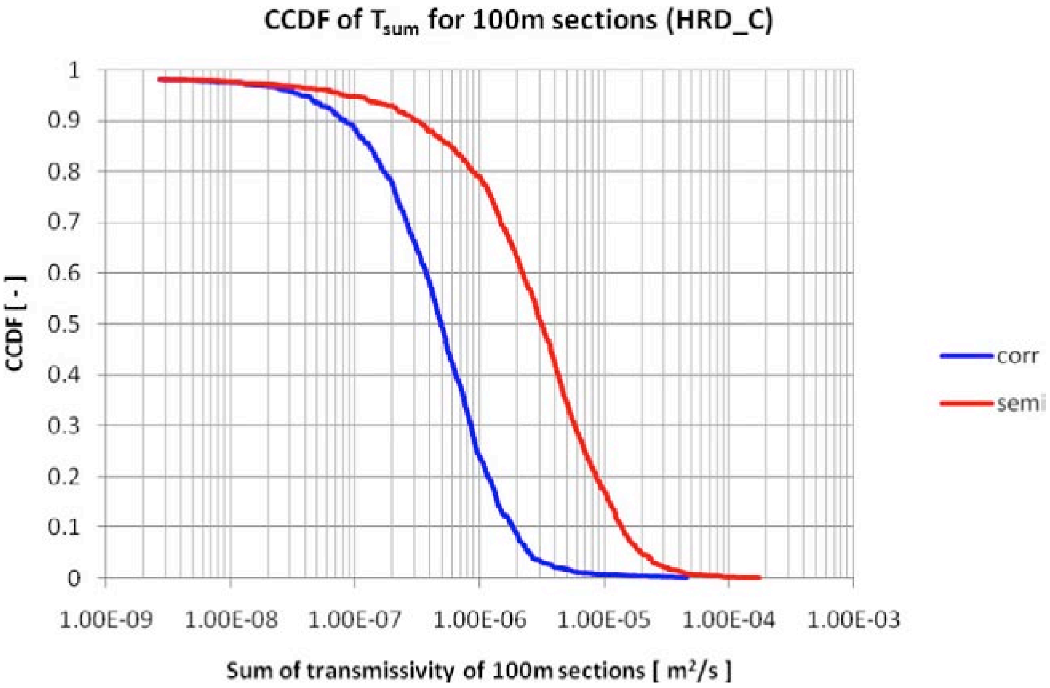
| Property | Mean value |
|---|---|
| Density [kg/m$^3$] | 1,520 |
| Marsh-cone time [s] | 52 |
| Shear limit [Pa] | 15 |
| Viscosity [mPas] | 30 |

## Appendix D

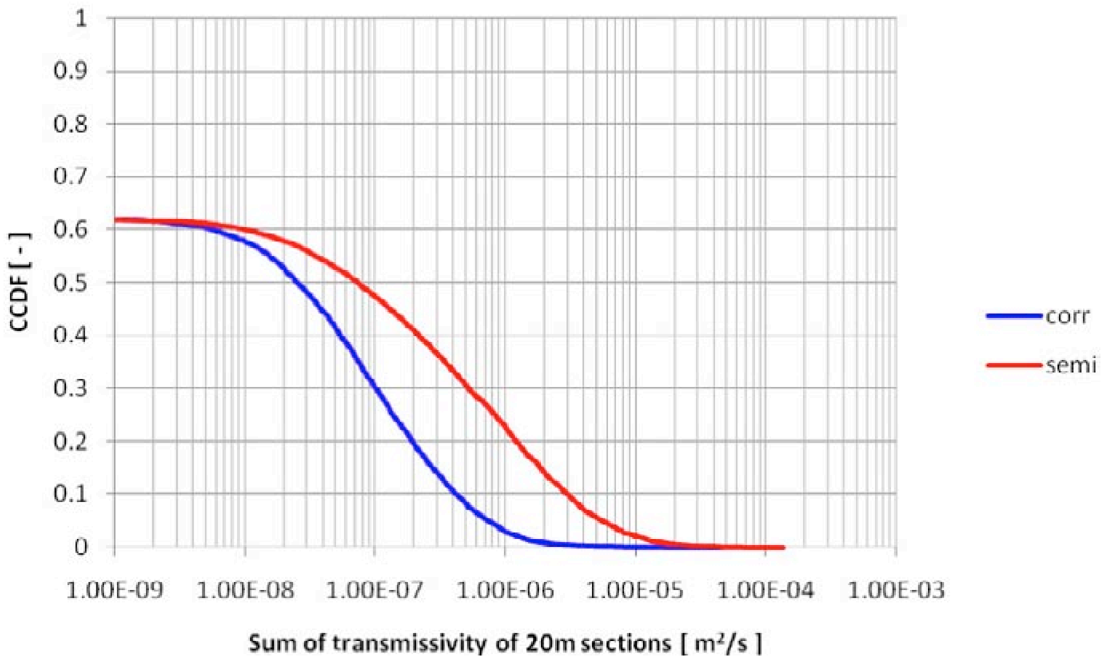
# Distribution of the transmissivity for deposition tunnels and hole

/Stigsson, 2009/ present cumulative distributions for the sum of the transmissivity along 20 and 100 meter horizontal sections for deposition tunnels and 8 meters vertical section for deposition holes.

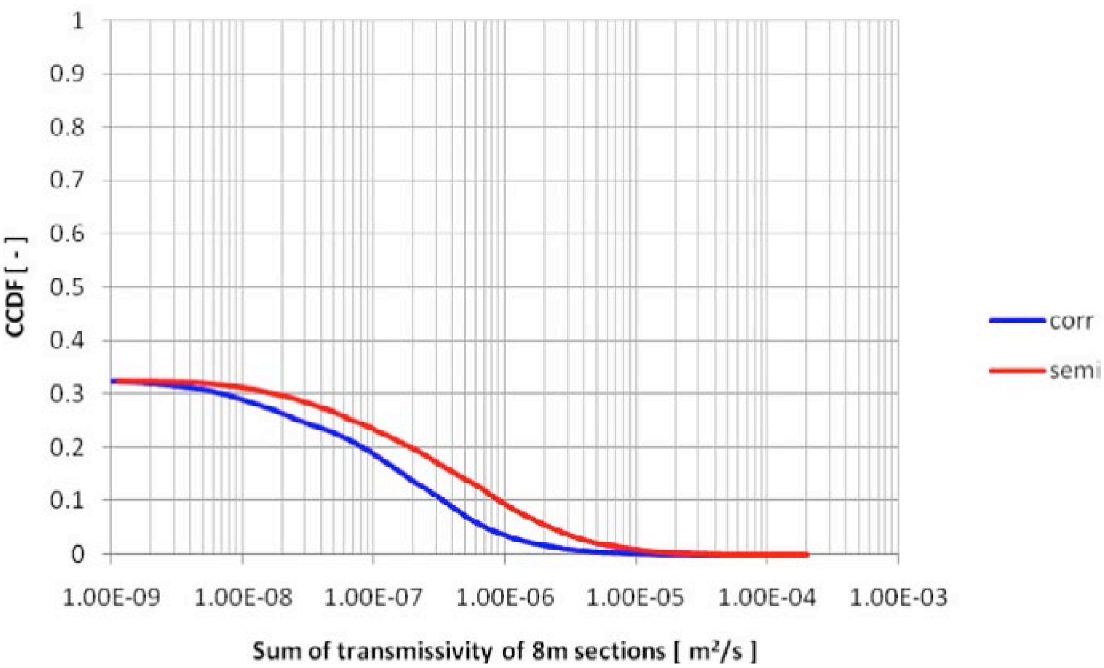
### HRD_C:



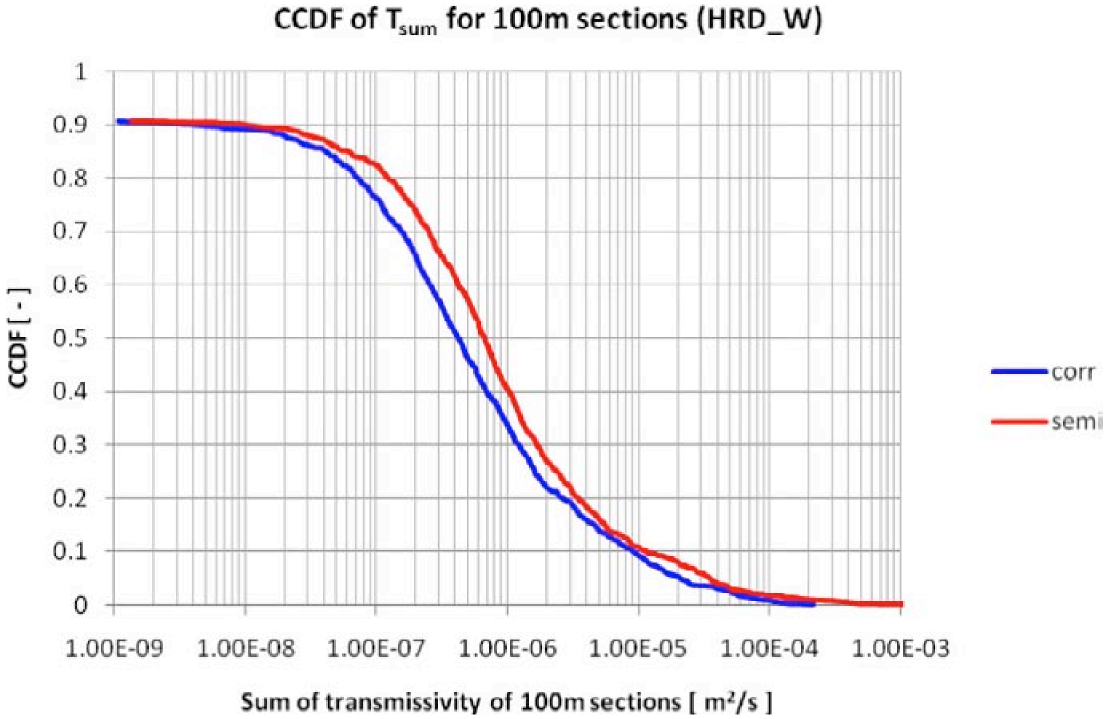
CCDF of $T_{sum}$ for 20m sections (HRD_C)

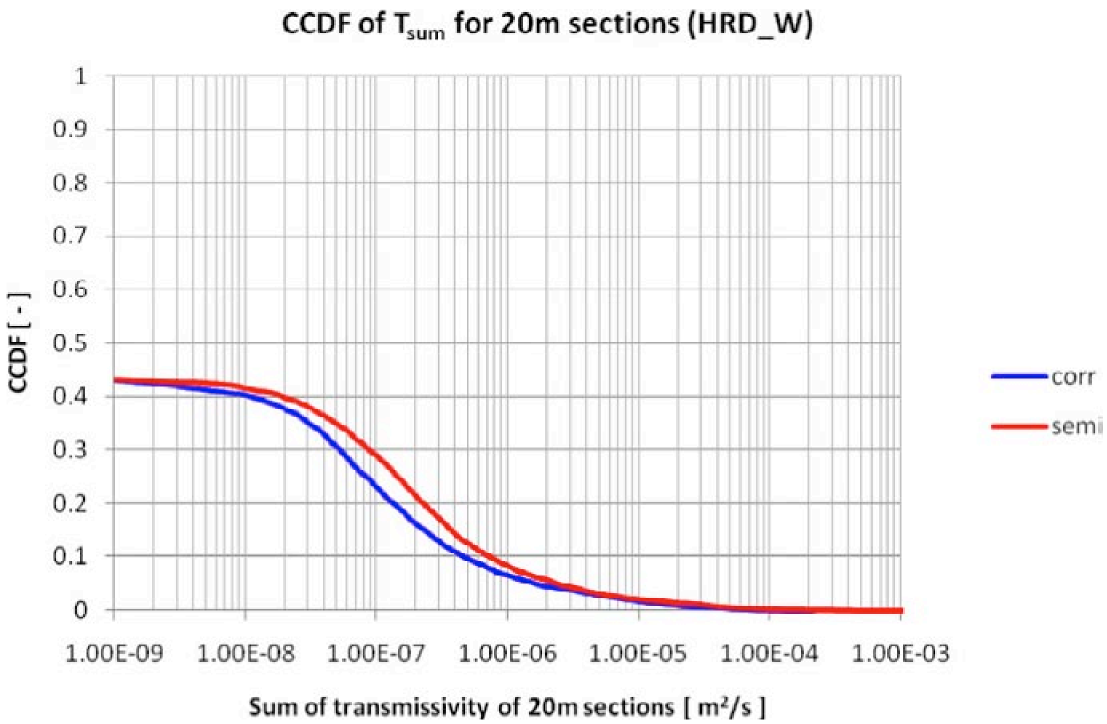


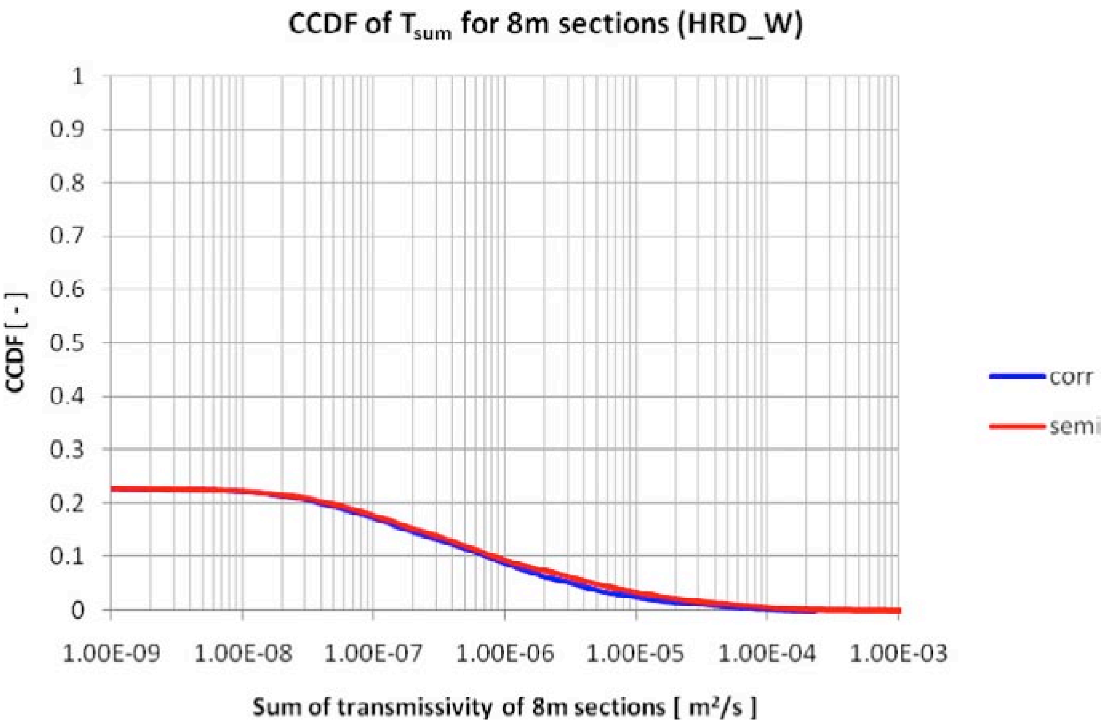

CCDF of $T_{sum}$ for 8m sections (HRD_C)



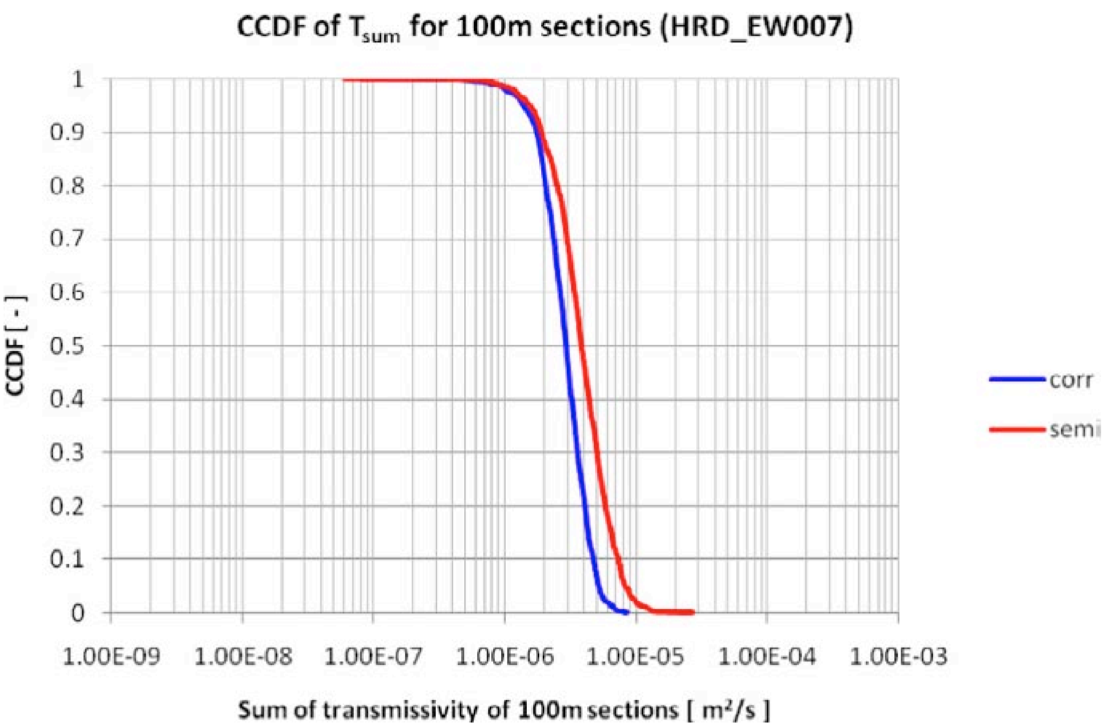
**HRD_W:**

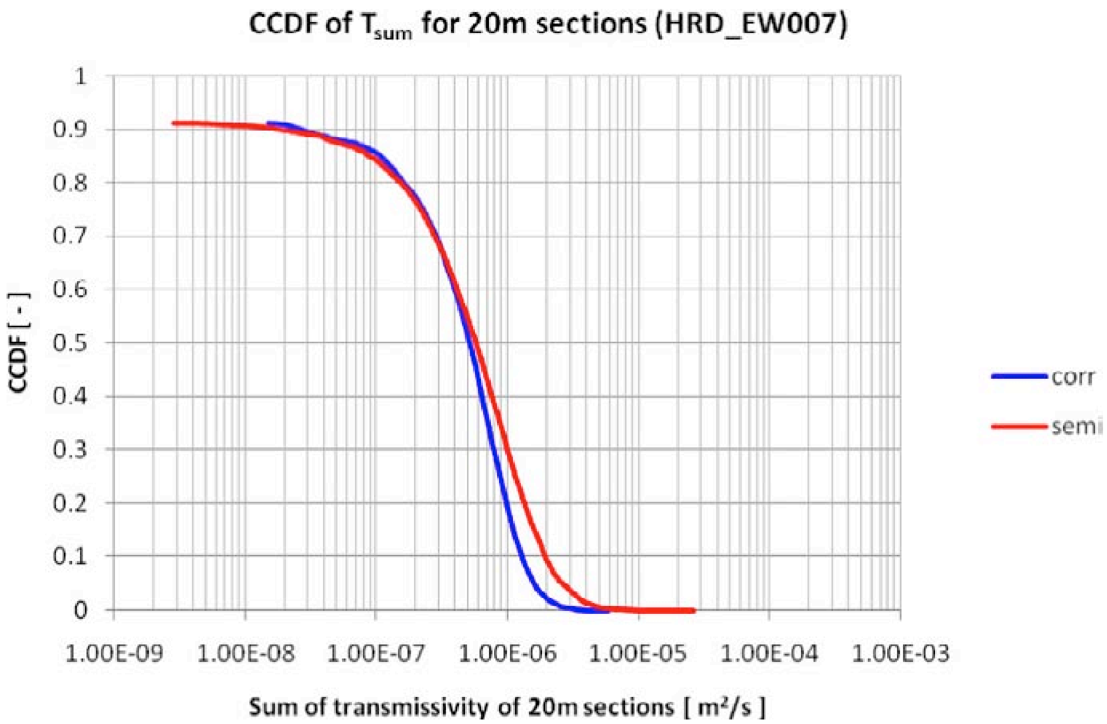

CCDF of $T_{sum}$ for 100m sections (HRD_W)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 100m sections [ m²/s ]
corr
semi


CCDF of $T_{sum}$ for 20m sections (HRD_W)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 20m sections [ m²/s ]
corr
semi

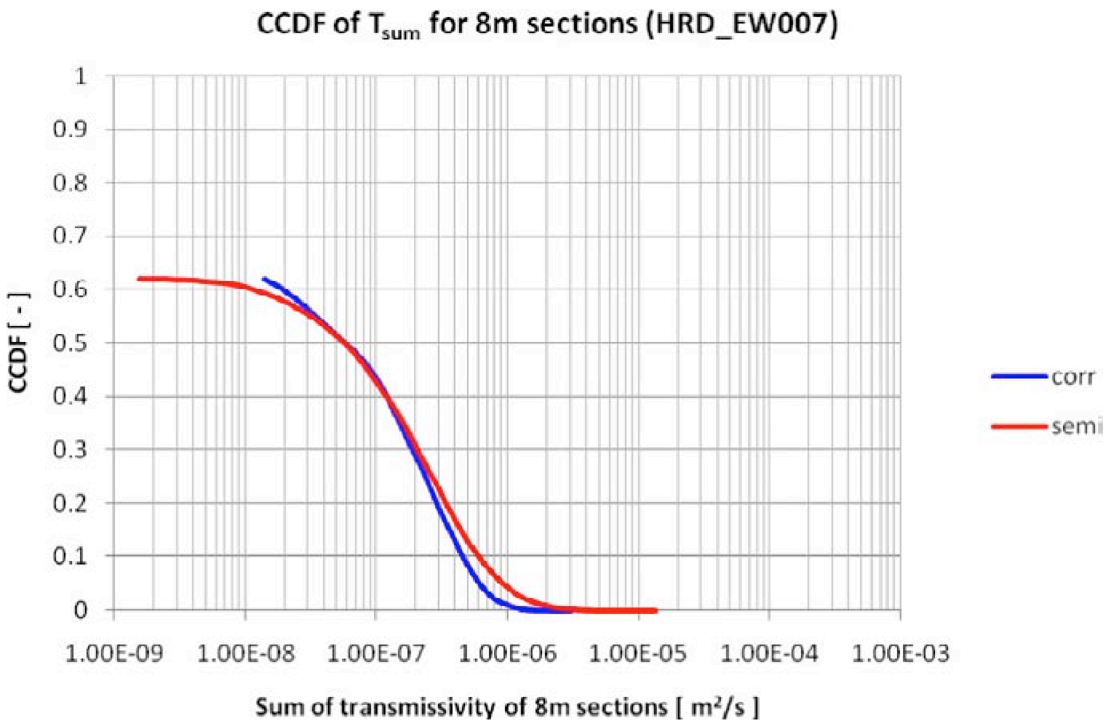

CCDF of $T_{sum}$ for 8m sections (HRD_W)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 8m sections [ m²/s ]
corr
semi

**HRD_EW007:**


CCDF of $T_{sum}$ for 100m sections (HRD_EW007)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 100m sections [ m²/s ]
corr
semi



CCDF of $T_{sum}$ for 20m sections (HRD_EW007)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 20m sections [ m²/s ]
corr
semi

CCDF of $T_{sum}$ for 8m sections (HRD_EW007)
CCDF [ - ]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
1.00E-04
1.00E-03
Sum of transmissivity of 8m sections [ m²/s ]
corr
semi