R-09-16

# Underground design Laxemar Layout D2

Svensk Kärnbränslehantering AB

November 2009

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ISSN 1402-3091
SKB Rapport R-09-16

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# Summary

Laxemar candidate area, which is part of the Laxemar-Simpevarp area, is located in the province of Småland, some 320 km south of Stockholm. The area is located close to the shoreline of the Baltic Sea and is within the municipality of Oskarshamn (County of Kalmar), and immediately west of the Oskarshamn nuclear power plant and the Central interim storage facility for spent fuel (Clab). The easternmost part (Simpevarp subarea) includes the Simpevarp peninsula, which hosts the power plants and the Clab facility. The island of Äspö, containing the Äspö Hard Rock Laboratory (Äspö HRL) is located some three kilometres northeast of the central parts of Laxemar. The Laxemar subarea covers some 12.5 km$^2$, compared with the Simevarp subarea, which is approximately 6.6 km$^2$. The Laxemar candidate area has been investigated in stages, referred to as the initial site investigations (ISI) and the complete site investigations (CSI). These investigations commenced in 2002 and were completed in 2008.

During the site investigations, several studies and design steps (D0, D1 and D2) were carried out to ensure that sufficient space was available for the 6,000-canister layout within the target volume at a depth of approximately 500 m. The guidelines for the layout were outlined in the Underground Design Premises/D2 and the parameters and constraints for the underground design were provided in the Laxemar Site Engineering Report. The findings from design Step D2 for the underground facilities including the access ramp, shafts, rock caverns in a Central Area, transport tunnels, and deposition tunnels and deposition holes are contained in this report. The layout for these underground excavations at the deposition horizon requires an area of 5.7 km$^2$, and the total rock volume to be excavated is $3,008 \times 10^3$ m$^3$ using a total tunnel length of approximately 115 km.

The layout includes provision for all deterministic deformation zones identified in the site descriptive model. In addition there is a respect distance of 100 m for deformation zones with a trace length longer than 3 km. There are no deposition positions placed in any of these zones. The layout has a gross capacity of 8,031 deposition positions, which provides for a loss of 2,031 deposition (approximately 25%). The 2,031 extra deposition positions are not expected to be sufficient to accommodate all losses due to unacceptable water inflows and intersection of long fractures. To achieve this target, design modifications are likely to be needed.

The behaviour of the underground openings associated with this layout is expected to be similar to the behaviour of other underground openings in the Scandinavian shield at similar depths. The dominant mode of instability is expected to be structurally controlled wedge failure. Stability of the openings will be achieved with traditional underground rock support and by orienting the openings relative to the maximum horizontal stress. The estimated amount of support is on average very low because of the very good quality rock mass anticipated. This conclusion is also supported by the underground experience at the Äspö Hard Rock Laboratory. The layout of the repository area has the deposition tunnels aligned $< 30^o$ relative to the maximum horizontal stress. With this orientation spalling is not anticipated in the deposition tunnels or deposition holes.

The excavations for the Repository Access (shafts and ramps) will encounter the greatest frequency of open water-bearing fractures located between 0 and 150 m depth. These access excavations may result in a groundwater drawdown that will need to be minimised. The rock mass at the repository horizon is expected to encounter water-bearing fractures approximately 1 every 10 m, and in some areas even more frequently. Groundwater inflows are expected to be significant at repository level requiring extensive grouting. Results from grouting analyses indicate that conventional grouting measures may not be sufficient to meet the inflow criterion. In some areas of the repository, e.g. hydraulic domain HRD_EW007, sealing may not be practical with cement-based grouts and other sealing technologies, such as silica sol technology, may be required.

The design and layout presented in this report is based on information compiled at the end of the complete site investigation phase and contained in the report SDM Site TR-09-01 /SKB 2009a/. As with all site investigations, at the scale of the repository, there are uncertainties associated with the interpretation of geological information based on borehole investigations. These uncertainties were identified and the impact of these on the current design was evaluated using risk assessment

methodologies. The conclusion from the risk assessment is that the available gross capacity of about 8,000 deposition positions is unlikely to be sufficient to host a repository with 6,000 deposited canisters, without significant design changes. The reason is that the water inflows to many deposition holes are expected to exceed the allowable values. The problems are worse in hydraulic domain HRD_EW007, which in the current repository layout accounts for about 2,000 positions. This risk can be reduced by avoiding HRD_EW007 and by revising the thermal dimensioning such that the remaining area is used more efficiently, although at the expense of constructing more deposition tunnels. Additional rock volumes for deposition may also be necessary to utilize. Even with these design changes substantial grouting will be needed and it may require special technology to reduce the inflows to acceptable levels. Such research technology has recently been developed by SKB, but it remains to be developed to an industrial scale.

Furthermore, several uncertainties were identified that would provide greater flexibility for the design/layout and should be resolved during the next design step and/or during construction of Repository Access:

- The frequency and distribution of the open water bearing fractures, and their potential drawdown, in the vicinity of the shaft and ramp access.
- The frequency and distribution of the open water bearing fractures requiring non-cement-based grouts.
- Spatial dimensions of minor deformation zones that impact the repository layout.

One means of reducing the risk associated with geological uncertainties is the integration of the Observational Method with the Detailed Design and Construction. A preliminary implementation plan was outlined during this design step that showed how uncertainty in the design parameters could be reduced using the principles of the Observational Method. During the Detailed Design these plans must be fully developed.

# Sammanfattning

Kandidatområdet, Laxemar, som är en del av Laxemar-Simpevarpområdet, ligger i Oskarshamns kommun i östra Småland, cirka 320 km söder om Stockholm. Området är beläget nära Östersjöns kustlinje och omedelbart väster om Oskarshamns kärnkraftverk och centrallagret för använt kärnbränsle (Clab). Den östligaste delen (Simpevarp delområde) inkluderar Simpevarphalvön, där kärnkraftverket och Clab är belägna. Äspölaboratoriet (Äspö HRL) är beläget på Äspö, som ligger cirka tre kilometer nordost om de centrala delarna av Laxemar. Laxemar delområde omfattar cirka 12,5 $km^2$ och Simpevarp delområde cirka 6,6 $km^2$. Platsundersökningen av kandidatområdet har utförts i etapperna, inledande (IPLU) och kompletta platsundersökningar (KPLU).

Under och parallellt med platsundersökningarna genomfördes ett antal studier och tre projekteringssteg (D0, D1 och D2) för att säkerhetsställa, att tillgängligt utrymme fanns tillgängligt för en layout omfattande 6 000 kapselpositioner inom det fokuserade området och på ett djup av cirka 500 m. Riktlinjer för layouten angavs i *Underground Design Premises/D2* (UDP/D2) och parametrar och restriktioner för designen av undermarksanläggningen redovisades i *Laxemar Site Engineering Report* (SER). Resultaten från projekteringssteg D2 redovisas i föreliggande rapport och omfattar tillfartsramper, schakt, bergrum i ett centralområde, transporttunnlar, huvudtunnlar, deponeringstunnlar och deponeringshål. Layouten på deponeringsnivå omfattar 5,7 $km^2$ och den totala uttagna bergvolymen uppgår till 3 $008 \times 10^3$ $m^3$. Den totala tunnellängden är cirka 115 km.

Layouten innefattar samtliga deterministiska deformationszoner och respektavstånd (100 m) för deformationszoner längre än 3 000 m. Inga kapselpositioner är placerade i dessa zoner. Layouten har en bruttokapacitet av 8 031 kapselpositioner, vilket möjliggör ett kapselbortfall av 2 031 positioner (cirka 25 %). Dessa extra 2 031 kapselpositioner förväntas inte vara tillräckliga för att inrymma samtliga bortfall på grund av oacceptabla vatteninflöden och kontakt med långa sprickor. För att uppnå detta krav är det troligt att designen måste modifieras.

Undermarksutrymmenas bärförmåga/respons i layout D2 förväntas motsvara övriga utrymmen i berg, som byggts på motsvarande djup i den skandinaviska urbergsskölden. Den vanligaste formen av instabilitet, som kan förväntas är endera strukturellt betingade blocknedfall och/eller spänningsinducerad spjälkning. Undermarksutrymmenas stabilitet kommer att åstadkommas genom att tillämpa traditionell bergförstärkning och genom att orientera utrymmena i förhållande till största horisontella spänningen. Den analyserade förstärkningsmängden bedöms vara låg, som en följd av bergmassans förväntade mycket goda kvalitet. Denna slutsats stöds också av de erfarenheter, som finns dokumenterade från Äspö HRL. I deponeringsområdets layout är deponeringstunnlarna placerade $< 30^o$ i förhållande till största horisontella spänningen, och med denna orientering förväntas inte spjälkning i deponeringstunnlar eller i deponeringshål.

Den högsta frekvensen av öppna vattenförande sprickor kommer att påträffas i samband med berguttaget av förvarets tillfarter (schakt och ramper) från påslagen ned till 150 m djup. Berguttaget av tillfarterna kan därför medföra en grundvattensänkning, som kräver att förebyggande åtgärder vidtas för att förhindra miljömässiga konsekvenser. Bergmassan på förvarsnivå innehåller vattenförande sprickor med ungefär en spricka på varje 10 m, men i vissa delar av förvaret är sprickfrekvensen högre. Grundvatteninflöde på förvarsnivå förväntas vara betydande, och omfattande injekteringar kommer därför att krävas. Resultaten från injekteringsanalyserna indikerar, att konventionella injekteringsmetoder inte kommer att vara tillräckliga för att möta inflödeskriterierna. Inom vissa delar av förvarsområdet, inom t ex den hydrauliska domänen HRD_EW007, kommer tätning med cementbaserade injekteringsmedel troligen inte vara praktiskt genomförbart, utan andra tätningsmetoder kommer att behöva användas, som t ex tätning med silica sol.

Designen och layouten som presenteras i denna rapport är baserade på den information, som sammanställdes i slutet av KPLU, och som ingår i SDM Site TR-09-01 /SKB 2009a/. I likhet med alla förundersökningar finns osäkerheter i tolkningen av geologisk information från borrhål. Dessa osäkerheter har identifierats, och inverkan på den nuvarande designen har utvärderats genom tillämpning av riskanalysmetoder. Den genomförda riskbedömningen visar, att en tillgänglig bruttokapacitet av 8 000 kapselpositioner inte kommer att vara tillräcklig för att inrymma ett

förvar för 6 000 kapselpositioner utan betydande ändringar av designen. Orsaken till detta är, att vatteninflödet i många deponeringshål förväntas överstiga tillåtna värden. De största problemen förväntas i den hydrauliska domänen HRD_EW007, som i föreliggande förvarslayout skall inrymma cirka 2 000 kapselpositioner. Denna risk kan reduceras genom att undvika HRD_EW007 och genom att revidera den termiska dimensioneringen. Detta skulle innebära, att det återstående området kan användas mera optimalt, men också ökad byggkostnad på grund av flera deponeringstunnlar. Dessutom kan det bli nödvändigt att ta ytterligare bergvolymer i anspråk för deponering. Även med dessa ändringar i designen kommer omfattande injektering att behövas, och troligen kommer speciella injekteringsmetoder att krävas för att reducera inflödena till acceptabla nivåer. SKB har nyligen genomfört forskningsarbete av nya injekteringsmetoder, men det återstår att utveckla detta arbete till industriell tillämpning.

Flera osäkerheter har identifierats, som erbjuder större flexibilitet för designen/layouten, och som kan hanteras under nästa projekteringssteg och/eller under berguttaget av förvarets tillfarter:

- Frekvensen och fördelningen av öppna vattenförande sprickor och deras potentiella inverkan på grundvattensänkning i närheten av schakt och ramp.
- Frekvensen och fördelningen av öppna vattenförande sprickor som kräver icke-cementbaserade injekteringsmedel.
- Rumslig fördelning av mindre deformationszoner som kan påverka förvarets layout.

Ett sätt att reducera risk som sammanhänger med geologiska osäkerheter är integrering av observationsmetoden med detaljprojektering och berguttag. Ett preliminärt genomförandeprogram för observationsmetoden har utarbetats under projekteringssteg D2, som visar hur osäkerheter i designparametrar kan reduceras genom tillämpning av observationsmetoden. Under detaljprojekteringen skall detta program utvecklas i detalj.

# Contents

# 1 Introduction

The Swedish Nuclear Fuel and Waste Management Co, SKB, manages the radioactive waste from nuclear power plants in Sweden. The Swedish programme for geological disposal of spent nuclear fuel is approaching major milestones in the form of permit applications for an encapsulation plant and a final repository. The final repository consists of several functional components (Figure 1-1): Surface facilities, Repository Access, Central Facility, and the Deposition Area, with each component having specific design requirements. This report is focused on the underground components of the Final Repository with the primary object of developing an excavation strategy and providing a functional design and layout for the facility that meets the overall objective of providing long term safety for the disposal of 6,000 canisters.

Site investigations at Laxemar were completed in 2008 (Figure 1-2). The investigations were carried out according to the guidelines provided in /SKB 2000a, b/ and the findings from these investigations were used to develop a site descriptive model (SDM) for the site /SKB 2009a/. A SDM is an integrated model for geology, thermal properties, rock mechanics, hydrogeology, hydro-geochemistry, bedrock transport properties and a description of the surface system.

During the site investigations, several studies and design steps (D1 and D2, see Figure 1-2) were carried out to develop a suitable layout based on the data contained in the site descriptive model. The findings from design Step D2 for the underground facilities including the access ramp, shafts, rock caverns in a Central Area, transport tunnels, and deposition tunnels and deposition holes are contained in this report.

## 1.1 Site investigations

The Laxemar site is part of the Simpevarp candidate area located in the municipality of Oskarshamn, about 300 km south of Stockholm. The Simpevarp candidate area is divided into two parts, the Simpevarp sub-area, concentrated on the Simpevarp Peninsula and the Laxemar sub-area on the mainland west of the Simpevarp Peninsula (Figure 1-3).

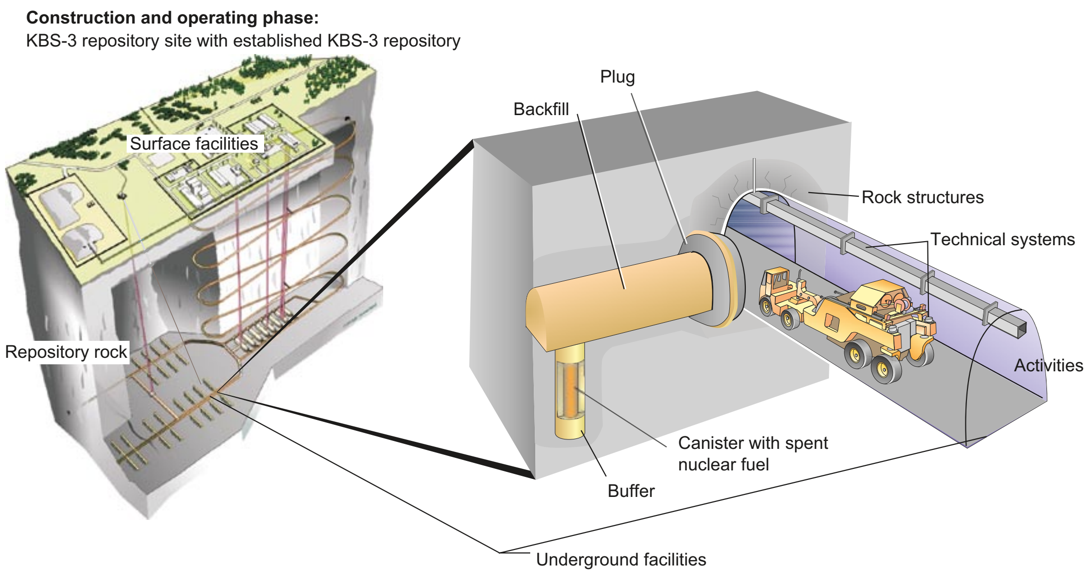



*Figure 1-1.* *General three dimensional overview of three major underground functional areas of the Final Repository, (Access Area, Central Area and Deposition Area). The location of the surface facilities is also shown.*

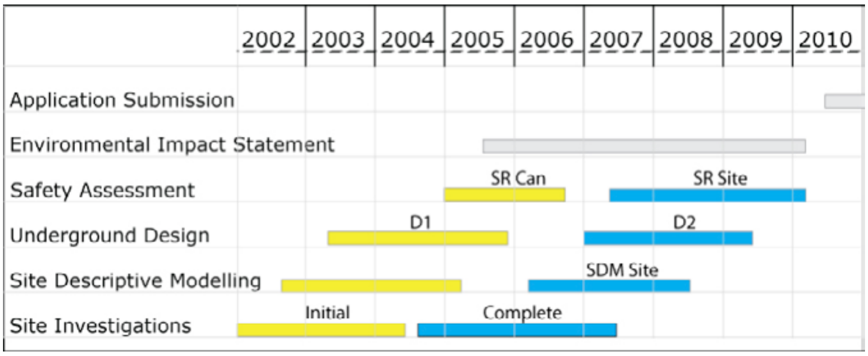



*Figure 1-2. Schedule for the design of the Final Repository Project up to the submission of the application.*

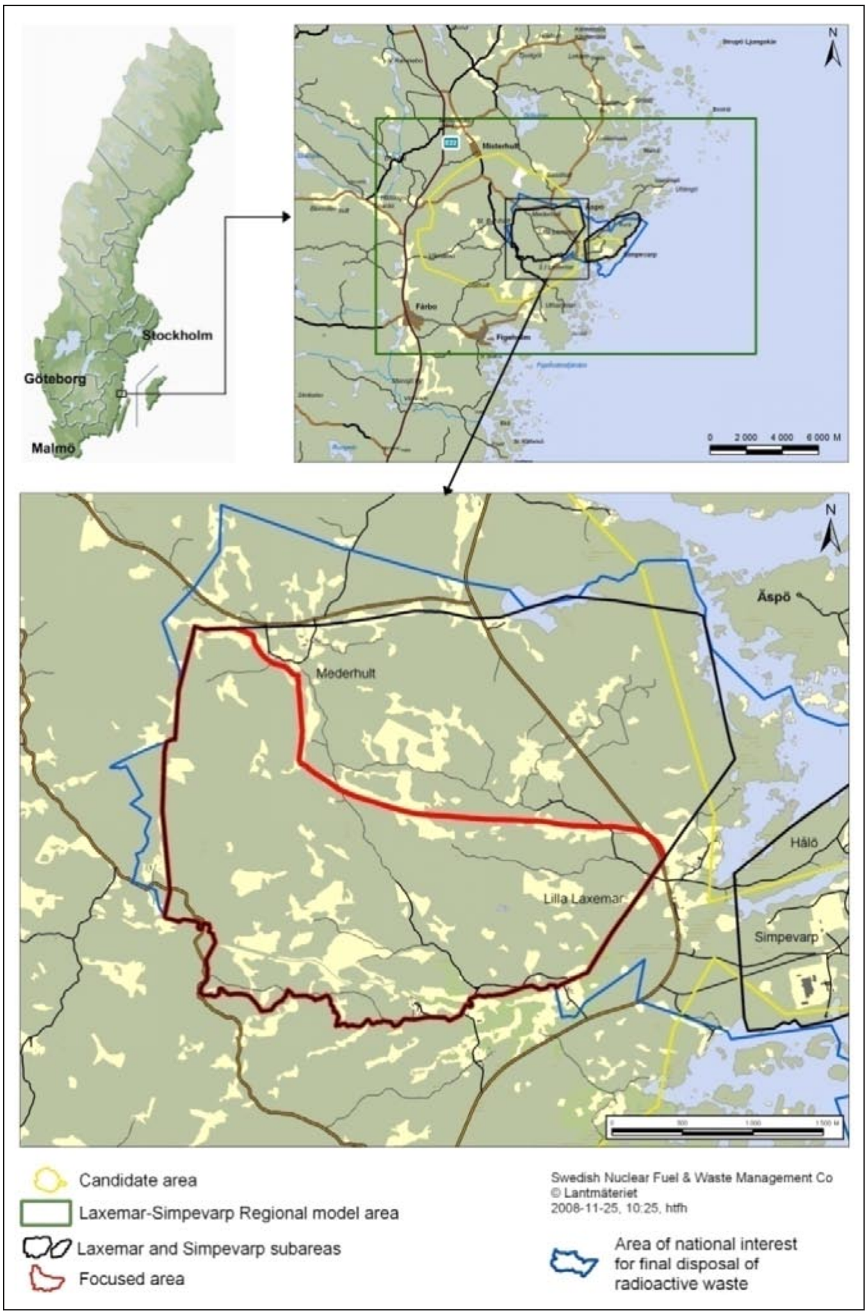



*Figure 1-3. Location of the Laxemar-Simpevarp regional model area and identification of the Simpevarp and Laxemar subareas and Äspö Hard Rock Laboratory. (From /SKB 2009a/.)*

The goal of the site investigation phase was to obtain sufficient information to enable application for permission to site and build the final repository /SKB 2000c/. The geoscientific findings from the site investigation phase provided the knowledge-base required to evaluate the suitability of the investigated sites for a final repository. According to /SKB 2000c/ this knowledge-base must be comprehensive enough to:

- Show whether the selected site satisfies fundamental safety requirements and whether civil engineering prerequisites are met.
- Permit comparisons with other investigated sites.
- Serve as a basis for adaptation of the final repository to the properties and characteristics of the site with an acceptable impact on society and the environment.

The site investigation phase was subdivided into two stages: (1) Initial site investigations and (2) Complete Site Investigations (see Figure 1-2). The initial investigations commenced in 2002 and the complete investigations were finalised in 2008 and are described below. The locations of the drill holes used for both the Initial and Complete site investigations and the boundary of the investigated areas are given in Figure 1-4 and Figure 1-5.

### Initial Site Investigations (ISI)

The initial site investigation stage (ISI) investigations at Laxemar focused on characterising conditions at depth with a limited amount of drilling /SKB 2001/. It was of primary importance to identify any conditions at depth that could not be accepted or were clearly unsuitable for the final repository. During the ISI stage, the candidate area was investigated in order to:

- Provide an initial basis for understanding of the rock and the surface ecosystems on a regional scale.
- Provide a basis for choosing a site within the area for continued investigations.
- To collect information by drilling a limited number of deep investigation boreholes on the site to determine whether the site is suitable for complete site investigations.

***Figure 1-4.*** *Photograph showing the flat topography of the Laxemar site and the outline of the focused area (in red) for the site investigations. The view is looking towards the northwest with the Clab facility in the foreground. (From /SKB 2009a/.)*

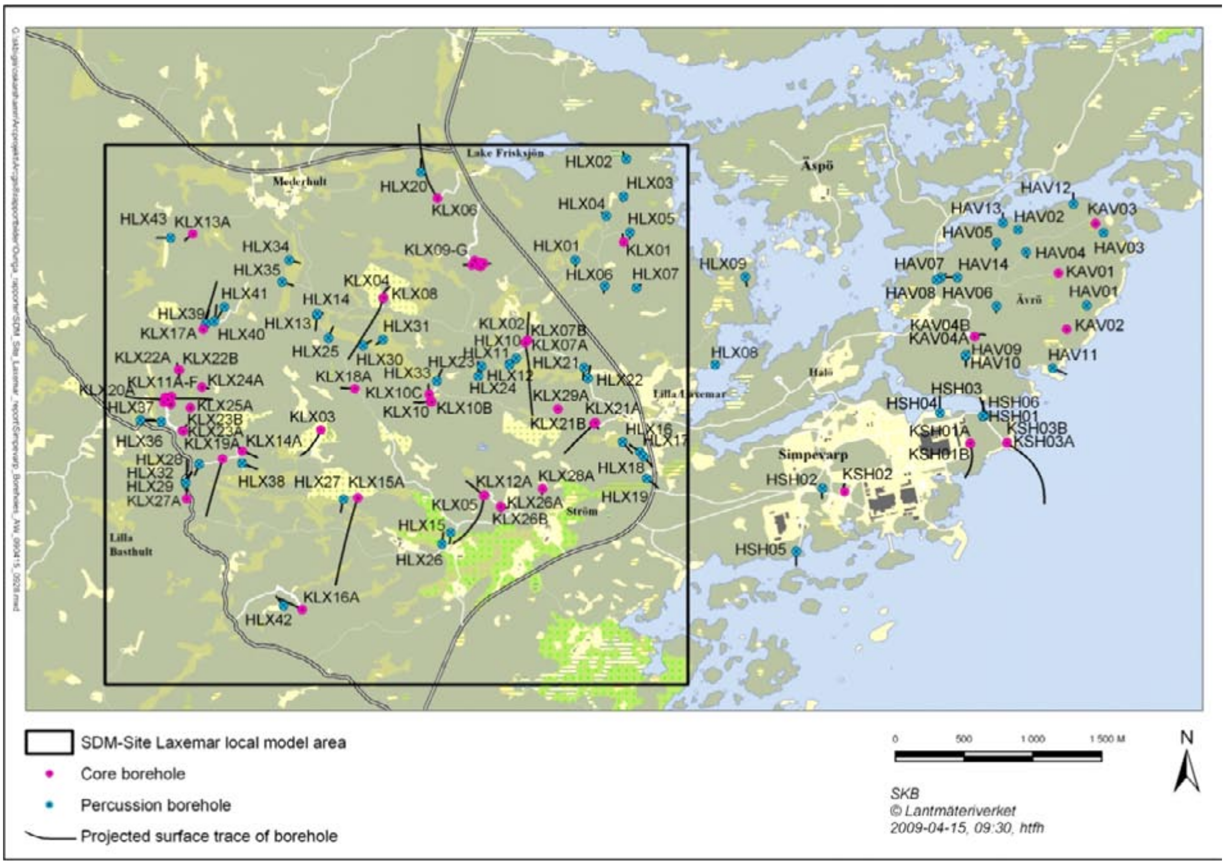



*Figure 1-5. All telescopic, conventionally core-drilled and percussion-drilled boreholes produced during the site investigation at Laxemar and Simpevarp. (From /SKB 2009a/.)*

A drilling and investigation programme comprising a few deep-cored boreholes and a several additional percussion boreholes was carried out to establish the general characteristics of the area that had been identified as a potentially suitable rock volume. In addition, surface geological mapping was performed together with surface and airborne geophysical surveys. The initial investigations were also used to establish the base-line undisturbed site conditions and initiated monitoring of key-parameters that are on-going today. The ISI concluded that the Laxemar site was favourable, and complete investigations were commenced.

## Complete Site Investigations

The Complete Site Investigations (CSI) commenced in 2005 and was completed in 2008. During the stage the investigations focused on:

- Completing the geoscientific characterisation of the site and its environment so that, if the site was found to be suitable, design and safety assessment could produce the supporting material required for a siting application.
- Compiling and presenting all information in site-specific databases and descriptive models of the site's geosphere and biosphere conditions.

The findings from the CSI are compiled in the site descriptive model and given in /SKB 2009a/. Those results have been used as the primary input to this report.

## 1.2 Design process

### 1.2.1 Objectives

The objectives of the overall design activities during the site investigations are given in /SKB 2007/ as:

- Develop facility description(s) for the sites with a proposed layout for the Final Repository Facility's surface and underground parts as a part of the supporting document for an application. The description shall present constructability, technical risks, costs, environmental impact and reliability/effectiveness. The underground layout shall be based on site-specific information from the CSI phase and serves as a basis for the safety assessment.
- Provide a basis for the environmental impact assessment (EIA) and consultations regarding the site of the Final Repository Facility's surface and underground parts with proposed final locations of ramp and shafts, plus the environmental impact of construction and operation.
- Carry out the design work for the entire final repository facility to such an extent that it is possible to plan for the construction phase.

To meet these objectives design activities were carried in parallel to the site investigation program. The reporting of the results from those activities and the process used to achieve them are described below.

### 1.2.2 Design steps

The stages of the site investigation described in Section 1.1 were linked to steps in the design process. Each step was based on the products of preceding design step and the updated site description from the corresponding stage of the site investigations. The design steps carried out during the site investigation phase were named D0, D1 and D2. Design D0 contained feasibility studies on the industrial area. The results from design step D1 were summarised by /Janson et al. 2006/. Design Step D2 presents the design of the reference repository based on the findings in SDM Site (this Report).

### 1.2.3 D2: Objectives, methodology and organisation

The objectives of the underground design activities during design step D2 were to present a site-specific facility description that:

- Demonstrate a site-specific adaptation for a repository considering the overall requirements on functionality, reliability and long term safety based on current state of knowledge after the CSI.
- Demonstrate the constructability and the effectiveness of a step-wise development of the underground parts of the repository.
- Identify site-specific facility-critical issues and provide feedback to:
  - The design organisation regarding technical risks as well as additional studies that needs to be addressed in the next design phase.
  - The safety assessment organisation regarding technical criteria that have an impact on the extent of the repository and its engineered barriers.
  - The SKB management regarding investigation strategies that needs to be included into the step-wise development of the repository.
- Can accommodate all the 6,000 canisters foreseen in SKB's reference scenario.
- Provide material for consultations and EIA according to Chapter 6 of the Environmental Code regarding:
  - The location of the surface facility.
  - The location and extent of the underground facility and the justification of the proposed layout.
  - The technical and functional description of the layout including justification of proposed measures for grouting and support.

To meet these objectives, a steering document, Underground Design Premises/D2 (UDP/D2) /SKB 2007/ was developed and the strategies and approach in UDP/D2 are described in Section 2.1. The design guidelines with regard to long term safety are given in a document called Design Premises

Long Term Safety /SKB 2009b/ and are summarised in UDP/D2. They build on feedback from the safety assessment described in SR-Can /SKB 2006a/, a preparatory stage for SR-Site safety assessment, based on the preliminary site descriptions /SKB 2006b, c, d / and associated layouts. This feedback was considered in /SKB 2007/. The feedback from /SKB 2006a/ and the results from the site investigations were used to develop general guidelines and site-specific constraints for the repository. These guidelines were documented in the Laxemar Site Engineering Report /SER, SKB 2008a/. The flow of information in the design step D2 from SR-Can, SER, and UDP/D2 is shown in Figure 1-6.

The flow of information given in Figure 1-6 was controlled through the *Design Coordinator* and *Project Manager*. The Design Coordinator engaged external resources, hereinafter called *the Designer*, to carry out design, as well as other independent resources, hereinafter called *Reviewers*, to formally review the design results. The overall organisation is illustrated in Figure 1-7. The Design Coordinator was also responsible for coordination with other technical areas and disciplines in matters that impacted the design (see Figure 1-7). An *Advisory Expert Team* supported the Design Coordinator in the development of the Site Engineering Report (cf. Section 2.3) and in developing the risk assessment methodology.

Various teams carried out the design studies for the Laxemar site. The results from those design studies are presented in the following reports:

- Layout studies /Leander et al. 2009/.
- Rock mechanics and rock support /Eriksson et al. 2009/.
- Ground behaviour and grouting measures /Brantberger and Janson 2009/.

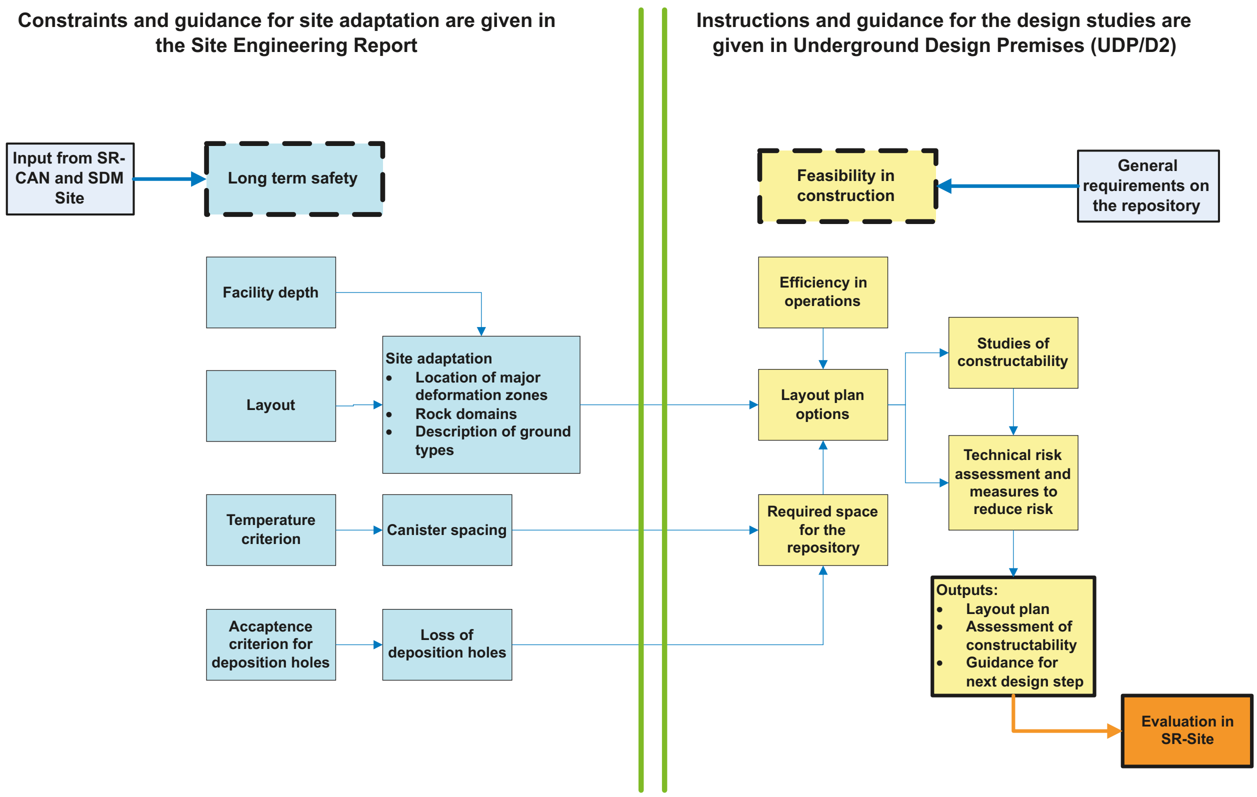



*Figure 1-6.* *Overview of the constraints and main deliverables from the SER (blue boxes) into design activities in accordance to UDP/D2 (yellow boxes).*

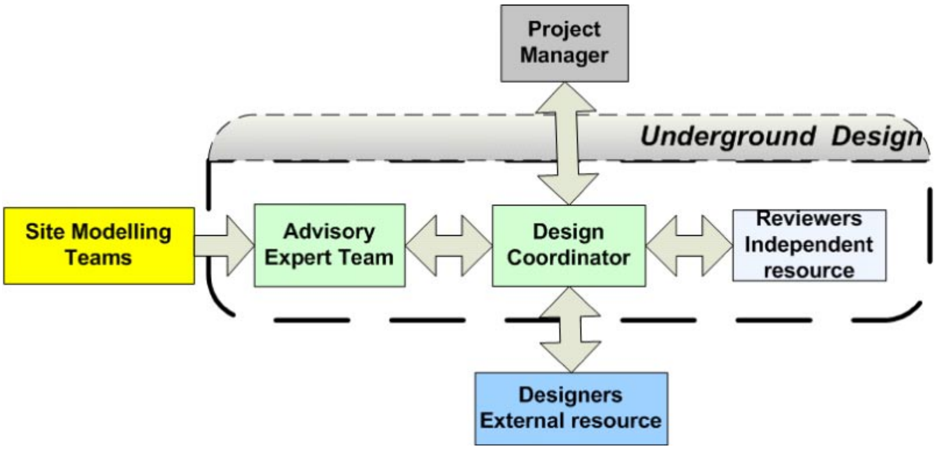



***Figure 1-7.*** *Overall Organisation of the Rock Engineering Design and its interfaces with respect to division of responsibilities and information /SKB 2007/. Compare Figure 2-1 by the colour codes for the different deliverables.*

## 1.3 Objectives and structure of this report

The primary objective of this report is to present the underground layout and design that satisfies the technical issues identified for the site. This report also addresses how the site uncertainties related to the geological description of the site will be addressed during the Detailed Design and repository construction.

Chapter 2 presents a brief description of the steering documents that were used for the underground design in design step D2, and the document Site Engineering Report, which gives general guidelines and site-specific constraints for the underground openings required for the repository. Other constraints such as administrative limits on the surface given by the SKB are also presented.

Chapter 3 provides a summary of the site conditions of importance for the design studies. The Chapter is a résumé of the Site Engineering Report (Section 2.3) /SKB 2008a/ and addresses repository depth, general site description, rock mechanics and hydraulic properties. This includes a brief presentation of rock and fracture domains. Attention is drawn to issues such as deformation zones and respect distances, deposition tunnel alignment, thermo mechanics and canister spacing, and loss of deposition hole positions. Ground type distribution, stress magnitudes and orientation, and categories of ground behaviour are highlighted as well as hydraulic conductivity for different fracture domains and depth intervals.

Chapter 4 describes the proposed underground facility layout including, by way of introduction, some brief characteristics of the surface facility. The first part of the chapter focuses on dimensions of the Repository Access and functions of the underground openings in the Central Area, after which follows a short overview of utilisation of available Deposition Area including ventilation and fire protection, drainage and rock hauling system. Justification of the proposed layout is discussed with reference to Central Area, and to transport, cross, main and deposition tunnels. Alternative layouts are also discussed in this chapter.

Chapter 5 addresses repository development of the Deposition Area. The two construction strategies, separation by side change and separation by the linear development method are described, and in this context, health and safety aspects are recognised. The strategy for step-wise excavation/operation is presented by illustrating the general principle of the extension sequence for repository development. Production volumes for each construction step are given, and transport issues are discussed on the basis of construction strategy.

Chapters 4 and 5 are based on the studies by /Leander et al. 2009/.

Chapter 6 applies to ground control and rock support for each functional area of the repository. The chapter presents analytical and numerical calculations of stress concentration that occur around the openings in different directions in relation to the in situ stress field. Different cases for study of stress concentrations around a deposition hole are illustrated. Furthermore, the chapter deals with support types for different ground behaviour, and estimated amounts of ground support are presented. This Chapter is based on the work by /Eriksson et al. 2009/.

Chapter 7 deals with groundwater control and grouting. The chapter firstly provides estimated amount of water inflow to various functional areas before and after grouting. In the second place, measures to reduce environmental impact of drawdown are described encompassing grouting, infiltration and lining. The chapter addresses a grouting strategy for configuring the grouting measures such as fan geometry, grout, execution, equipment and control measures. Estimated amounts of pre-grout injected before blasting for different functional areas are given. This Chapter is based on the work by /Brantberger and Janson 2009/.

Chapter 8 assesses uncertainty and risk in Design D2. In this Chapter the key uncertainties identified from the findings of the site characterisation programme (SDM Site) that impact the facility layout and underground design were evaluated using risk assessment techniques. The likely occurrence of these uncertainties is also assessed. The risk assessment process and its linkage to the Observational Method are illustrated. The Chapter also outlines the steps needed to reduce the uncertainties during the Detailed Design and repository construction. The Design Coordinator and the Advisory Expert Team have carried out the assessments presented in Chapter 8 (cf. Figure 1-3).

Appendix A contains the typical drawings that describe the dimensions and configurations associated with the underground openings. Appendix B contains a 40-year plan for construction and deposition development.

# 2 Guidelines for the design D2 studies

An overview of the documents that were used in the rock engineering design in design step D2 is shown in Figure 2-1. The documents are presented and described in UDP/D2 /SKB 2007/ and in SER /SKB 2008a/.

## 2.1 Underground Design Premises/D2

The report Underground Design Premises/D2 (UDP/D2) /SKB 2007/ is the steering document for the design of underground openings for a Final Repository Facility during design step D2. UDP/D2 includes design premises, strategy and instructions for the design of underground openings and rock construction works at the two candidate sites Laxemar and Forsmark. The design premises are based on current SKB requirements and on specially elaborated documents, based on the experiences from previous design steps and the needs and objectives of the rock engineering design in design step D2.

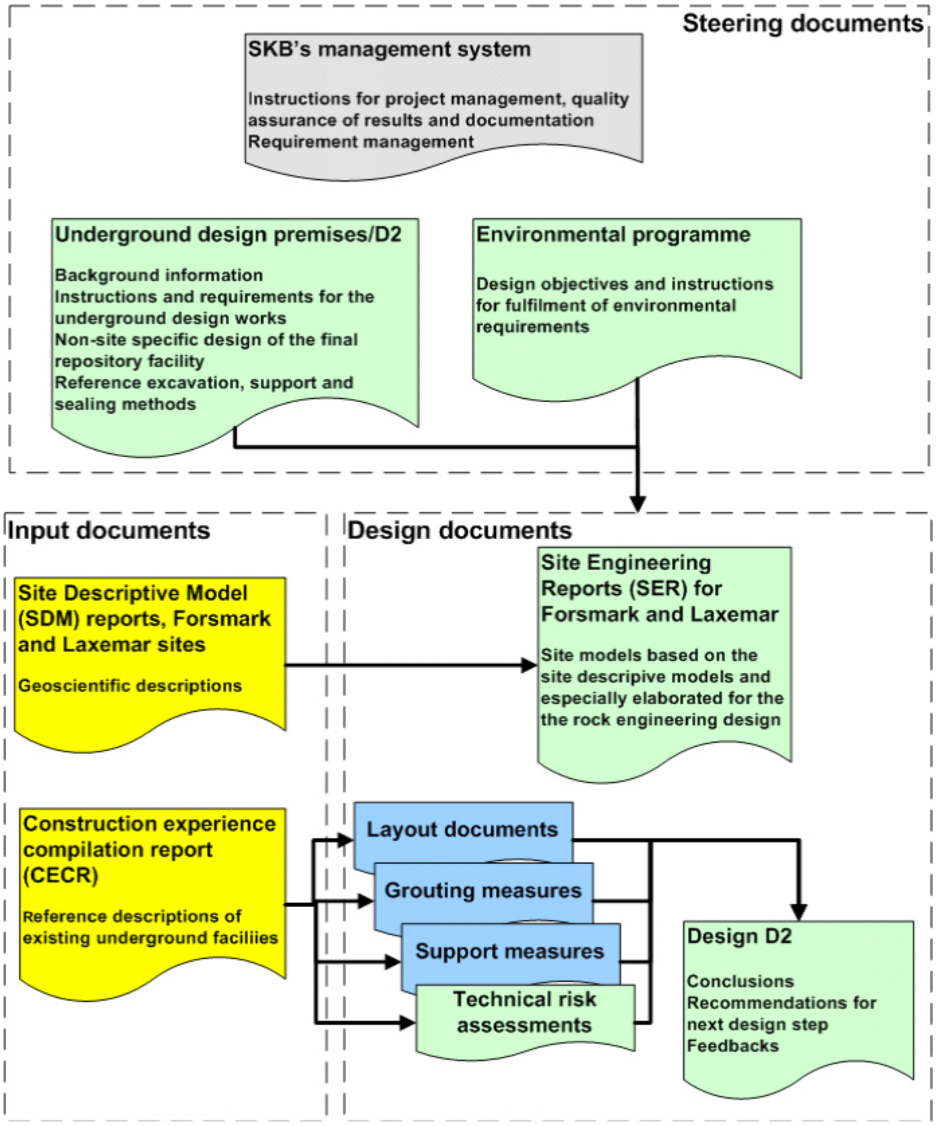



*Figure 2-1. Overview of the documents that were used in the underground design in design stepD2 /SKB2007/. Compare colour codes in Figure 1-7 for responsibilities of the different documents in this Figure.*

The instructions are presented in UDP/D2, in other steering documents and in SKB's management system. The design methodology devised in /SKB 2007/ was to:

1) Carry out a study, based on the design results from design step D1 considering available site information, and defining to what extent new information have any impact on the early design sketches.
2) Study the functionality of the repository in terms of a preliminary logistic plan for step-wise development.
3) Update the estimated required size of the repository and outline an updated sketch layout, in similar detail as the D1 layout.
4) For the layout alternative that is estimated to be most beneficial, study the impact on constructability and assess the System Behaviour, i.e. the interaction between the ground behaviour and construction measures.
5) Each step in the design work should be carried out from a risk perspective, which includes risk assessments for the proposed layout and proposed design solutions.
6) The documentation of design D2 shall also explain which technical solutions do not need to be engineered in detail in this phase.

### 2.1.1 Site Engineering Report

The Site Engineering Report (SER) /SKB 2008a/ presents general guidelines and site-specific constraints for the design of underground openings required for the repository. The general guidelines are based on the current state of practice for underground design while respecting the special needs of the long term safety requirements of the repository. The constraints provided in the SER are site-specific interpretations of the design premises with regard to long term safety listed in Design Premises Long Term Safety /SKB 2009b/.

The SER provided:

- Site-specific constraints.
- Design parameters for the underground design.
- Design procedures/approaches for addressing site-specific constraints.
- Engineering guidelines based on analysis of problems of specific concern for the repository.

SER extracted the relevant data from the SDM Site to develop an engineering description of the rock mass that was adequate for Design Step D2. SER considers the rock domains (relating to intact rock properties), fracture domains, ground water conditions and in situ stress conditions, and incorporates parameters that are required to provide an engineering description of the rock mass. The ground types (GT), which will be encountered during construction is the product of this description. The SER identified the number of ground types to be used in the design and also addressed the site-specific geological conditions that needed to be evaluated during the design.

### 2.1.2 Observational Method

The design was carried out using the principles of the Observational Method. The Observational Method is a risk-based approach to underground design and construction that employs adaptive management, including advanced monitoring and measurement techniques, to substantially reduce costs while protecting capital investment, human health, and the environment. Development of the Observational Method in geotechnical engineering is generally attributed to /Terzaghi and Peck 1948/. /Peck 1969/ formally outlined the essential elements of the methodology and /Stille 1986/ described the adaptation of the method in Sweden under the name "Active Design". Outlining the method in 1969, Peck wrote: *"In brief the complete application of the method embodies the following ingredients:*

(a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.

(b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role.

(c) Establishment of the design based on the working hypothesis of behaviour anticipated under the most probable conditions.

(d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.

(e) Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.

(f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.

The reference design was carried out using the principles of the Observational Method as outlined in /Eurocode EN 1997-1:2004, Section 2.7/, which requires that for the reference design:

1. **Acceptable limits of behaviour** shall be established;
2. The **range of possible behaviour** shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
3. A **plan for monitoring the behaviour** shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits.
4. The **response time of the monitoring** and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
5. A **plan of contingency actions** shall be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.

As noted above the inherent complexity and spatial variability in the geological setting prohibits a complete picture of the ground structure and quality before the facility is excavated. In accordance with the Observational Method, sufficient information was obtained during the site investigation to establish the reference design based on the most probable site conditions. These conditions, i.e. site constraints, were documented in the Site Engineering Report and formed the input for the design and layout. Chapters 4 through 7 of this report documents design and layout based on these most probable site conditions. The Observational Method also requires that possible deviations from the most probable conditions should also be evaluated. The approach used to address this design requirement and the findings are presented in Chapter 8.

## 2.2 Surface-layout constraints

SKB located the industrial area within the *Laxemar area*, and specified the proposed location of the surface facilities (Figure 2-2). A minimum distance of 200 m was specified between the surface facilities and the existing 400 kV transmission line (Figure 2-2).


*Figure 2-2. General view of the Laxemar site looking towards the northeast, with the Oskarshamn Power Plant and the Clab facility in the background.*

# 3 Site conditions of importance for the design studies

## 3.1 Rock domains

A number of rock types were identified at Laxemar during the site investigations (see Table 3-1). The rock types have been grouped into three main rock domains: RSM-A, RSM-D and RSM-M /SKB 2009a/. The distribution of rock types within each rock domain is provided in Table 3-1, and as shown in Table 3-1 there is a number of different rock types in each domain. The variability in rock types impacts the variability in the sampled intact rock properties. The location of rock domains at the ground surface is shown in Figure 3-1.

## 3.2 Fracture domains

SDM Site Laxemar /SKB 2009a/ identified four fracture domains, i.e. volumes with statistically homogeneous fracturing, within design target volume, namely FSM_C (central), FSM_W (west), FSM_EW007 (closer to deformation zone ZSMEW007) and FSM_NE005 (closer to ZSMNE005) (see Figure 3-2). A brief description of those domains is given below.

- Fracture domain FSM_EW007 represents an approximately 250 m thick volume of rock distributed unevenly along deformation zone ZSMEW007A and features a reduced intensity of both N-S striking and open sub-horizontally dipping fractures, although most open fractures appear to belong to the WNW set. Both fracture intensity and orientation have been interpreted as being affected by the E-W striking deformation zone ZSMEW007A.
- Fracture domain FSM_NE005 represents a rock volume west of the regional deformation zone ZSMNE005A, being one of several major belts of NE-SW trending ductile deformation in the region. Domain FSM_NE005 is characterised by a significant increase in the relative intensity of N-S striking sealed fractures.
- Fracture domain FSM_C is the volume of rock south of FSM_EW007, north of zone ZSMNW042A. This domain is dominated by sealed N-S striking fractures in a fashion similar to FSM_W, and open WNW striking fractures.
- Fracture domain FSM_W represents the volume of rock bounded by deformation zones ZSMNS001C, ZSMNS059A, ZSMNW042A, and ZSMEW002A. Borehole data suggest dominant north-south fracture strikes in both subvertically dipping and subhorizontal fracture sets. The third set of fractures strikes ENE and is roughly parallel the NE-SW striking sinistral shear zones that make up the tectonic fabric of the region, and shows a relatively weak intensity.
- FSM_N (north of FSM_EW007) and FSM_S (hanging wall of zone ZSMNW042A) complete the fracture domains interpreted within the local model volume. Overall, patterns of relative fracture intensity inside each domain appear to correspond well to the tectonic history interpreted as part of the deformation zone modelling.

## 3.3 Deformation zones and respect distances

According to the Design Premises – Long Term Safety /SKB 2009b/ deposition positions are not allowed to be placed closer than 100 m to the deformation zones with a trace length longer than 3,000 m. SDM Site identified seven deformation zones that are potentially long enough to require a respect distance: EW002, EW007, NW042a, NS001c, NS059a, NE107a, and NE005a (Figure 3-3).

**Table 3-1. Proportions of rock type occurrence in the three largest rock domains in Laxemar. Compiled from /Hakami et al. 2008/.**

| Occurrence of Rock type (SKB rock code) | Rock Domain RSMA01 [%] | RSMD01 [%] | RSMM01 [%] |
|---|---|---|---|
| Ävrö granodiorite (501056) | 62 | 0.5 | 24 |
| Ävrö quartz monzodiorite (501046) | 22 | 0.6 | 43 |
| | | | |
| Oxidized Ävrö quartz monzodiorite (501046) | 4 | – | 7 |
| Quartz monzodiorite (501036) | 3 | 80 | 0.4 |
| Oxidized quartz monzodiorite (501036) | – | 8 | – |
| Diorite-gabbro (501033) | 0.2 | 0.1 | 16 |
| Fine-grained granite (511058) | 3 | 5 | 5 |
| Fine-grained dioritoid (501030) | 3 | 0.3 | 0.4 |
| Fine-grained diorite-gabbro (505102) 1) | 2 | 2 | 2 |
| Granite (501058) 1) | 1 | 0.4 | 2 |
| Pegmatite (501061) 1) | 0.3 | 1 | 0.5 |
| Dolerite (501027) 1) | – | 2 | – |
| Dolerite (501027) 1) | – | 2 | – |

1) Not included in rock mechanics description.

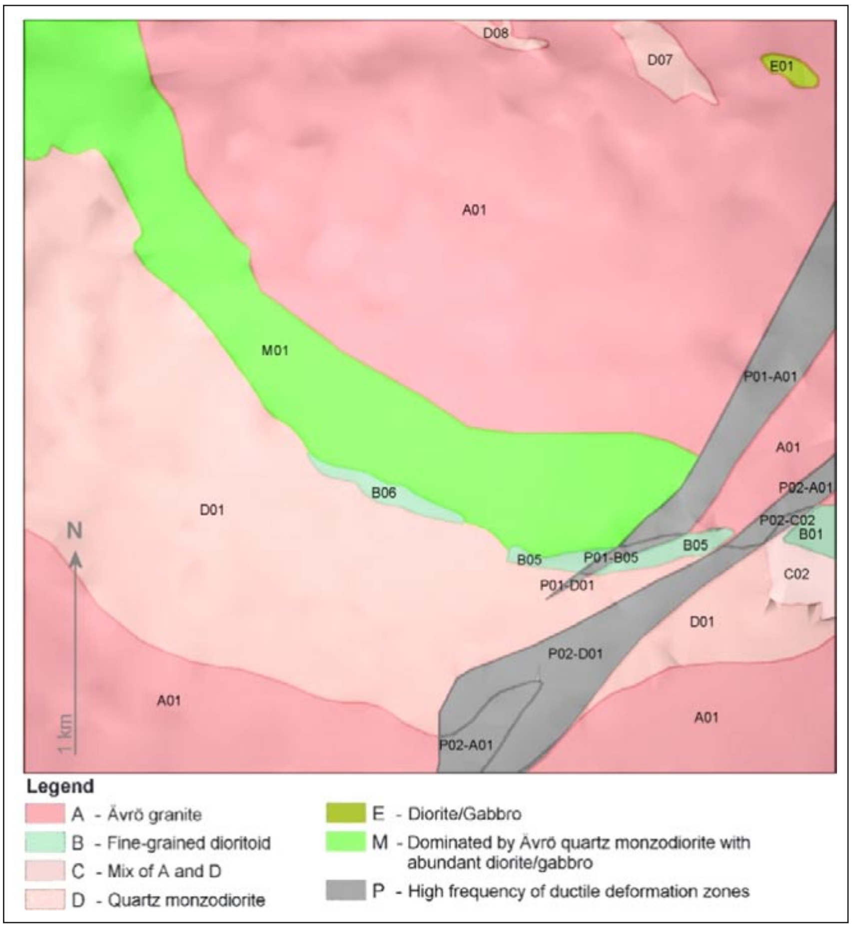
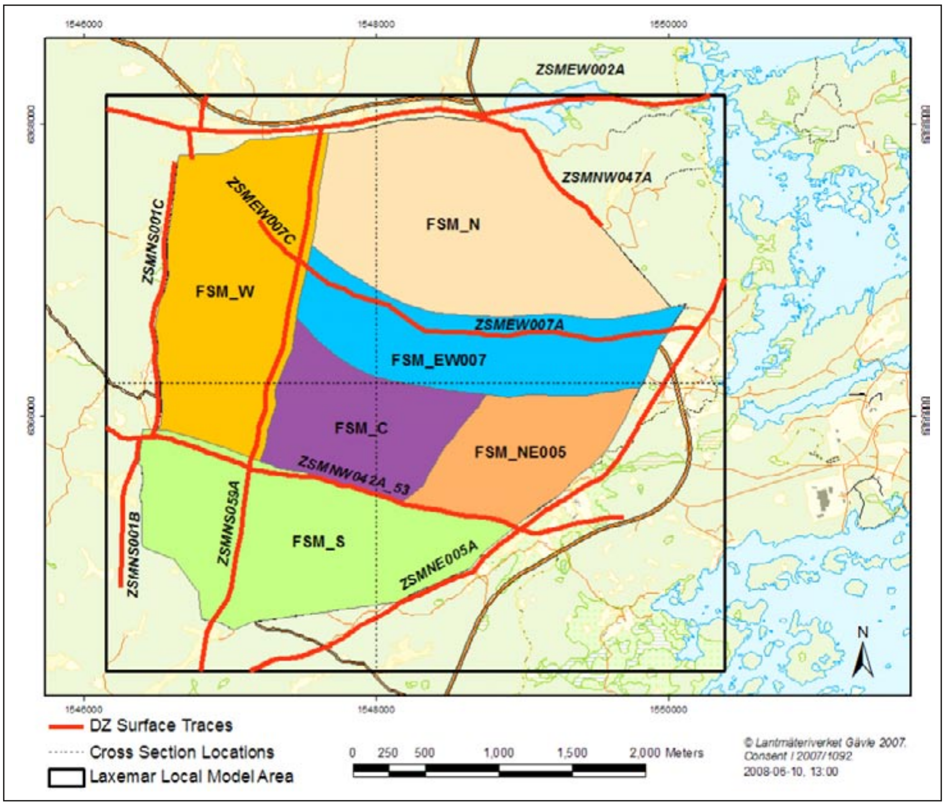



*Figure 3-1. Two-dimensional model at the surface for rock domains in the Laxemar local model. For reasons of simplicity, the prefix RSM has been excluded in the denomination of the rock domains /SKB 2009a/.*




*a) Plan view*


*b) North-South Section view*

***Figure 3-2.*** *Laxemar fracture domains and bounding deformation zones. The black box in (a) represents the limits of the Laxemar local model, while the coloured polygons represent the surface limits of the fracture domains. (from SKB 2009a)*

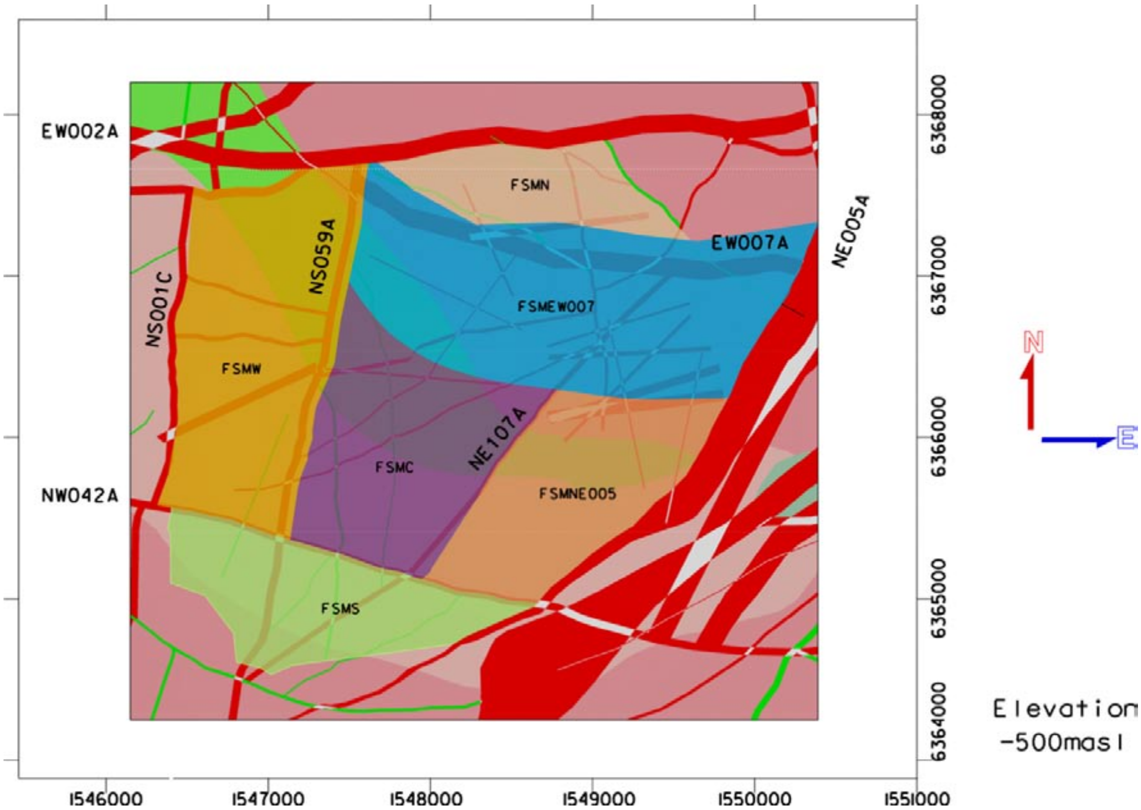



*Figure 3-3. Plan view of rock domains, fracture domains and deterministic deformation zones at Elevation –500 m.Deformation zones EW002, EW007, NW042a, NS001c, NS059a, NE107a, and NE005a require a respect distance.*

## 3.4 Rock mechanics

The laboratory properties for the dominant rock types at Laxemar are provided in Table 3-2. It should be noted that some of the rock types at Laxemar exhibit minor alteration and this alteration may reduce the strength and deformation properties by less than 10%. For this design step this alteration was not evaluated, as there is considerable uncertainty as to the spatial distribution of this alteration, e.g. it may be constrained by its proximity to hydraulically connected deformation zones. The intact properties, in particular the crack initiation stress were used for the spalling analyses.

In SER, the rock mass at Laxemar was divided into four Ground Types (GT) /SER, SKB 2008a/, Table 3-3. These ground types are a general description of the rock type and the discontinuities and used as input when assessing the ground control measures for the site. The anticipated distributions of these ground types are given in Table 3-4.

Two stress models have been proposed for the site and these are summarised in Table 3-5 /SKB 2009a/. The most likely values area based on the borehole stress measurements at Laxemar while the "possible maximum" model is based on the stress measurements made at the Äspö HRL at a depth of 450 m. There is high confidence in the "possible maximum" model as the stress magnitudes and orientation from stress measurements are in agreement with those obtained from large scale back analysis /Andersson 2007/. The "possible maximum" stress model" were used to assess the potential risks for spalling. When the stress magnitudes in Table 3-5 were combined for probability analyses, the ratio of maximum to minimum horizontal stress was constrained between 2.5 and 4.5.

**Table 3-2. Intact laboratory strength and deformation properties for different rock types /Hakami et al. 2008/.**

| Parameter | 501030 Fine-grained dioritoid<br>**Mean/stdev Min–Max** *Uncertainty* | 501033 Diorite/ gabbro<br>**Mean/stdev Min–Max** *Uncertainty* | 501036 Quartz monzo-diorite Unaltered<br>**Mean/stdev Min–Max** *Uncertainty* | 501046 Ävrö quartz monzodiorite Unaltered<br>**Mean/stdev Min–Max** *Uncertainty* | 501056 Ävrö granodi-orite<br>**Mean/stdev Min–Max** *Uncertainty* | 511058 Fine-grained granite<br>**Mean/stdev Min–Max** *Uncertainty* |
|---|---|---|---|---|---|---|
| Uniaxial compressive strength,(MPa) | 239/72<br>100–360<br>*±16%* | 225/20<br>200–270<br>*±5%* | 186/30<br>110–240<br>*±5%* | 167/11<br>140–190<br>*±3%* | 198/19<br>150–240<br>*±3%* | 280/45<br>210–350<br>*±11%* |
| Crack initiation stress (MPa) | 122/53<br>48–190<br>*±28%* | 130/14<br>105–155<br>*±6%* | 104/22<br>52–130<br>*±7%* | 88/19<br>50–110<br>*±9%* | 104/16<br>70–135<br>*±5%* | 148/20<br>110–180<br>*±9%* |
| Indirect tensile strength (MPa) | 19/2.5<br>14–24<br>*±5%* | 16/1<br>15–17<br>*±4%* | 16.5/3.0<br>10–23<br>*±4%* | 13/1.3<br>10–16<br>*±4%* | 13/1.5<br>10–16<br>*±3%* | – |
| Young's modulus (GPa) | 80/8<br>70–97<br>*±5%* | 80/6<br>70–92<br>*±4%* | 76/6.5<br>63–83<br>*±3%* | 71/4<br>63–80<br>*±3%* | 72/5.5<br>60–83<br>*±3%* | 74/2.5<br>70–79<br>*±3%* |
| Poisson's ratio | 0.26/0.05<br>0.17–0.33<br>*±3%* | 0.33/0.03<br>0.30–0.39<br>*±5%* | 0.29/0.03<br>0.20–0.33<br>*±4%* | 0.28/0.06<br>0.16–0.33<br>*±9%* | 0.25/0.05<br>0.15–0.34<br>*±7%* | 0.28/0.03<br>0.22–0.32<br>*±8%* |
| Cohesion (MPa) | 33/7<br>19–47<br>*±10%* | *30* [7)] | 26/3.5<br>19–33<br>*±4%* | 24/1.5<br>21–27<br>*±2.5%* | 24/2<br>20–28<br>*±2.5%* | – |
| Friction angle (°) | 53/0.8<br>51–54<br>*±1%* | 60 | 56/0.3<br>56–57<br>*±0.2%* | 55/0.3<br>55–56<br>*±0.2%* | 60/0.3<br>59–60<br>*±0.2%* | – |

**Table 3-3. Summary of the four ground types for design step D2 /SKB 2008a/.**

| Ground type | Description |
|---|---|
| GT1 | Sparsely fractured rock mass |
| GT2 | Blocky rock mass. Moderately fractured rock contains fractures and hair cracks, but the blocks between joints are intimately interlocked. |
| GT3 | Minor deformation zone |
| GT4 | Major deformation zone |

**Table 3-4. Estimated distribution of ground types (GT1 to GT4) in modelled deformation zones and fracture domains /SKB 2008a/.**

| | GT1 [%] | GT2 [%] | GT3 [%] | GT4 [%] |
|---|---|---|---|---|
| Modelled deformation zones | | | | |
| NE107A | 0 | 30 | 30 | 40 |
| NS059A | 0 | 70 | 30 | 0 |
| Respect distance to EW007, NE107A, NE042A | 0 | 70 | 30 | 0 |
| Respect distance to NS059A | 0 | 80 | 20 | 0 |
| Gently dipping zones < 3 km (0–30°) | 0 | 80 | 10 | 10 |
| Steep zones < 3 km (30–90°) | 20 | 50 | 30 | 0 |
| ***Fracture domains*** [1] | | | | |
| FSM_W | 80 | 20 | – | – |
| FSM_C | 70 | 30 | – | – |
| FSM_NE005 | 70 | 30 | – | – |
| FSM_EW007 | 60 | 30 | 10 | – |

**Table 3-5. Estimated stress models for Laxemar based SDM Site /SKB 2009a/.**

| Parameter | Most likely value (1) | Possible maximum value (2) |
|---|---|---|
| Maximum horizontal stress (MPa) | 3+ 0.039z (±20%) | 31–34 |
| Minimum horizontal stress (MPa) | 1+ 0.022z (±20%) | 10–14 |
| Vertical stress (MPa) | 0.027z (±3%) | 0.027z (± 3%) |
| Maximum horizontal stress Azimuith (degrees) | 135 (±15) | 135(±15) |

1) Depth 0–600 m, 2) Depth 450–650.

## 3.5 Hydraulic properties

The bedrock groundwater system at Laxemar is divided into different hydraulic domains that are dominated by either Deformation zones (HCD) or fractured bedrock between the deformations zones (HRD). These domains contain many water bearing fractures. However, in an average sense the hydraulic conductivity of the Laxemar rock mass between deterministic deformation zones is about an order of magnitude lower than the rock within deterministic deformation zones /SKB 2009a/. The boundaries of the HCDs essentially coincide with the deformation zones (DZ) modelled deterministically in the SDM-Site. Overall assessment of the hydraulic data from the HCDs suggests that the transmissivity of the deformation zone decreases with depth and that the NW-SE, N-S and NE-SW deformation zones are less transmissive than E-W deformation zones within the regional model. Besides the deterministically modelled deformation zones, there are several "minor local deformation zones" (MDZ) observed in boreholes and outcrops. According to /Rhen et al. 2008/ about 60% of the MDZs can be expected to have a conductive feature with a transmissivity $T > 10^{-9}$ $m^2/s$, i.e. the MDZ are usually hydraulically significant features. At depth, these MDZ are part of the hydrogeological DFN model.

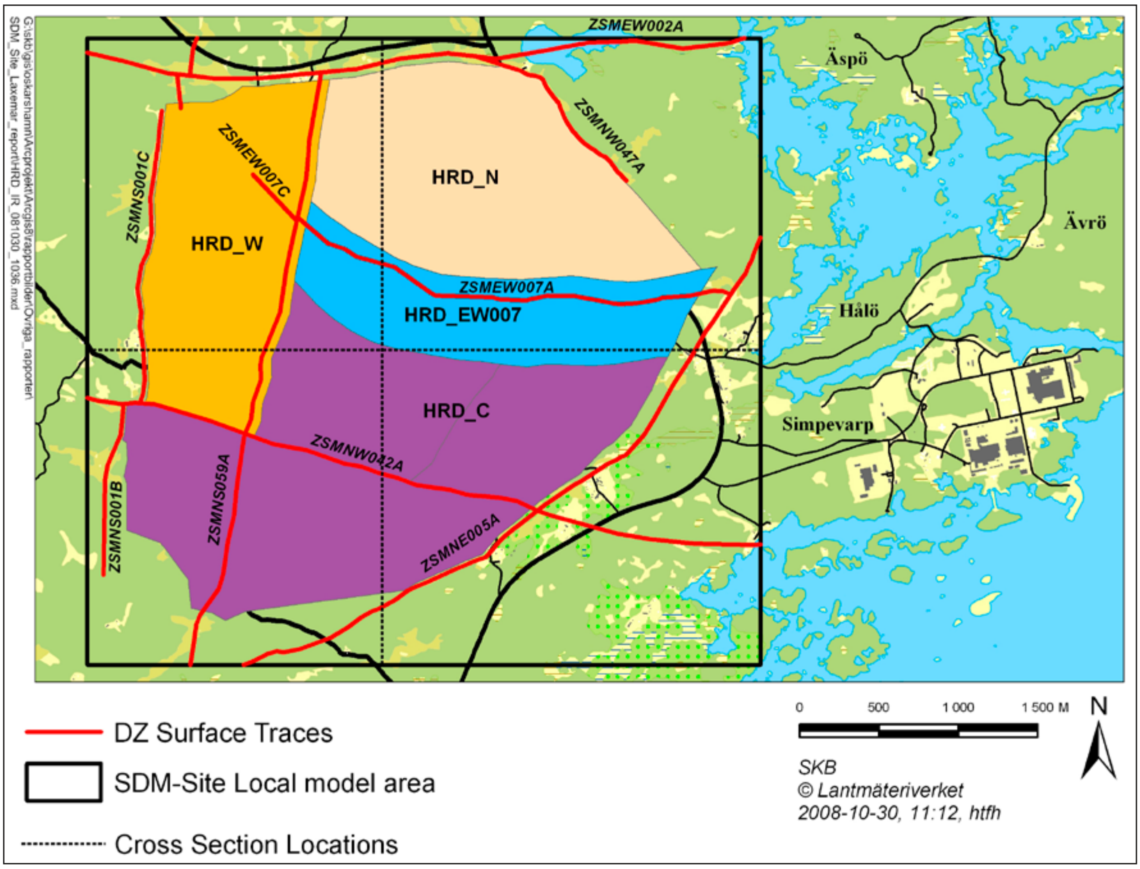
/Rhén et al. 2008/ have identified the following HRD hydraulic domains:

- 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.

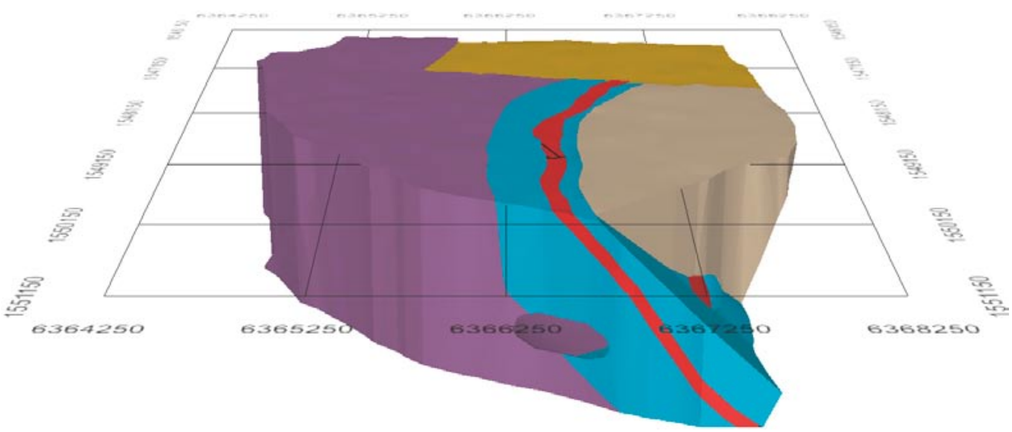
The hydraulic properties of these HRD domains described below and summarised in Table 3-6 and their locations are shown in Figure 3-4.

**Table 3-6. Summary of flowing fracture transmissivity statistics for the different HRD. $P10_{,PFL}$ denotes the linear fracture frequency [$m^{-1}$], T denotes transmissivity [$m^2/s$]. /Compiled from Table 8-3 and 8-4 in /SKB 2009a/ MDZs are included in these statistics, but the numbers of individual PFL fractures are summed within an MDZ such that each is treated as a single feature.**

| HRD (depth range m) | Σ BH Length (m) | No. of flowing Features | Flowing feature frequency ($P_{10,PFL}$) (corrected) | ΣT/L (m/s) | Min T ($m^2/s$) | Max T ($m^2/s$) | Mean LogT | SdlogT |
|---|---|---|---|---|---|---|---|---|
| HRD_C | | | | | | | | |
| *50–150* | 741 | 236 | 0.564 | 2.1E-07 | 3.9E-10 | 3.8E-05 | –7.5 | 1.1 |
| *150–400* | 1,451 | 124 | 0.167 | 6.2E-08 | 3.7E-10 | 3.4E-05 | –7.9 | 1.1 |
| *400–650* | 1,655 | 68 | 0.107 | 3.4E-09 | 3.3E-10 | 1.1E-06 | –8.1 | 0.9 |
| HRD_W | | | | | | | | |
| *50–150* | 1,282 | 379 | 0.499 | 2.8E-07 | 0 | 4.6E-05 | –7.5 | 1.0 |
| *150–400* | 904 | 34 | 0.079 | 1.3E-07 | 1.1E-09 | 8.7E-05 | –7.7 | 1.6 |
| *400–650* | 677 | 23 | 0.060 | 2.8E-08 | 6.7E-10 | 9.2E-05 | –7.5 | 1.4 |
| HRD_EW007 | | | | | | | | |
| *50–150* | 279 | 107 | 0.816 | 3.1E-07 | 4.4E-10 | 3.2E-05 | –7.4 | 1.2 |
| *150–400* | 1,001 | 241 | 0.550 | 1.2E-07 | 3.1E-10 | 3.7E-05 | –7.5 | 0.9 |
| *400–650* | 843 | 72 | 0.225 | 1.2E-08 | 7.9E-10 | 1.8E-06 | –7.6 | 0.7 |




*a) Plan view of the hydraulic domains*




*b) Isometric of the hydraulic domains*

***Figure 3-4.*** *Location of Laxemar hydraulic rock domains /SKB 2009a/.*

The hydrogeological data summarised in Table 3-6 was used to estimate the transmissivities expected at 500 m depth in the direction of the deposition tunnel (parallel to the maximum horizontal stress). Table 3-7 summarises the results from those simulations for 20-m-long deposition tunnel sections for each hydraulic rock domain. These values can be used to estimate the groundwater flow into deposition tunnels in each hydraulic rock domain, and to identify the portions that may require grouting and to assess the grout quantities.

Table 3-8 provides an estimate of the distribution of transmissivity values that could be expected for vertical 8-m-long holes drilled from the 500-m-depth. These values can be used to estimate the flow into deposition holes in each hydraulic rock domain.

## 3.6 Site adaption

### 3.6.1 Repository depth

In SKB /2006b/ it was suggested that a repository in typical Scandinavian shield could be safely constructed at a depth interval between 400–700 m. At this depth range there are also several site specific factors related to long term safety that must also be considered when selecting the repository depth. An overview of these factors are provided in Table 3-9 /SER, SKB 2008a/ and SR-Can /SKB 2006d, Section 13.6.8/ describes the role each factor can play in the depth selection. The depth of the repository must, in general, balance the safety requirements for the repository and the constructability of the underground excavations required for the deposition tunnels and deposition holes. The safety requirements are largely influenced by the hydrogeology of the site, i.e. frequency and occurrence of transmissive fractures with depth while the constructability is mainly related to rock mechanics issues, i.e. stability of the deposition holes prior to emplacement.

These factors are assessed in the SER /SKB 2008a/. The main reason for placing the repository deeper would be to find rock with a low frequency of water conducting fractures, whereas most other factors favour a more shallow location. Since the hydraulic properties are assumed to be statistically the same over the depth interval 400 to 650 m, see Table 3-6, there is little reason to place the repository deeper than 500 m. It should also be noted that the statistical foundation for a much lower frequency below Elevation –650 m is weak, and this boundary may be deeper than currently assumed. In summary, the repository shall be located at elevation 500 m, or lower. This means that the roofs of the deposition tunnels shall be set at elevation 500 m or below.

### 3.6.2 Deposition tunnel alignment

SER /SKB 2008a/ concluded that if the deposition tunnels were aligned within ±30° of the trend of the maximum horizontal stress the risk of spalling will be significantly reduced. At Laxemar, the orientation of the maximum horizontal stress is 135±15 degree /SER, SKB 2008a/. Hence Design D2 optimised the layout with respect to 135±15 degree.

**Table 3-7. Distribution of transmissivity values (divided into 5 classes) for 20-m-long sections in the proposed direction of deposition tunnels at 500 m depth. The values are determined from correlated hydrogeological DFN models. The associated grouting requirements for each class are also given /Stigsson 2009/.**

| Hydraulic Domain | Transmissivity ($m^2/s$, 20 m tunnel sections) | | | | |
|---|---|---|---|---|---|
| | <1e-8 | 1e-8–8e-8 | 8e-8–2e-7 | 2e-7–1e-6 | >1e-6 |
| HRD_C | 0.42 | 0.24 | 0.14 | 0.16 | 0.04 |
| HRD_W | 0.60 | 0.15 | 0.09 | 0.10 | 0.06 |
| HRD_EW007 | 0.09 | 0.04 | 0.10 | 0.58 | 0.19 |
| Suggested Grout requirements | No Grouting Required | Silica Sol Grout | Silica Sol Grout | Cement Grout + Silica Sol | Cement Grout |

**Table 3-8. Distribution of transmissivity values (divided into 6 classes) for 8-m-long vertical sections at 500 m depth. The values are determined from correlated hydrogeological DFN models /Stigsson 2009/.**

| Hydraulic | Transmissivity ($m^2/s$, 8 m vertical sections) | | | | | |
|---|---|---|---|---|---|---|
| Domain | <4e-9 | 4e-9–3e-8 | 3e-8–2e-7 | 2e-7–5e-7 | 5e-7–1e-6 | >1e-6 |
| HRD_C | 0.69 | 0.06 | 0.11 | 0.07 | 0.02 | 0.02 |
| HRD_W | 0.78 | 0.01 | 0.05 | 0.03 | 0.02 | 0.09 |
| HRD_EW007 | 0.38 | 0.06 | 0.28 | 0.20 | 0.07 | 0.01 |

**Table 3-9. Engineering and safety factors considered for the recommendation of repository depth.**

| Engineering factors | Safety factors |
|---|---|
| **Initial temperature:** Up – lower in situ temperature favourablefor canister spacing | **Initial temperature :**Considered in design, no direct effect |
| **Water inflow, grouting efforts:** Up – lower groundwater pressure favourable. Down – if hydraulic conductivity decreases with depth | **Salinity and upconing;** Up – possibly lower inflow to facility<br>**Groundwater pressure:** Up – marginal importance |
| **Rock stability, rock stress:** Above a tentative triggering depth were stress conditions may be unfavourable for tunnelling | **Rock stress:** Above a tentative triggering depth were stress conditions may be unfavourable for long term effects around the deposition holes |
| **Available space, layout adaptation – 3D structural model:** Undecided, site specific | **3D structural model – layout adaptation, degree of utilization**<br>Site specific – fracturing, thermal properties, hydraulic properties, stability |
| **Degree of utilization – fracturing, thermal properties, inflow, stability:** Site specific | **Length and transport resistance of travel paths:**<br>Down, longer paths generally favourable |
| **Environment (short term):** Up, less excavated rock volume, less inflow (drawdown) | **Fracture frequency and Transmissivity:** Undecided, site specific |
| **Time and cost:** Up, shorter access shafts and ramp | **Inadvertent human intrusion:** Down, lower risk of intrusion, difficult to quantify |
| **Design of underground openings:** Not affected | **Freezing:** Down – reduces risk associated to permafrost<br>**Surface erosion:** No importance |

### 3.6.3 Deposition hole spacing

For design stage D2, the strategy for thermal dimensioning was based on the proposal by /Hökmark et al. 2009/. The strategy applied focus on avoiding any canister to exceed the temperature criterion 100°C in the buffer. No optimisation on canister spacing based on the thermal criterion was carried out in Design step D2. This is discussed in Section 8.2.2. The pre-requirements for the thermal dimensioning of layout D2 are constant canister spacing, maximum thermal power 1,700 W, tunnel spacing 40 m and maximum allowed peak temperature at the bentonite 100°C.

For design step D2, the centre to centre spacing between deposition tunnels was set to 40 m, and the canister spacing for the rock domains are given in Table 3-10.This spacing is selected to ensure that the highest permissible temperature in the buffer does not exceed the 100°C criterion.

**Table 3-10. Canister spacing for different rock domains at 500 m depth with deposition tunnels spaced 40-m centre to centre. From SER /SKB 2008a/.**

| Rock Domain | Canister Spacing |
|---|---|
| RSMA01 | 9 m |
| RSMM01 | 10.5 m |
| RSMD01 | 8.1 m |

### 3.6.4 Loss of deposition positions

The design premises document /SKB 2004/ identified criteria to be used for assessing the degree-of utilisation to ensure that the repository is large enough to locate 6,000 canisters. There are two primary factors that contribute to the potential loss of deposition positions /SER, SKB 2008a/:

- Loss due to the intersection with long (large) fractures that have the potential for secondary shear movement more than 5 cm. According to the Design Premises Long Term Safety /SKB 2009b/, this means that deposition holes meeting the "EFPC" criterion must not be used.
- Loss due to unacceptable water inflows.

At Laxemar it is likely that many fractures meeting the long-fracture criterion would be the same fractures that exceed the inflow criterion. There is also uncertainty in our ability to predict these long fractures at repository level based on surface mapping and core logging. As a result, for design step D2, alternative layouts were evaluated for a gross capacity of 8,031 deposition positions.

# 4 Repository facility and layout

The Final Repository Facility will consist of several functional areas: Surface facilities, Repository Access (ramps and shafts), Central Area and the Deposition Area (Figure 4-1). This chapter provides an overview of each functional area and the recommended layout for the Deposition Area.

## 4.1 Surface facility

SKB located the industrial area within the *Laxemar local model*, and specified the proposed location of the surface facilities (Figure 4-2). SKB also specified the location of the ramp and thus no optimisation has been performed regarding geotechnical conditions, because the area in this southeast part of the design target volume was found to be rather homogenous outside major deformation zones. The access to the repository is on an outcropping rock surface that is slightly higher elevated above the nearby areas. It was specified that the buildings within the surface facilities must be located a minimum distance of 200 m from the existing transmission line north of the industrial area (see Figure 4-2).

The surface facility comprises various civil structures and buildings above ground, which are required for the operation, support and supervision of the Final Repository (cf. Figure 4-1). The surface facility is connected to the underground Central Area by the four shafts (skip shaft, elevator shaft and two ventilation shafts) and a ramp. Hence the location of the Central Area is dictated by the location of the surface facility and vice-versa.

The main part of the surface facility is concentrated in an Operation Area, which is subdivided into an outer and inner area. The nuclear industrial activities are contained within the inner operation area; while other activities related to traditional operational activities are carried out in the outer operation area. An information building, ventilation stations and storage for bentonite are also included in the surface facility.

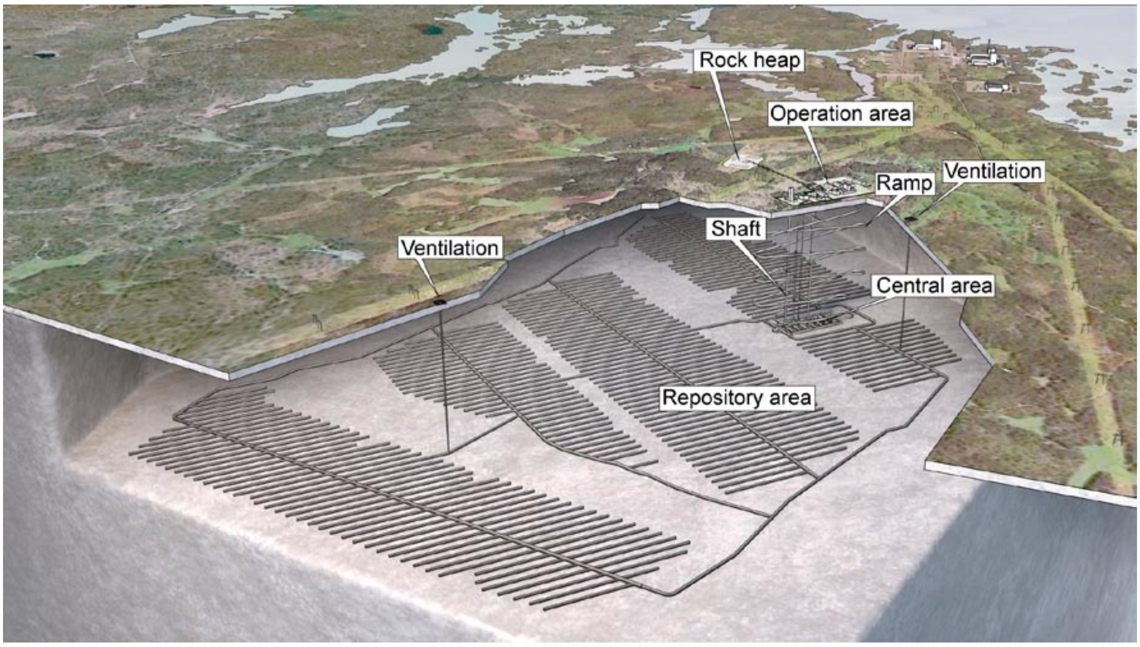



***Figure 4-1.** General view of the layout showing the location of the underground functional areas (Access, Central and Deposition Area) and the surface facilities.*

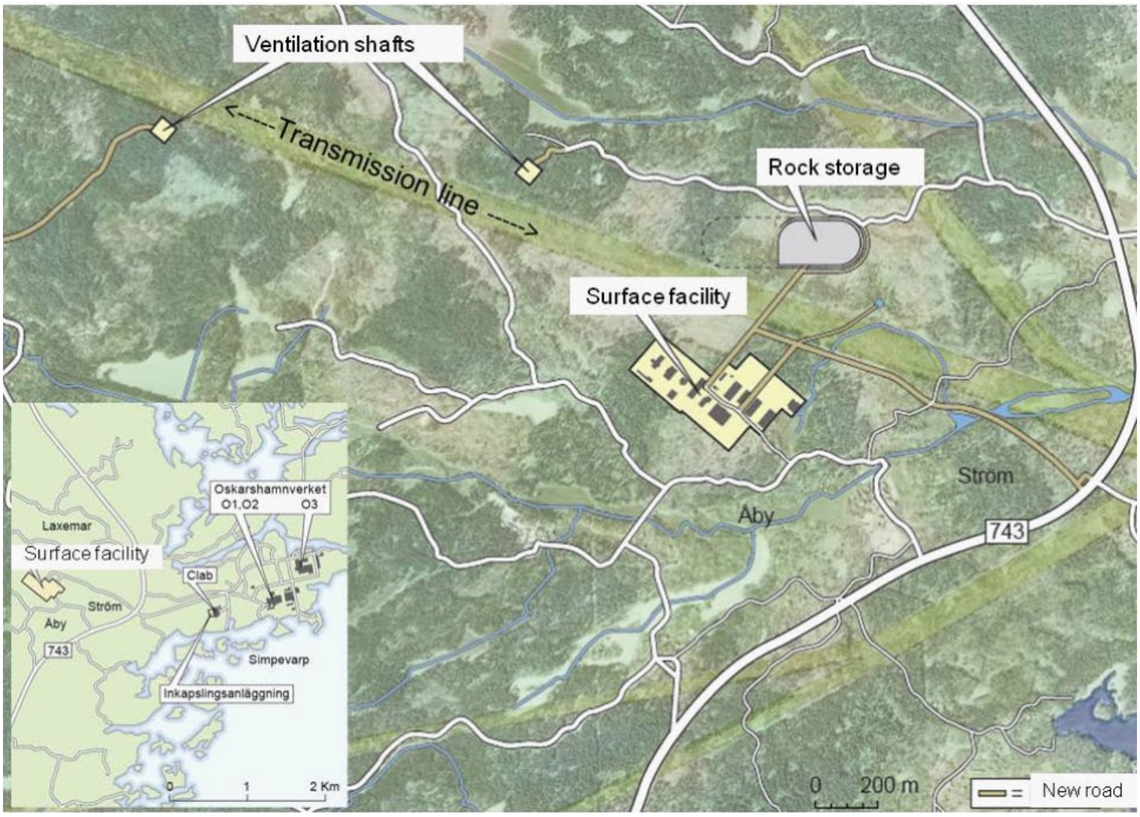



***Figure 4-2.*** *Plan view of the surface facility showing its proximity to the existing transmission line infrastructure.*

## 4.2 Repository Access

The Repository Access consists of four shafts (skip shaft, elevator shaft and two ventilation shafts) and a ramp. The excavations associated with the Repository Access are described below. The operation of the repository will require transport of containers with canisters, construction and installation material, machinery, etc through these accesses.

### 4.2.1 Ramp

The function of the ramp is to provide a transport route for vehicle traffic between the inner operation area of the Surface Facilities and the underground Central Area. The ramp will be used for transport of the canisters during operation phase. In addition, the ramp will function as a secondary escape route from the underground area as well as a secondary route for the rescue service.

The ramp, a 6 m high 5.5 m wide D-shaped tunnel, is theoretically designed as an extended spiral with inclined long sides connected with 180$^{o}$ curves at the ends (Figure 4-3). The spiral needs to do five loops at a gradient of 1:10 in order to reach the –500 m level. Minimum curve radius is set to 25 m. The total length of the ramp is approximately 5 km having a theoretical cross sectional area of 31 m$^2$. Passing locations are arranged at each 500 m.

### 4.2.2 Skip shaft

The skip shaft is the shaft, which connects the skip hall of the Central Area with the inner operation area of the surface facility (Figure 4-3). The skip shaft shall accommodate transport and handling equipment for transport of rock, buffer and backfill material. The shaft shall also have room for power-supply cables, and also a pipe for refuelling of the diesel cistern in the Central Area. The net diameter of the shaft is approximately 5.5 m.

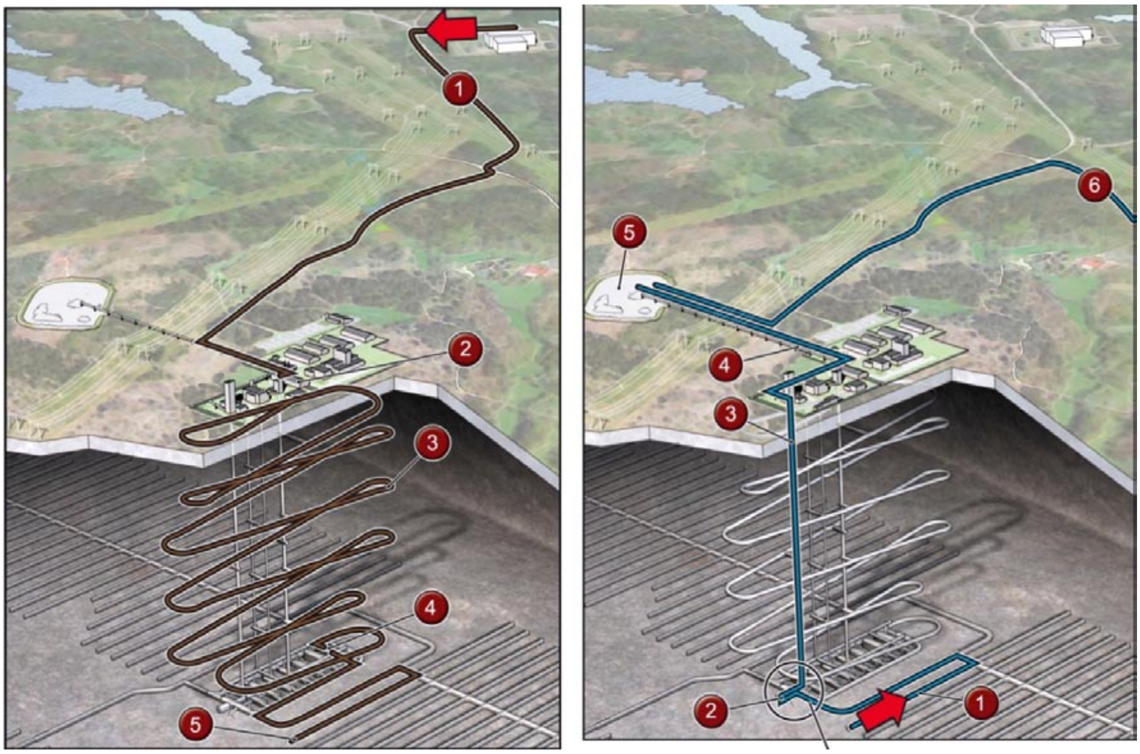



*Figure 4-3. Repository Access. The left figure shows the ramp access while the right figures shows shaft access.*

### 4.2.3 Elevator shaft

The elevator shaft provides space for two elevators for transport between the surface facility and the Central Area. During operation, the elevators will be used for transport of personnel to and from the underground facility, transport of lightweight material, and primary escape route from the Central Area, and also primary route for the Rescue service. The shaft will also be equipped for pipe installations for drainage and tapping water. The cross section of the shaft is Ø 6 m (net diameter).

### 4.2.4 Ventilation shafts

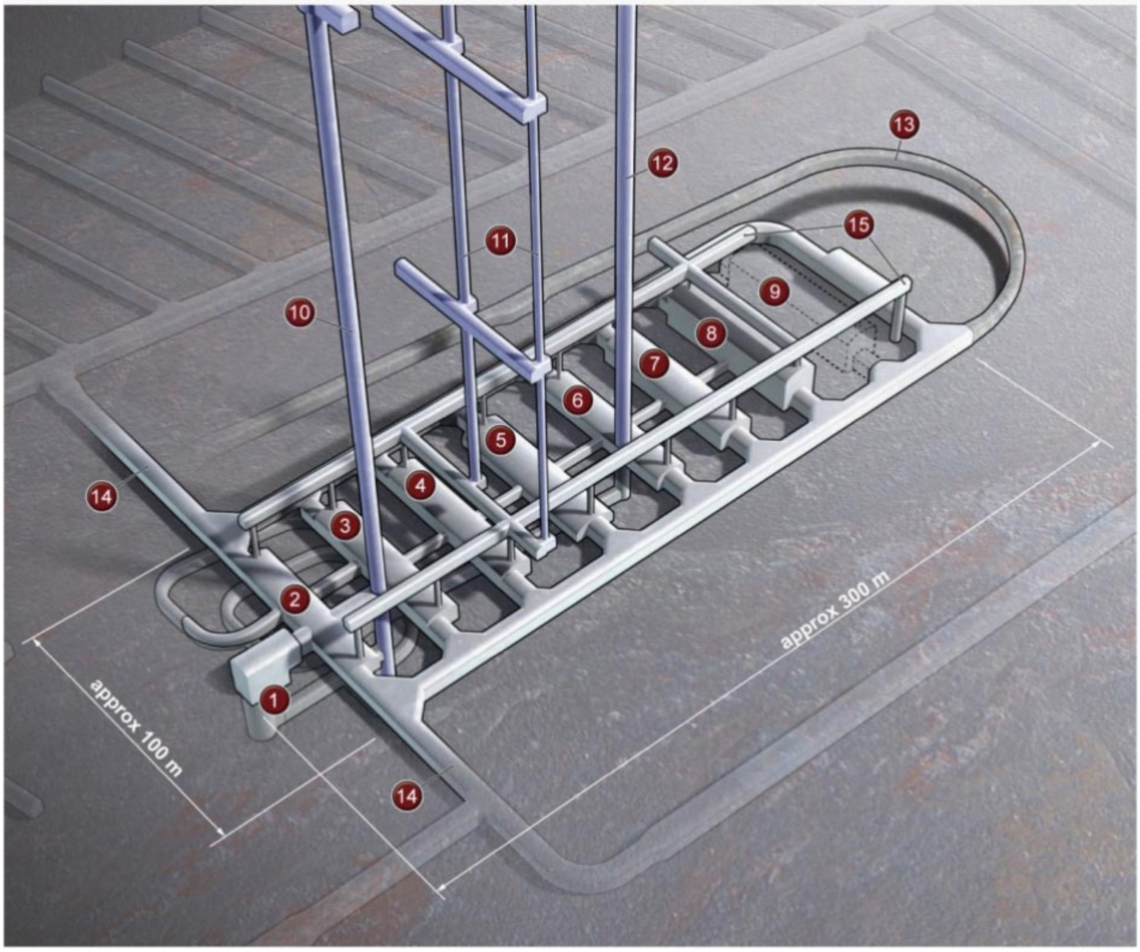
There are one fresh air intake shaft and one exhaust shaft connecting the surface to the Central Area. The cross section of the each shaft is Ø 3.5 m (net diameter).

## 4.3 Central Area

The basic function of the Central Area is to supply openings for operation and maintenance of the deposition work and the rockwork activities. The Central Area has outer and inner connections with the ramp, tunnels and shafts. The Central Area is connected with the surface facility via four vertical shafts. The rock hall and skip hall are placed nearest the Deposition Area to avoid that rock haulage is carried out within the Central Area. The rock openings and their related functions, and a general layout of the Central Area is shown in Figure 4-4.

## 4.4 Deposition Area

Site-specific strategies have been compiled for the different parts of the facility on the grounds, accounting for the overall objectives and purposes of the work based on the UDP/D2 /SKB 2007/ and the SER /SKB 2008a/. The facilities and operation are also adapted to ovoid unfavourable environmental consequences.




| Rock opening | Function | H/W/L* (m) |
|---|---|---|
| 8. Reloading hall | Reloading of canisters from ramp vehicles to deposition vehicles. Disposition of canister containers on load carrier and deposition vehicle. Maintenance of deposition vehicles. Identification and control of canisters. Monitoring of air airborne activity for control of the tightness of the canister. Testing of the cleanness of the canister. | 17/15/65 |
| 7. Store and workshop hall | Repair work and maintenance of machines and vehicles, and also store supply of construction and installation materials, mobile equipment such as drainage pumps, transforming stations, welding sets etc. | 10/15/65 |
| 6. Elevator hall | Space for personnel and visitors, lightweight materials, and also for rescue chamber and rescue vehicles. | 8/13/65 |
| 5. Vehicle hall | Parking lot for vehicles and refuel of machines and vehicles. | 9/16/65 |
| 4. Power supply hall | Equipment for power supply to all equipment in the underground facility | 9/15/65 |
| 3. Skip hall | Storage and loading of buffer and backfill re-transport of packing and loading stools. | 9/13/65 |
| 2. Rock hall | Collecting and cleaning of drainage water by sedimentation and oil deflection. Disposition of pumps. Parking lot and refuel of dumps. Water-jet installation for cleaning grouting and shotcrete equipment. Connection route to the rock loading station. | 8/13/65 |
| 1. Rock loading station | Reception of blasted rock, crushing of rock, storage of rock, loading of skip. | |

*H =Height, W=Width, L=Length

***Figure 4-4.*** *Isometric view of the Central Area and a general description of main caverns.*

### 4.4.1 Layout constraints

There were a number of guidelines provided by SER /SKB 2008a/ that constrained the layout of the Deposition Area. These are summarised below:

1. Deposition holes and deposition tunnels: The Deposition Area was to be located within rock domains RSMA01, RSMM01 and RSMD01. The deposition hole centre-to-centre spacing was 9 m (RSMA01), 10.5 m (RSMM01), and 8.1 m (RSMD01). The centre-to-centre spacing of the deposition tunnels was 40 m, and the deposition tunnels have a maximum length ≤ 300 m. The first deposition position lies at least 17.6 m from the entrance to the deposition tunnel and the last deposition position will be located 10 m from the end of the deposition tunnel. Deposition positions were not placed in deterministic deformation regardless of trace length.
2. Deformation zones: Deformation zones EW002, EW007, NW042a, NS001c, NS059a, NE107a and NE005a require respect distances, (see Figure 4-5).
3. Repository depth: The elevation of the roof of any deposition tunnels shall not be higher than Elevation –500 m.

In addition to these constraints the layout of the Deposition Area must consider the potential loss of deposition positions and this is addressed in Section 8.2.1

Furthermore, /Eriksson et al. 2009/ recommended that the deposition tunnels should be aligned parallel to the major horizontal stress (Azimuth 135 degrees) to minimise the risk for spalling in deposition holes.

### 4.4.2 Transport to/from Central Area

Four main tunnels will allow full utilisation of the available surface at the repository depth. With a maximum length for deposition tunnels of 300 m a minimum of four main tunnels will be required in order to cover the available target area with deposition tunnels. With the objective to reduce the transport work short cuts have been introduced by connecting deposition tunnels from different main tunnels and to temporary use these deposition tunnels for transport of rock and/or fill, see Figure 4-6.

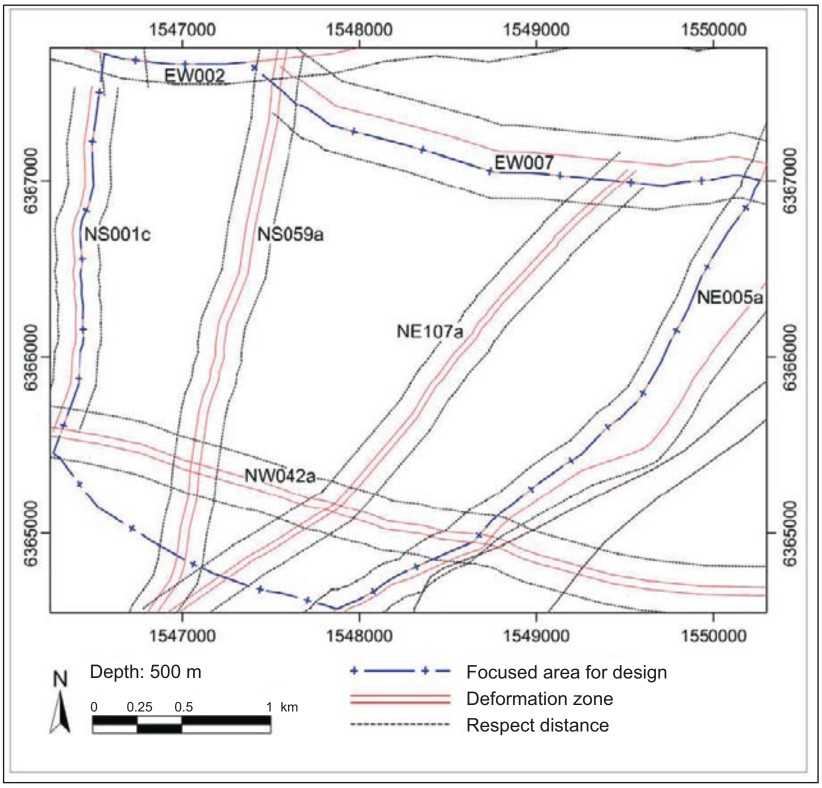



*Figure 4-5. Plan view of the 500-m level showing the location of the deformation zones requiring a respect distance.*

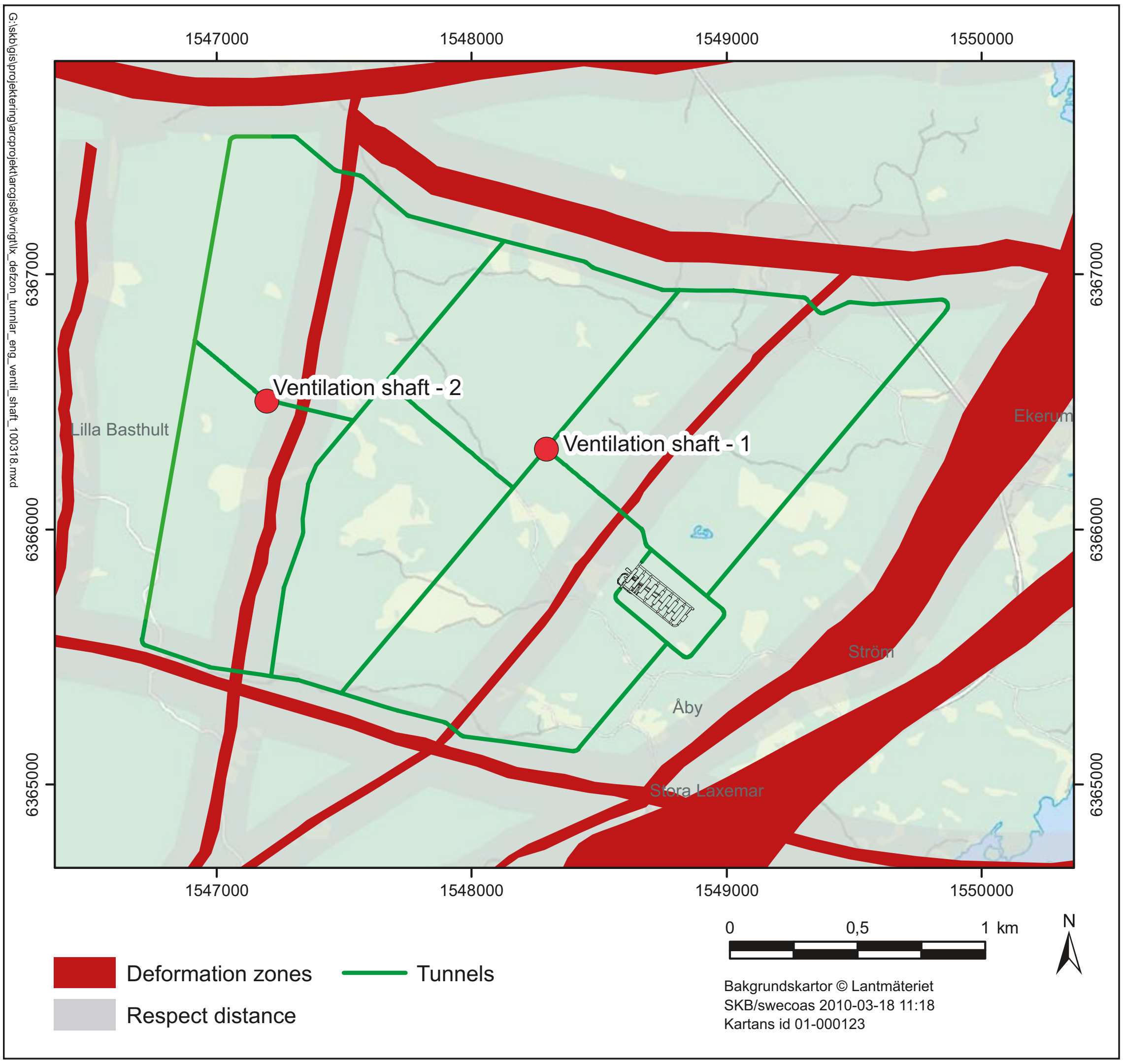



***Figure 4-6.** Position of ventilation shafts. Notice also the short transport tunnels connecting the deposition tunnels.*

### 4.4.3 Ventilation

The Deposition Area must have a sufficient number of ventilation shafts to ensure a favourable and safe working environment and to enable ventilation at the deposition level. Results from functional studies have specified positions for two ventilation shafts within the Deposition Area (Figure 4-6). The positions of the ventilation shafts are chosen from a best ventilation point of view. This means that the airflow has the highest consideration in which order the repository zones are developed. The positions of the ventilation shaft are also chosen in respect of transportation routes for the excavated rock and the transportation routes for the canisters to be deposited. Thus, the way the air flow is never to cross any transportation routes of the canisters or enter a repository zone where repository work is carried out after the air is polluted in areas where rock works are carried out.

### 4.4.4 Drainage

Drainage of the Deposition Area is arranged by means of a gravity system, where all tunnels are inclined 1:100 towards local pumping pits located in the main-/transport tunnel system (Figure 4-7). Local pumping pits are in general arranged at a distance of 1 km from each other, allowing the maximum height difference in the repository being limited to approximately 5 m. From the local pumping pits the drainage water is pumped up and on to the next section of the Deposition Area, and by gravity subsequently led further on until it reaches next pumping pit or the Central Area. At the Central Area temporary storage basins for removal of sediments and oil fragments is arranged, and

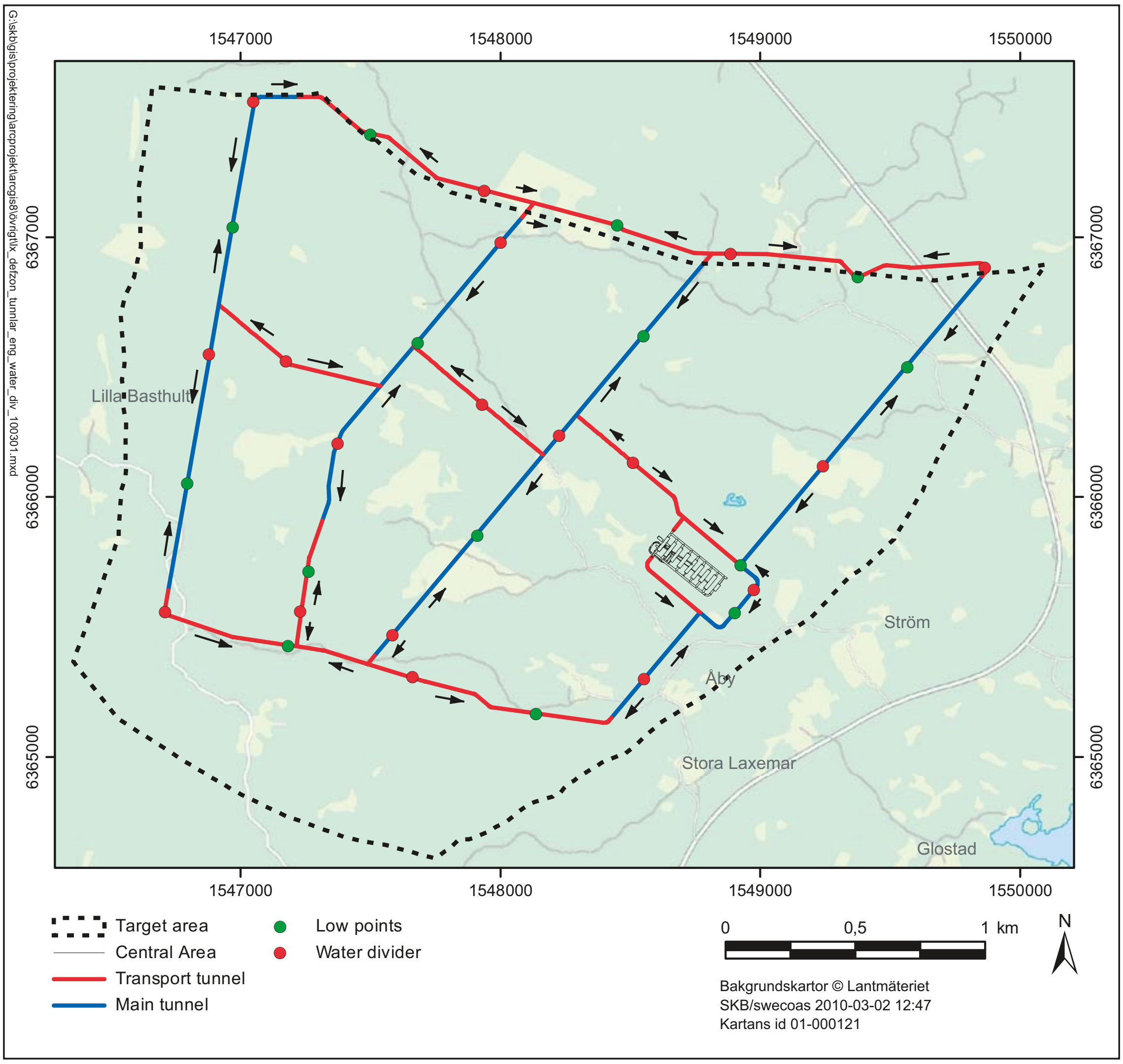



***Figure 4-7.*** *Drainage system plan for the Deposition Area. The water will run from the higher points (blue spots) to the lower points (red) where it will be pumped away.*

from these basins the water is pumped up to the surface water treatment plant. The water handling system will be designed to withstand a power cut of minimum 24 hours for the Central Area electrical system. In case of emergency as major fire, explosion, etc, jeopardizing the power supply for longer periods, an additional storage capacity for drainage water may also be arranged by an automatic overflow system leading surplus drainage water to the bottom of the skip shaft.

### 4.4.5 Rock hauling system

Excavated rock from the Deposition Area will be transported by trucks to the Central Area where it is dumped into a coarse rock crusher combined with an outlet silo for temporary storage. The silo feeds a conveyor belt leading to the hoisting skip. The conveyor belt is provided with a weighing device to define the volume/weight to be loaded into the skip. The system will be designed to work in an automatic mode and the system will be remotely controlled and supervised.

At the surface the rock material will be unloaded from the skip directly on conveyor belts and transported to the main waste-rock storage area.

## 4.5 Summary of the proposed layout

The overall strategy for the layout of the repository was to optimise the number of deposition positions taking into account the available rock volume, the geometric limitations, condition of the bedrock such as rock domains, abundance/type of fracture/fracture zones and water conditions.

### *Deposition tunnels and deposition positions*

The selected orientation of the deposition tunnels in combination with the geometry of the resulting deposition blocks controls the possible deposition tunnel layouts with limited opportunity for flexibility. To achieve efficient utilisation of the area, a strategy to orientate the 100–300 m long deposition tunnels either parallel or at right angles in relation to boundaries of the Deposition Area must be applied, although always avoid placing any deposition position inside deformation zones. Parallel orientation of the deposition tunnels coincides with the criterion that the direction of deposition tunnels should correspond with the main direction of the major horizontal stress, and in this case this was consequently the obvious strategy to select. To optimise the available Deposition Area, the possibility of locating transport tunnels within respect distance volumes to deformation zones has been considered. The studies for the Deposition Area commenced by a thorough analysis of all functions of the different element constituting the Deposition Area, leading to the following strategy for the layout design work, given in order of priority:

- To reach the required deposition capacity of 6,000 canisters, after reduction with a given loss of deposition positions due to discriminating fractures.
- To align main tunnels in principal northeast/southwest, a consequence of previous requirement since main tunnels preferable also should be perpendicular to the deposition tunnels. Due to the geometry of the selected Deposition Area and the position of the Central Area the selection of four main tunnels is the only prevailing option.
- To locate all deposition tunnels aligned with the maximum principal stress (~130°) and only to allow small deviations from that direction.
- To align the deposition tunnels parallel to the northern and southern borderlines of the deformation zones, giving higher priority to fracture domains of less hydraulic conductivity.

Having the strategy given above in mind during the preparation of the layout, alternative possible configurations of the main- and deposition tunnels gradually could be ruled out during the design process. The proposed layout is presented in Figure 4-8. Nevertheless, depending on actual principal rock stress levels obtained during construction of the repository, small modifications and adjustments of the layout might be introduced at a later stage.

### *Transport tunnels*

The Deposition Area consists in total of 104 km of tunnels, of which slightly more than 8% is in modelled deformation zones. Of the transportation tunnels are slightly more than 80% located within deformation zones or respect distances to major zones. The total length of transport tunnels within the major zones ZSMNE107A and ZSMNS059A amounts to 256 m, and within respect distances the length of the transport tunnels to such zones is 6,458 m. The Deposition Area is located outside major deformation zones > 3 km or their respect distances. A total of nine modelled deformation zones in the length interval 1–3 km (Figure 4-5), and eleven zones defined in single boreholes and modelled as discs with a standard radius of 564 m are involved in the Deposition Area. Three of these zones (ZSMEW946A, KLX07_DZ10 and KLX11_DZ11) are gently dipping with dips less than 30°. About 2% of the deposition tunnels and 2% of the main tunnels are located within gently dipping zones. To reduce the transport required for the separated construction and deposition activities is it proposed to use transport routes. This temporary transport route is preferably located central in the Deposition Area (cf. Section 4.4.2). Because most of the transport tunnels in the Deposition Area to a great extent are located within the respect distance to east-west striking major deformation zones (EW007A and NW042A), the proportion of ground types GT2 and GT3 is significantly higher than in other tunnels.

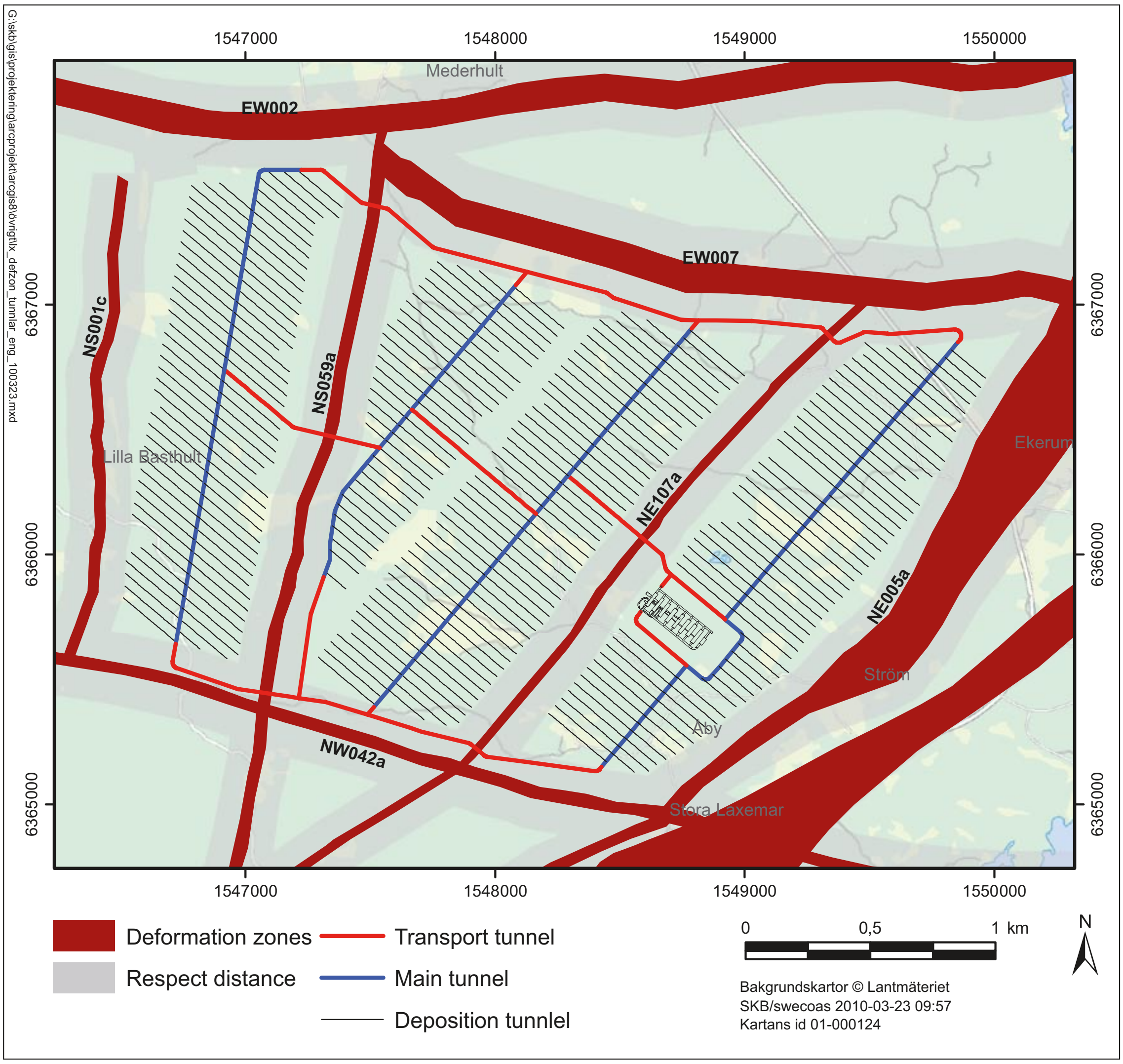



***Figure 4-8.*** *The proposed layout.*

### Main tunnels

Four main tunnels will allow full utilisation of the available surface at the repository depth. With a maximum length for deposition tunnels of ≤ 300 m a minimum of four main tunnels will be required in order to cover the available target area with deposition tunnels (cf. Section 4.4.2). The total length of the main tunnels is 8,603 m of which 7,508 m is located in ground types GT1 and GT2, and the remaining in ground types GT3 and GT4. Only a 43-m-long segment of these tunnels is located within the boundary of the respect distance for deformation zones NE107A and NS059A.

### Excavation volumes and length

The proposed layout requires excavation of various tunnels, shafts and caverns. The estimated volumes and tunnel lengths are provided in Table 4-1 for the layout shown.

**Table 4-1. Tunnel length and excavated volumes for tunnels at repository level shown in layout below.**

| | Theoretical volume ($m^3 \times 10^3$) | Length (km) |
|---|---|---|
| **Repository Access** | | |
| Ramps & shafts | 217 | 8 |
| Rock loading station | 7 | |
| **Central Area** | | |
| Caverns | 324 | |
| **Deposition Area** | | |
| Transport tunnels | 230 | 6.5 |
| Main tunnels | 515 | 8.6 |
| Deposition tunnels | 1,824 | 95 |
| Deposition holes (6,000) | 115 | 4.8 |

# 5 Repository development and operational strategy

The Deposition Area shall be developed step-wise, and the two activities construction and operation of the nuclear facility will always be separated by a physical protection that does not allow for contact between these activities. These development steps may range 2-4 years.

The operation and construction plan provides for a deposition rate of 50, 100, 150 canisters/year for the first three years of initial operation, and after that for a capacity of 150–200 canisters/year until deposition has reached 6,000 canisters.

As a basis for the repository development planning, location and duration time was allocated for each activity and all work tasks were analysed in the software Line of Balance (see details in /Leander et al. 2009/). These detailed time-studies were used to evaluate the construction capacities for the two main parallel-activities:

(1) *Rock construction works:*
Production of a deposition tunnel (maximum length approximately 300 m) varies from 113–120 ± 10 weeks depending on which hydraulic rock domain the deposition tunnels are driven in and the extent of grouting that is needed. *(Time includes investigation core drilling, grouting, gallery+bench excavation, installations, geological mapping, TBM-drilling, preparation of deposition hole floor, prep. of foundation for concrete plug, cleaning etc.)*

(2) *Deposition works:*
Deposition, backfill and construction of a concrete plug 32 ± 5 weeks *(Time includes final control, maintenance of installations, assembly of buffer, deposition of canister, backfilling, concreting of plug, etc.)*

## 5.1 Construction strategy

The main objective for the reference design was to separate the two main parallel-activities by conducting them in different tunnels. Two construction methods were evaluated that would meet this objective: (1) separation by side-change and (2) separation by linear-development. These methods are briefly described below.

### 5.1.1 Separation by side-change method

The construction strategy *Separation by side-change* simply requires the excavation and deposition activities to alternate sides as a deposition panel is excavated and filled (Figure 5-1). This requires that these two activities be tightly controlled to maintain production efficiency.

The functional studies that have been carried out for Laxemar show that there are two main disadvantages with the side-change strategy:

1. there must be at least three main tunnels available for side-change at a time, and
2. the reduction in construction efficiency as the rock excavation changes from one side to the other, see Figure 5-1. When the last canister has been deposited before the side-change all backfill works (belonging to the deposition works) in that deposition tunnel remains, which will need approximately 24 weeks to complete. If the deposition of canisters immediately continues at the other side, this would requires the excavation work to stop for a minimum of 24 weeks at both sides, since deposition work would be ongoing on both sides.

In Laxemar there are only two main tunnels available when the deposition of the first canister takes place. Hence this constrain means that the separation by side-change method cannot be effectively used in Laxemar. An alternative to side-change construction strategy, *Separation by linear development method* is described in the next section.

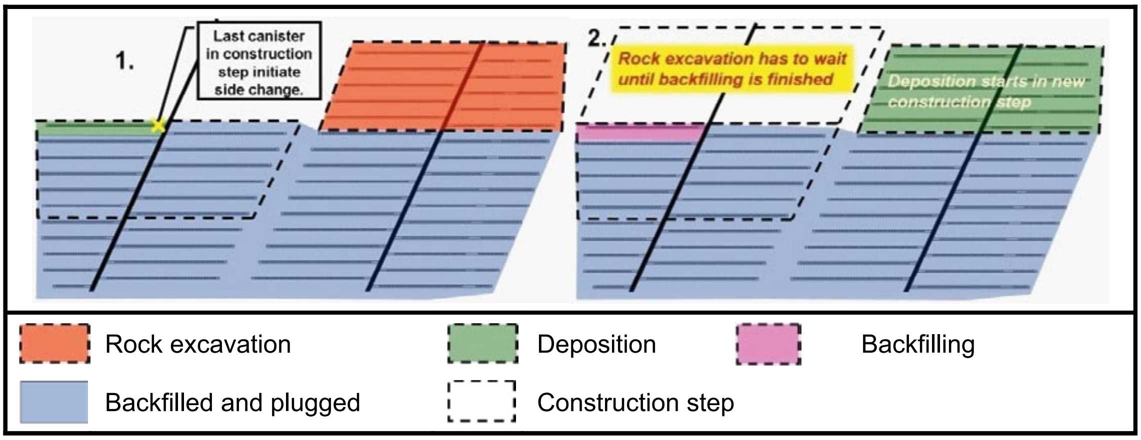



***Figure 5-1.*** *Simplified example of reduction in operational efficiency related to the side-change method /Hansson et al. 2008/.*

### 5.1.2 Separation by linear-development method

The basic concept of the "Separation by linear development method" is that rock excavation and deposition works initially progress in series, i.e. one following the other, but then progress in parallel without any need for alternating sides. It is proposed that both activities take place from a single main tunnel, as illustrated in Figure 5-2 and be separated by a barrier (wall/door).

Figure 5-3 illustrates a detailed-sequence that could be implemented using the linear-development method. The sequence will start with the construction of 14 deposition tunnels (7 on each side of the main tunnel). A separating door/wall is installed in the main tunnel and a safety distance of 80 m (2 deposition tunnels on each side, referred to as "Protection zone" in Figure 5-2) is maintained from the blasting in the next excavation phase. The excavation then continues forward in the main tunnel beyond the *safety-tunnels*, while deposition begins in the 10 deposition tunnels (5 on each side) from the preceding phase. When the first round of deposition is completed a new door/wall is installed in the main tunnel so that the four newly constructed deposition tunnels compose a safety distance during the next phase. The construction continues beyond the safety distance tunnels, at the same time as deposition begins in the completed excavation. The backfilling work now begins in the deposition tunnels that contain the canisters. These procedures allow the rock excavation, deposition and backfilling to advance along the main tunnel without any pauses for side-change or other interruptions. Neither is there any interruption when changing to another main tunnel.

The constructed barriers will have a standard to allow them to be a part of the fire-cell sectioning, see Section 5.3.3.

## 5.2 Strategy for step-wise excavation/operation

A tentative plan for the construction of Laxemar underground facility has been developed assuming time-steps of approximately 2 years /Leander et al. 2009/. The different development steps including main ventilation paths and transport routes for the two main activities are presented below. The presentation starts (year zero) with the deposition of the first canister when initial construction is completed, i.e. when the first loop of main/transport tunnels, one of the external ventilation shafts and some 10 deposition tunnels have been constructed (cf. Figure 5-4). A proposed sequencing for the step-wise development of the repository is given in Figure 5-5. A figure sequence for a detailed development for up to 40 years is given in Appendix A.

The proposed strategy for development of the Deposition Area is to start with the area south of the Central Area and from there, to go westward. The last part that will be developed is the area north of the Central Area. The motive for this strategy is that the thermal and hydraulic properties in the host rock in the southern and western parts are more suitable compared to the hydraulic rock domain in the north, HRD_EW007. To reduce the transport work required for excavation and backfill, short transport tunnels have been utilized in central portion of the Deposition Area to connect the main tunnels.

The proposed development strategy (Figure 5-5) allows for incorporation of site investigation plans that will ensure maximum utilisation. Establishing the spatial position of the geological boundaries/features that constrain the layout must precede this development. One of the first investigations will be to determine the location of the deformation zones NW042a and NE107. Gradually, as geological information is collected for the deformation zone locations and properties, as well as other site information, the layout plans can be fully developed. This layout must be developed to meet the needs of long term safety, as well as construction and operational efficiency. A formal methodology will be established for developing these layout plans based on the principles of the Observational Method (Section 2.1.2).

Each construction step as given in Appendix A comprise the following volumes (excluding deposition holes), given in theoretical in situ volume ($m^3 \times 10^3$), (see Table 5-1 below).

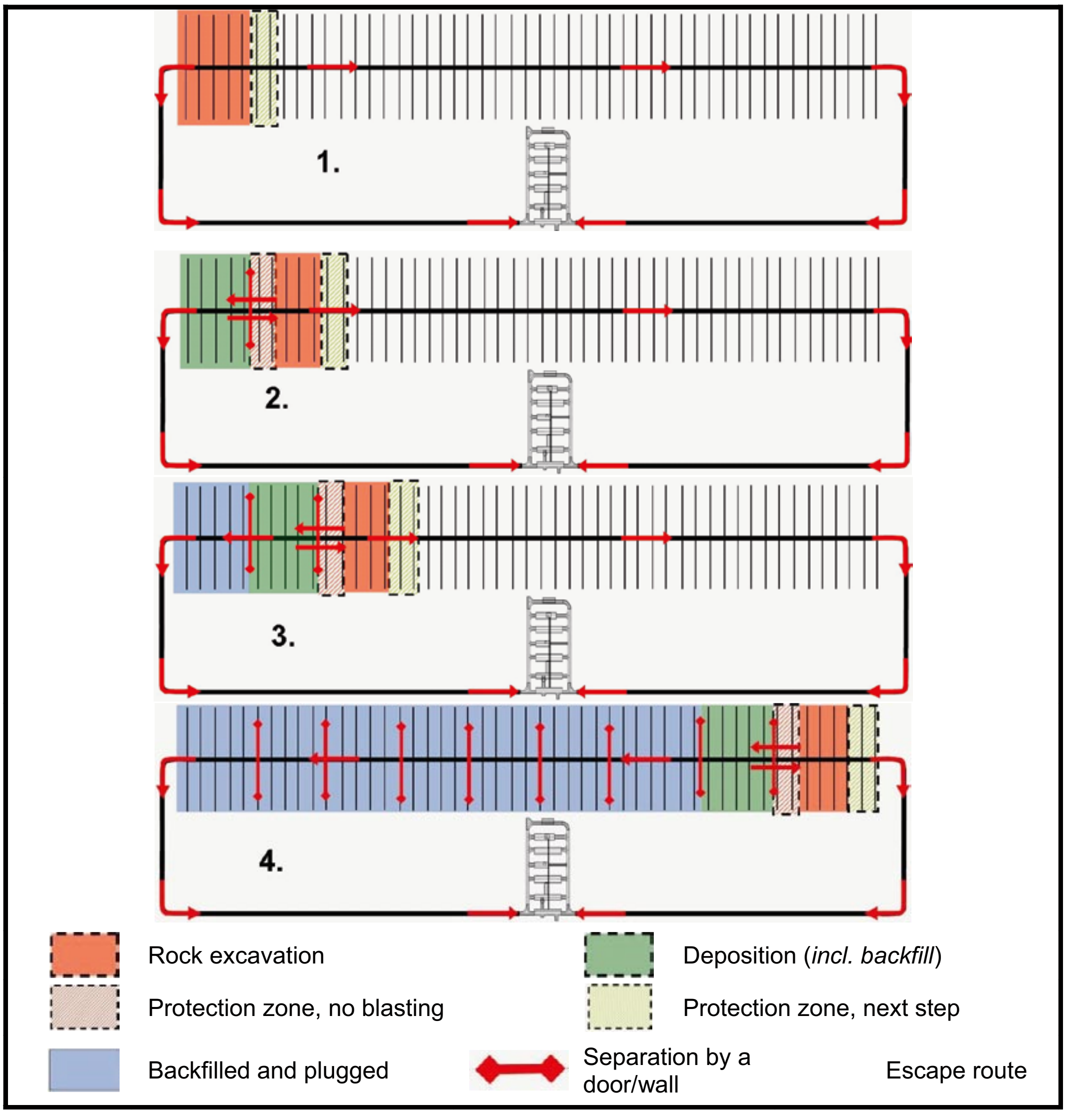



***Figure 5-2.*** *Schematic view of construction of the repository using the "Separation by linear development method". This method ensures that two evacuation paths are always available /Hanson et al. 2008/.*


*Figure 5-3. Outline of construction steps using the linear development construction method.*

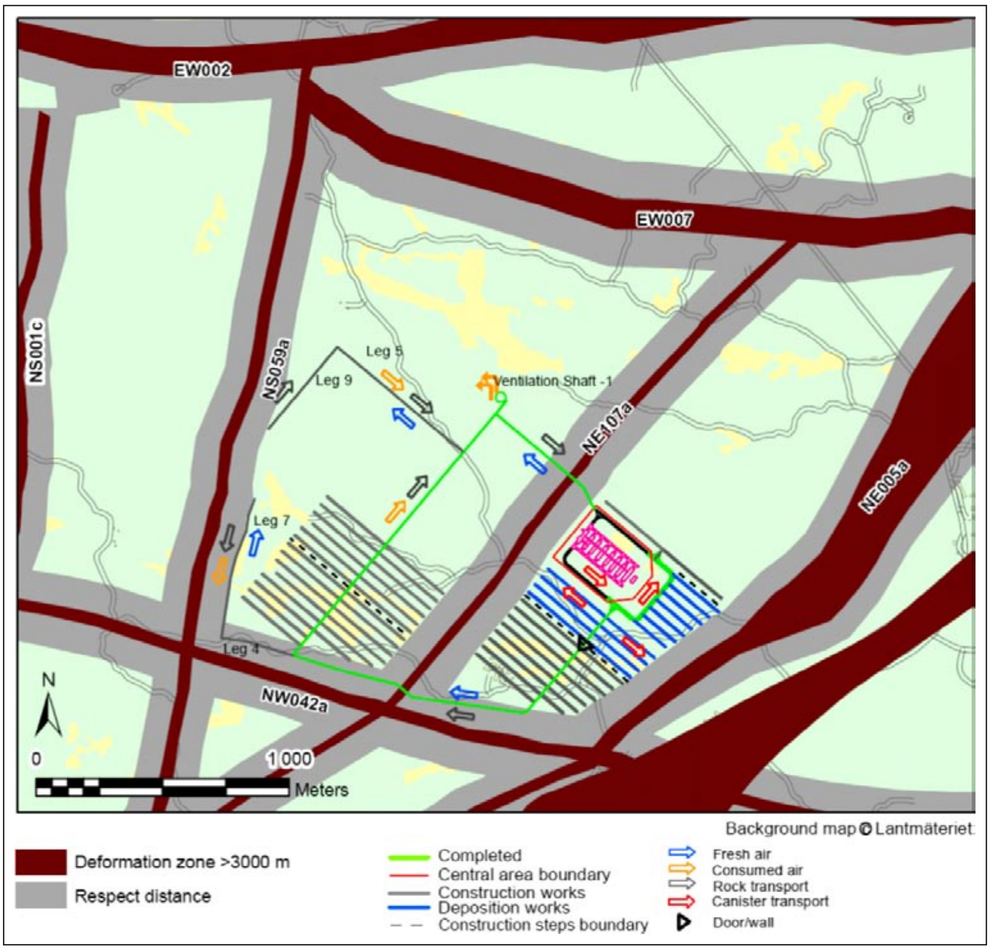



*Figure 5-4. Location for the proposed excavation, year 0.*

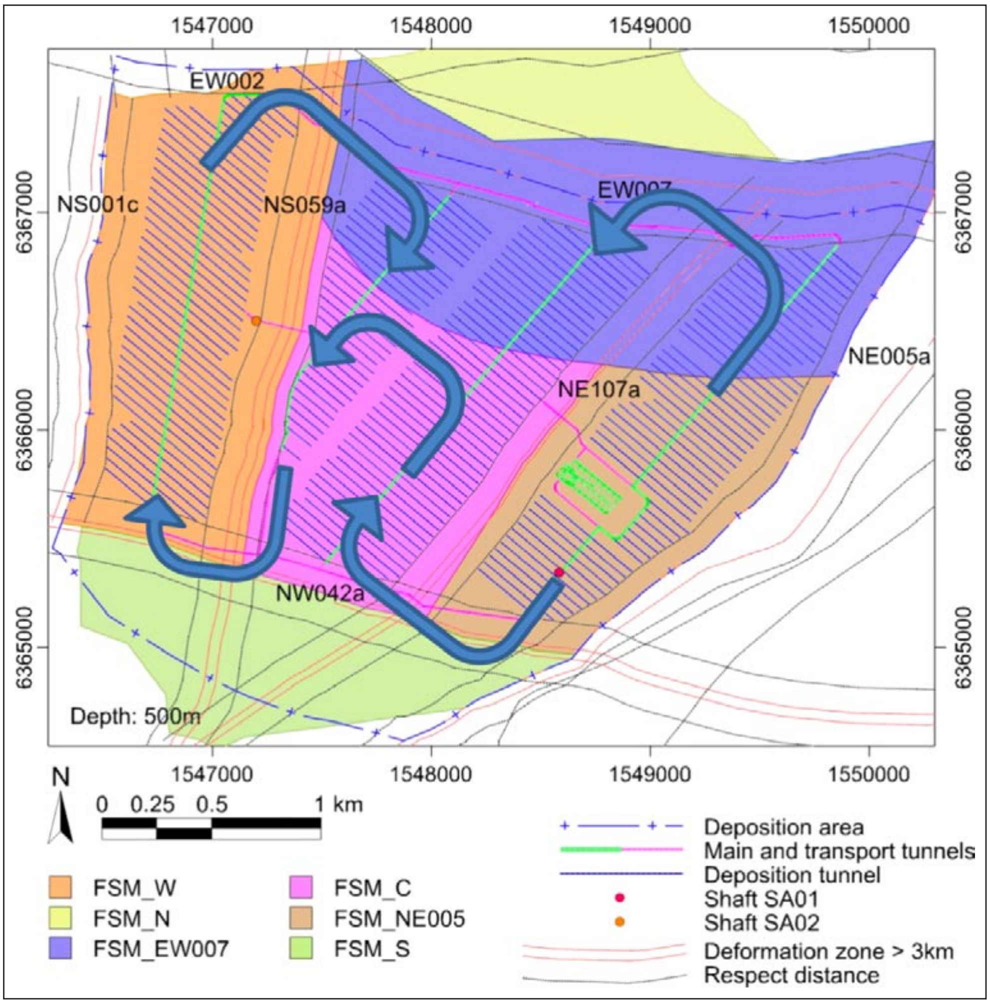



***Figure 5-5.*** *The proposed sequencing development of the repository.*

**Table 5-1. Volumes of tunnel excavation for each construction step.**

| | Year | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Type of tunnel | **6** | **10** | **14** | **18** | **22** | **26** | **30** | **34** | **40** | **Total** |
| Transport ($10^3$ $m^3$) | 52 | 37 | 45 | 0 | 44 | 0 | 52 | 0 | 0 | 230 |
| Main ($10^3$ $m^3$) | 147 | 65 | 0 | 80 | 0 | 97 | 126 | 0 | 0 | 515 |
| Deposition ($10^3$ $m^3$) | 76 | 220 | 182 | 192 | 161 | 288 | 192 | 200 | 313 | 1,824 |
| Total ($10^3$ $m^3$) | 275 | 323 | 227 | 272 | 205 | 385 | 370 | 200 | 313 | 2,569 |

A diagram of the required input of excavation for the various tunnel types, and as function of the construction steps given above in Table 5-1, is illustrated in Figure 5-6. It is assumed that when a new area is opened up and the new transport- and main tunnels are developed, that these parts most probably will be constructed by external construction companies, while deposition tunnels mainly will be constructed by SKB employed personnel. It is consequently advantageous if production volumes for deposition tunnels are evenly distributed through the years, which is not really the case in Figure 5-6. However, during Detailed Design of the repository production rates easily can be controlled, and production rates adjusted to available resources.

The total length of excavated tunnels is given in Table 5-2.

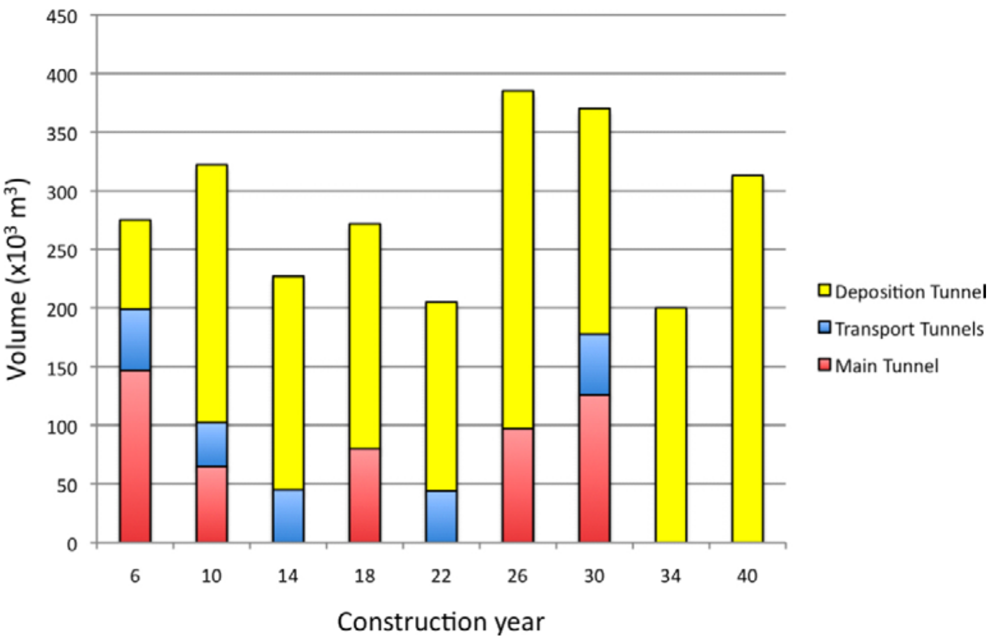



***Figure 5-6.*** *Estimated excavated volumes associated with the Main, Transport and Deposition tunnels required for the development of the repository layout given in Figure 5-5.*

**Table 5-2. Total length of tunnel types.**

| Type of tunnel | Tunnel length (km) |
|---|---|
| Transport tunnels | 6,5 |
| Main tunnels | 8,6 |
| Deposition tunnels | 95,0 |

## 5.3 Health and safety

The operations of the repository must satisfy Swedish regulations for underground construction. An overall preliminary risk assessment of these operations has been carried out following the requirements in AFS 2003:2 /Arbetsmiljöverket 2003/. The detailed results are given in /Leander et al. 2009/.

The issue of safety related to radioactivity is considered outside the scope of this report, and is also presumed not to influence the ventilation system (no air-borne activity can occur).

### 5.3.1 Escape routes

The Separation by linear-development method will facilitate deposition and back-filling immediately after rock excavation. A prerequisite of the method is that the transport of excavated rock can be made via a separate transport route that is not in use for transportation of canisters or backfilling material. This prerequisite also ensures that two separate exits are always accessible. The outline of this extension strategy is illustrated by Figure 5-2 and Figure 5-5, as well as in Appendix A.

### 5.3.2 Ventilation system

Fresh air to the repository will be provided from the surface ventilation building located just above the ventilation shafts to the Central Area. All equipment for the fresh air system is located in this ventilation building, and the distribution of air is arranged from a pressure chamber, designed as a small tunnel just above roof level of the Central Area caverns, and connected to the fresh air ventilation shaft. The surplus air pressure is then distributed in small shafts to each hall of the Central

Area, to the lower part of the ramp and to the starting point of the main- and transport tunnelling system. No ventilation ducts for pressurised inlet air is assumed to be necessary along main- and transport tunnels, which as such will be used as canals to transport the fresh air to the actual part of the repository where it is required for ongoing activities. Consequently a (surplus air-pressure) circulating system from inlet to outlet of ventilation will be arranged for each construction phase of the repository. As a general rule ventilation can pass from an area with deposition works and on into a construction area for rock works, but not in the other direction.

Outlet of exhaust air is arranged via three different routes:

1. Through the ramp using ramp ventilation.
2. Through the exhaust ventilation airshaft at the Central Area for ventilation of the Central Area.
3. Through external ventilation shafts for the ventilation of the Deposition Area.

The ventilation of the ongoing activities in the Deposition Area is arranged by air supply from the Central Area, and ventilation ducts will only be needed for short periods to ventilate dead-end new tunnels when a new depositions area is developed. This will require a higher air-pressure close to the Central Area and a lower pressure close to the external ventilation shafts, where exhaust ventilation fans will be arranged at repository level. However, to provide fresh air to the inner ends of the deposition tunnels, a local small temporary fan will be arranged at each deposition tunnel during the time the tunnel is in operation.

### 5.3.3 Fire-fighting system

The fire-fighting protection system for the repository will be designed as a combination of the following requirements:

- Operation regulations comprising ample control with the objective to reduce flammable materials brought down to the repository, compulsory fire-extinguishers for every vehicle, compulsory tracking system and personal rescue device for each individual visiting the repository, limitation of the total number of workers/visitors day by day, set up of a local rescue team, training of the operation personnel, etc.
- Fire-cell sectioning of the repository including arrangement of sliding fire proof doors and fire protection shut off valves for the ventilation, making it possible to close off and contain sections with fire. Evacuation of smoke is arranged by using the ordinary exhaust air paths, but with the possibility to reverse fans if necessary due to ongoing rescue actions.
- Arrangement of automatic fire-extinguishing equipment at places with potential risk for fire.
- Arrangement of warning systems and safe places for evacuation of personnel. The prime safe place will be arranged at the Central Area adjacent to the elevator shaft, which also will service as the main access for rescue. The shaft and the safe area will be provided with surplus air-pressure to protect it from smoke.
- Provision of multiple rescue passageways from all facilities in the repository, and for the cases where this will not be possible (such as inside deposition tunnels) arrangement of rescue chambers.

The system for smoke-evacuation in case of fire is presently studied, and the design features will be outlined in the next design phase.

## 5.4 Summary

The conclusion for Laxemar is that of the two development strategies, separation by side-change and separation by linear development method only the separation by linear development method is applicable. The linear development method provides a safe excavation and deposition environment that can meet the required deposition rates. The evaluation of the time-line studies indicates that there is ample time to dispose of the 6,000 canisters over a 40 year operating period.

# 6 Ground behaviour and support

One of the primary objectives of the underground design was to evaluate the stability of the various openings required for each of the functional areas: Repository Access, Central Area and Deposition Area. In hard crystalline rock such as those encountered in the Scandinavian shield, experience has shown that the most common forms of ground behaviour causing tunnel instabilities are:

1. Structurally (discontinuity)-controlled gravity-induced falls-of-ground, and
2. Stress-induced spalling.

A complete description of these ground behaviours are given in /Martin and Christiansson 2002/ and /Palmstrom and Stille 2007/ and summarised in SER /SKB 2008a/.

While the structurally controlled failure is prevalent at shallow depths, i.e. low in situ stress magnitudes, and the spalling failure is commonly observed at great depth, i.e. high in situ stress magnitudes, mining and tunnelling experience shows that these failure processes can be found at essentially any depth. To assess the ground control and support required for each functional area in the reference design /Eriksson et al. 2009/, using the tunnels profiles given in Figure 6-1, have:

- Assessed the range of ground behaviour without considering the effects from support measures or sequential excavations.
- Assessed the range of system behaviour based on interaction between ground types, support measures and construction measures.
- Determined the appropriate support measures based on the assessment of ground behaviour and the requirements of each functional area.

## 6.1 Analysis of the system behaviour

System behaviour refers to the interaction between reinforcement and the rock mass. The intention is to show that the system is stable, i.e. that the proposed reinforcement will work in relation to ground behaviour. The analysis is carried out for (a) the most probable system behaviour, and (b) the most unfavourable system behaviour.

In accordance with the UDP/D2 /SKB 2007/, analyses should be applied in rock reinforcement design work to verify the system behaviour, i.e. the interaction between the ground behaviour of the construction measures. Three methods are applied for analyses:

- Experience from comparable excavations.
- The Q-system.
- Analytical calculations of load-bearing capacity for rock reinforcement.
- Numerical simulations of intersections (main tunnel/deposition tunnel and deposition tunnel/deposition holes) using 2D and 3D elastic models.

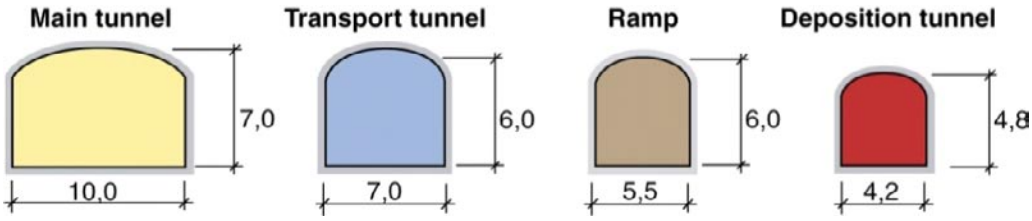



*Figure 6-1. Profiles and dimensions of tunnels used for Repository Access and deposition.*

The system behaviour was analysed using experiences from different projects in Oskarhamn area, as summarised in /Carlsson and Christiansson 2007/. In the upper parts of the repository (i.e. the uppermost parts of the shafts and access ramp) down to about 40–50 m depth, the comparison is generally based on reinforcement experience from three major construction projects in the area: the Oskarshamn Nuclear Power Station, including an underground storage for medium and low grade radioactive waste, the Central Interim Storage Facility for Spent Nuclear Fuel (CLAB) and the Äspö HRL. In the deeper parts, the construction experiences in the area are limited to the Äspö HRL, where the tunnel continues down to 450 m depth. Reinforcement solutions at repository depth are further verified by individual analytical calculations.

Two stress models have been proposed for the site and these are summarised in Table 3-5. These stress models were used in the assessment of the potential ground behaviour. The various combinations of ground types and ground behaviour expected to occur in the repository are GT1–GB1, GT2–GB1, GT2–GB3B, GT3–GB1 and GT4–GB3B (Table 6-1).

Table 6-2 provides a summary of the distribution of ground behaviour for various parts of the repository. The occurrence of GB3B at repository depth is restricted to tunnels crosscut by gently dipping zones (< 30°), as well as tunnel intersections of deformation zone NE107A. At more shallow levels irrespective of fracture domain, GB3B is expected to occur in 10% of GT2, and under the least favourable conditions in 30% of GT2. The affected facility parts are the ramp, including passing places and niches (mainly sumps and ventilation connections) along the ramp, as well as shafts that reach the surface. The lower limit for the occurrence of GB3B outside gently dipping deformation zones and NE107A was set to –477 m, i.e. to lower end of the main ventilation shafts of the Central Area. In the ramp, the lower limit was set where the spring line passes –477 m. All other facility parts are expected to consist of GB1, if they are located outside gently dipping deformation zones and NE107A.

At the repository level, it is assumed that all occurrences of GT4 belong to ground behaviour category GB3B. The distribution is identical both in the expected and most unfavourable case. Of the total length of main and transport tunnels, less than 2% are assessed as belonging to GB3B. In addition, there are 208 m GB3B in the deposition tunnels.

At the repository level, it is assumed that all occurrences of GT4 belong to ground behaviour category GB3B. The distribution is identical both in the expected and most unfavourable case. Of the total length of main and transport tunnels, less than 2% are assessed as belonging to GB3B. In addition, there are 208 m GB3B in the deposition tunnels.

### 6.1.1 Repository Access

The primary Repository Access will consist of a ramp excavated with the dimensions and profile given in Figure 6-1 at an approximate grade of 10%. The long legs of the ramp are oriented northwest-southeast, which means that long portions of the ramp will be approximately parallel to the maximum horizontal stress. The stress concentrations on the ramp boundary resulting from this orientation are not expected to induce spalling for the "most likely" or "possible maximum" stress model.

The distribution of expected Ground Behaviour for the Repository Access is summarised in Table 6-2. For the most "likely stress" model the Ground Behaviour is expected to be dominated by gravity-induced structurally-controlled block-falls (wedges).

### 6.1.2 Central Area

The Central Area consists of nine (9) 13–16 m wide rock caverns of different dimensions, as well as minor caverns, various tunnels, shaft and pits, designed to facilitate various activities (cf. Section 4.3). Two-dimensional elastic stress analyses were carried out to evaluate the stress concentrations on the boundary of the caverns. Because of the stress reduction caused by the end-effects these two dimensional analyses may be considered conservative. Figure 4-4 gives the general layout and description of the major caverns in the Central Area and Figure 6-3 shows the two-dimensional section, taken perpendicular to the length axes of the caverns that were used in the stress analyses.

**Table 6-1. The anticipated ground types and resulting ground behaviours using the ”most-likley” stress model.**

| GT–GB | Description |
|---|---|
| GT1–GB1 | Sparsely fractured, isotropic rock with gravity driven, mostly discontinuity controlled failures (block falls). |
| GT2–GB1 | Blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls). Water-bearing fractures occur, especially in MDZ <30°. |
| GT2–GB3B | Blocky rock mass with possible water assisted block falls, especially in fractures with soft mineral filling. |
| GT3–GB1 | Sealed fracture network. If reactivated it may result in blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls). |
| GT4–GB3B | Very blocky rock mass, locally in combination with significant water transmission, resulting in unstable conditions. |

**Table 6-2. Expected distribution of Ground Behaviour expressed as underground opening length (m) in each of the functional areas. The distributions are determined for the “most likely” and “possible maximum” stress model.**

| | Most likely stress | | Possible maximum stress | |
|---|---|---|---|---|
| | GB1 [m] | GB3B [m] | GB1 [m] | GB3B [m] |
| Ramp | | | | |
| Tunnel (5.5 m wide)[1] | 4,396 | 153 | 4,093 | 456 |
| Tunnel (6.0 m wide) | 114 | 9 | 97 | 26 |
| Tunnel (6.5 m wide) | 664 | 43 | 600 | 107 |
| Tunnel (7.0 m wide) | 191 | 39 | 177 | 53 |
| Passing places and niche (8.0 m wide) | 205 | 6 | 192 | 19 |
| Niche (10.0 m wide) | 23 | 1 | 22 | 2 |
| **Ventilation** | | | | |
| Shaft (ø 1.5 m) | 252 | – | 252 | – |
| Shaft (ø 2.5 m) | 475 | 15 | 446 | 44 |
| Shaft (ø 3.5 m) | 475 | 15 | 446 | 44 |
| Shaft (ø 4.5 m) | 25 | – | 25 | – |
| Tunnel (4.0 m wide) | 802 | 5 | 792 | 15 |
| Tunnel (8.0 m wide) | 31 | 1 | 29 | 3 |
| **Central Area** | | | | |
| Skip shaft (ø 5.0 m) | 475 | 15 | 446 | 44 |
| Elevator shaft (ø 6.0 m) | 475 | 15 | 446 | 44 |
| Silo (ø 9.5 m) | 22 | – | 22 | – |
| Tunnel (3.0 m wide) | 134 | – | 134 | – |
| Tunnel (4.0 m wide) | 533 | – | 533 | – |
| Tunnel (5.0 m wide) | 47 | – | 47 | – |
| Tunnel (7.0 m wide) | 933 | – | 933 | – |
| Halls (13.0 m wide) $n = 5$ | 290 | – | 290 | – |
| Halls (15.0 m wide) $n = 3$ | 186 | – | 186 | – |
| Crushing hall (10.0 m wide) | 22 | – | 22 | – |
| Vehicle hall (16.0 m wide) | 65 | – | 65 | – |
| Service hall (12 m wide) | 20 | – | 20 | – |
| **Deposition Area** | | | | |
| Ventilation shafts SA01 and SA02 (ø 3.0 m) | 1,002 | 27 | 952 | 77 |
| Main tunnel (10.0 m wide) | 7,866 | 15 | 7,866 | 15 |
| Transport tunnel (7.0 m wide) | 6,807 | 45 | 6,807 | 45 |
| Deposition tunnel (4.2 m wide) | 89,200 | 208 | 89,200 | 208 |

[1] Includes also transitions to wider tunnel sections and one nisch.

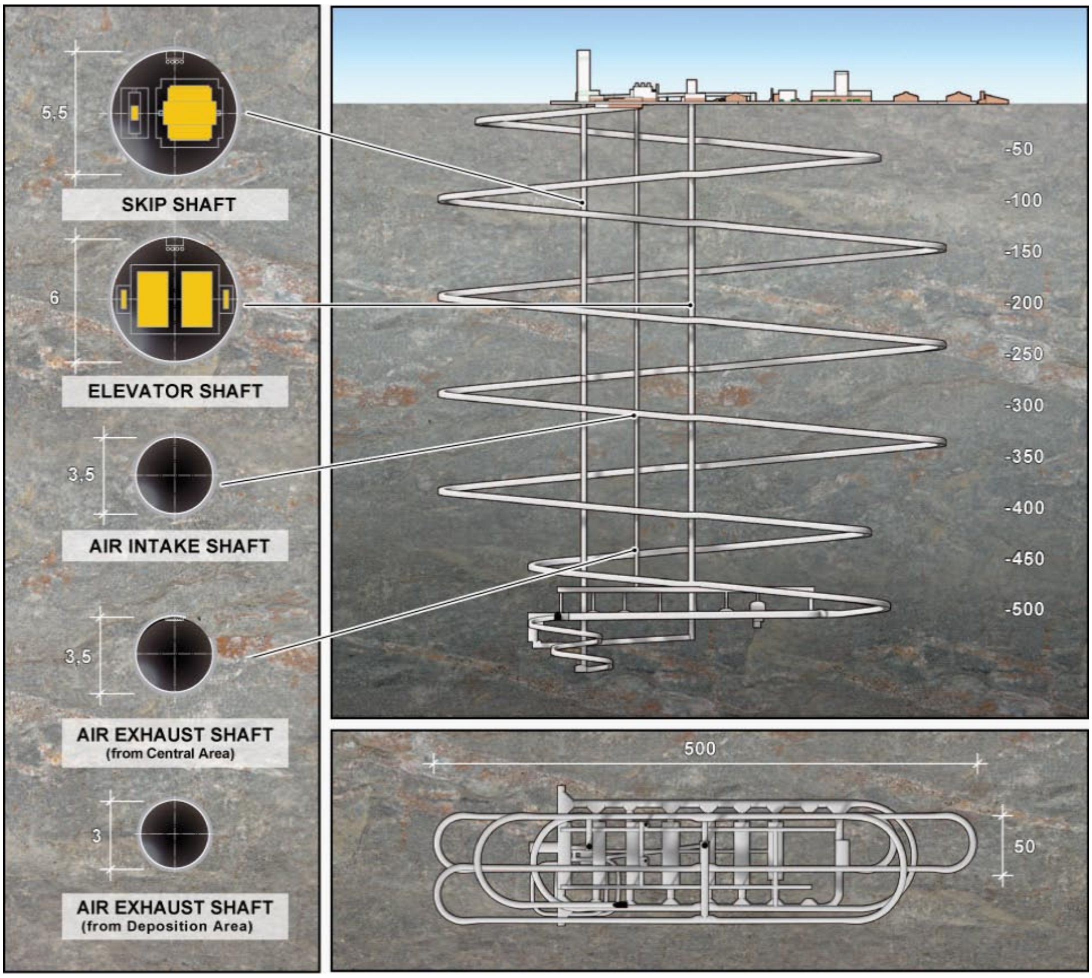



***Figure 6-2.*** *Vertical cross-section through the ramp and shaft excavations used for the Repository Access. The ramp will be excavated at an approximate grade of 10% (1:10).*

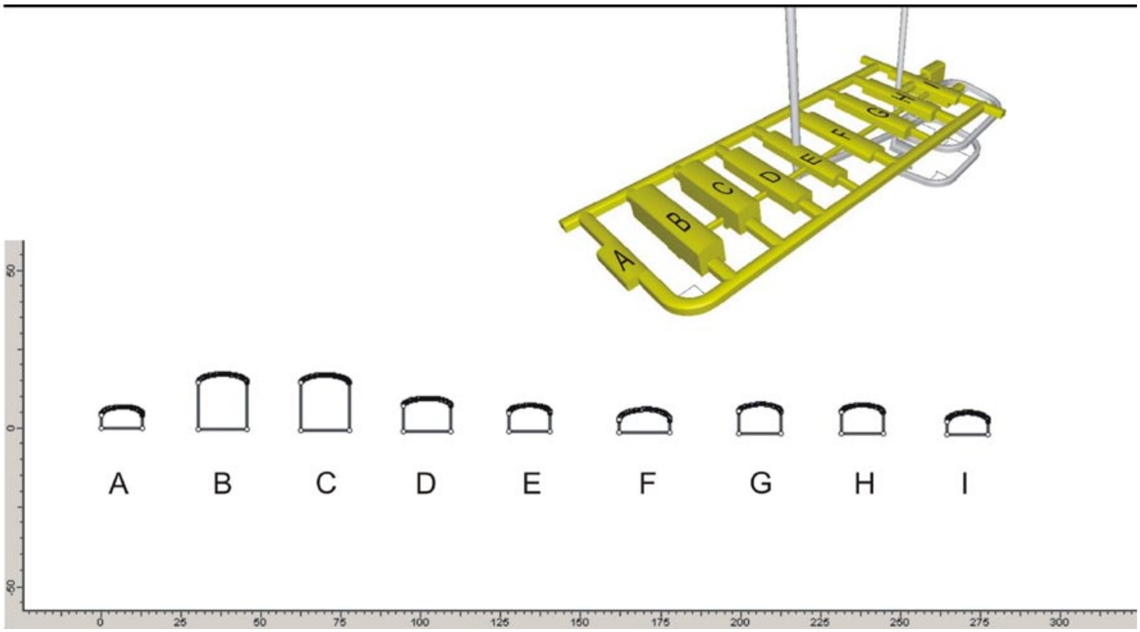



***Figure 6-3.*** *Layout of the Central Area and the two-dimensional section used in the calculations.*

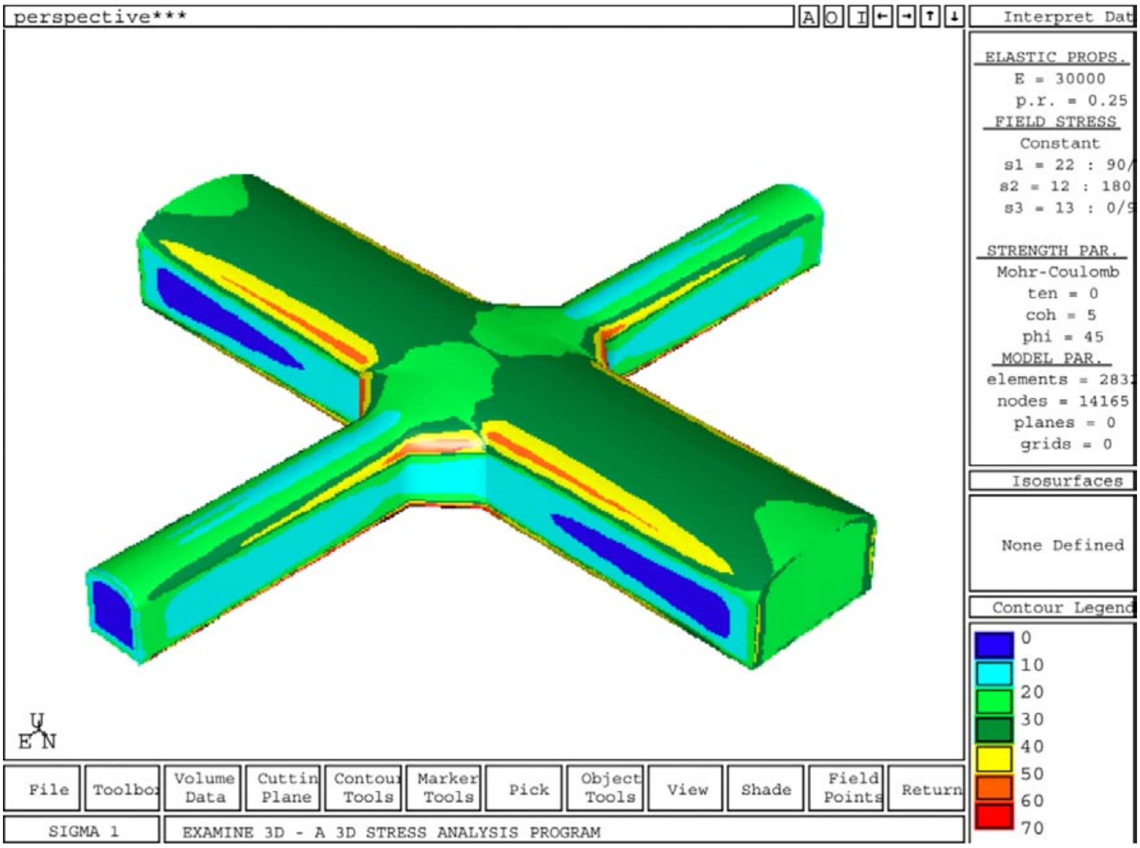



***Figure 6-4.*** *Example of the boundary stress concentrations on the main tunnel and the deposition tunnel. Notice that the maximum boundary stress occurs at the springline of the excavations and that the lowest stress (distressing) occurs in the sidewalls of the excavation / Eriksson et al. 2009, R-09-10/.*

The cavern stress analyses were carried out using orientations 0°, 30°, 60° and 90° relative to the maximum horizontal stress (Table 6-3).The results of those analyses are also expressed in Table 6-3 in terms of highest calculated stress in the roof. In most cases the maximum tangential stress occurs on the left side of Cavern B at the springline (see Figure 6-3 for orientation). Similar stress magnitudes stresses were found in Cavern C and D at a similar location. These tangential stresses are insufficient to induce spalling in any of the rock types encountered at Laxemar.

Because of the relatively low stress magnitudes at Laxemar it is unlikely that the Central Area caverns will be overstressed. However, the complex geometry of some of the excavations in the Central Area will need to be analysed using three-dimensional analyses in the final design to examine areas for potential over-stressing and de-stressing. Both over-stressing and de-stressing can induce in instability that is not captured in two-dimensional analyses when complex geometry is encountered.

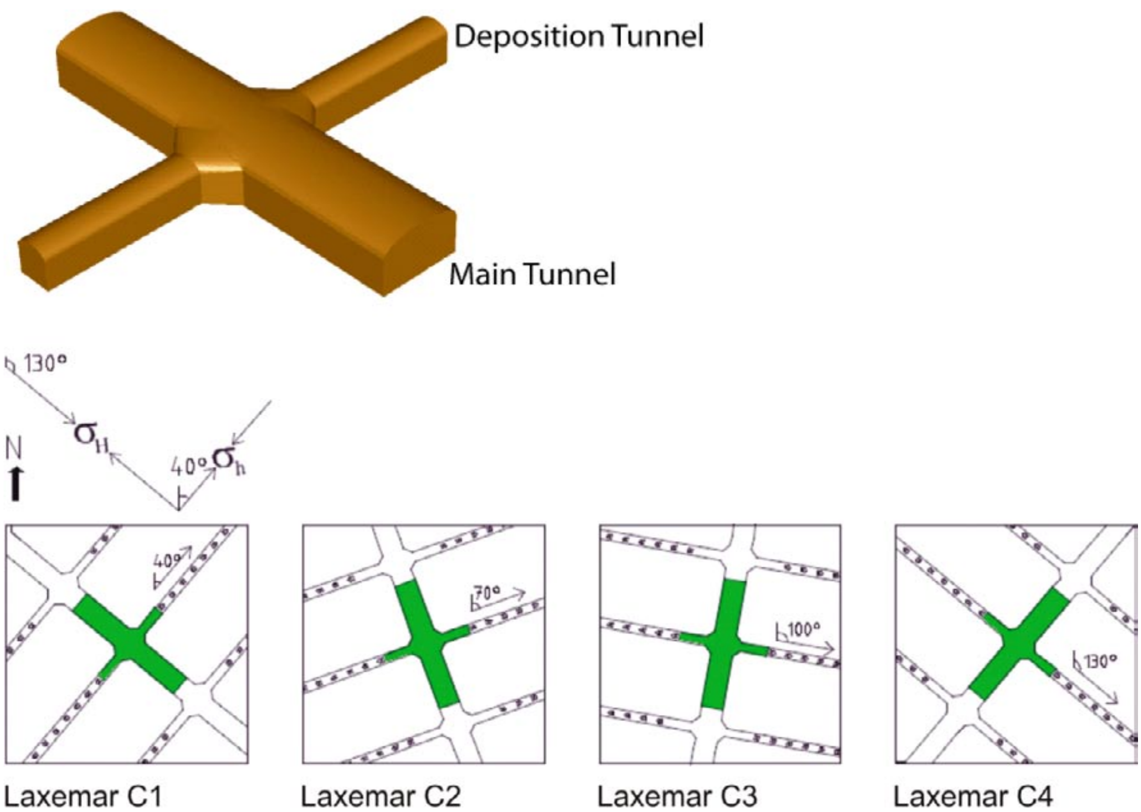
**Table 6-3. Maximum tangential stress calculated on the boundary of the caverns in the Central Area. The two dimensional analyses were carried out using the ”most likely” stress model.**

| Case | Azimuth of deposition tunnel [degrees) | Relative angle of cavern to maximum horizontal stress | Maximum tangential stress [MPa | Position of maximum tangential stress on the boundary of the cavern |
|---|---|---|---|---|
| C1 | 40 | 90 degrees | 79 | Left hand side springline on Cavern B |
| C2 | 70 | 60 degrees | 77 | Left hand side springline on Cavern B |
| C3 | 100 | 30 degrees | 72 | Left hand side springline on Cavern B |
| C4 | 130 | 0 degrees | 68 | Left hand side springline on Cavern D |

### 6.1.3 Deposition Area

The relatively low frequency of open fractures at the depth of the repository suggests that the most likely Ground Behaviour that will be encountered during construction of the tunnels (Main, Transport, Deposition) in the Deposition Area will either be spalling or minor wedge instability. Spalling will only occur if the tangential stresses acting on the boundary of the excavations exceed the spalling strength. To assess this potential, three dimensional elastic stress analyses were carried out using the boundary element program Examine3D to assess the magnitude of the tangential stress concentrations on the boundary of the excavations (Figure 6-4). As shown in Figure 6-4, tunnels boundaries are subjected to both stress magnitudes increases and decreases. The regions with the greatest stress concentrations may spall while regions that exhibit stress decreases may experience wedge failures.

The geometry of the tunnels used in the various three dimensional analyses is illustrated in Figure 6-4. For each tunnel configuration, the orientation of the main tunnel relative to the orientation of the maximum horizontal stress was varied from 0 to 90 degrees in increments of 30 degrees (Figure 6-5). The calculated maximum elastic tangential stress concentrations ranged in magnitude from 66 to 70 MPa. These stress concentrations are below the estimated spalling strength range (88 to104 MPa) expected for the major rock types in the Laxemar Rock domains (see crack initiation stress in Table 3-2). The layout with the deposition tunnels aligned at angles less than 30 degrees to the major horizontal stress gives the lowest stress concentrations on the deposition tunnels /Eriksson et al. 2009/.




| Case | Azimuth of deposition tunnel [degrees) | Relative angle of deposition tunnel to maximum horizontal stress | Maximum calculated stress [MPa] | Position of maximum stress |
|---|---|---|---|---|
| C1 | 40 | 90 degrees | 70 | springline in the crossing |
| C2 | 70 | 60 degrees | 70 | springline in the crossing |
| C3 | 100 | 30 degrees | 68 | springline in the crossing |
| C4 | 130 | 0 degrees | 66 | springline in the crossing |

***Figure 6-5.*** *Illustration of the three dimensional analyses carried out to investigate the stability of the main tunnel and deposition tunnel intersection. Compiled from /Eriksson et al. 2009/.*

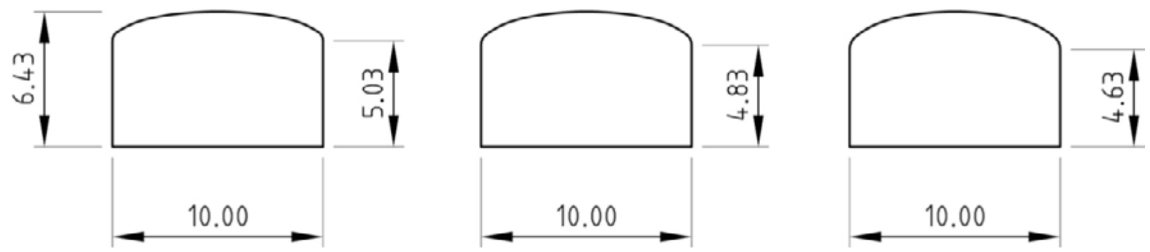



***Figure 6-6.** Illustration of the three different main tunnel cross-sections that analyzed to examine the effect of roof curvature on the tunnel stress concentrations /Eriksson et al. 2009/.*

Eriksson et al. /2009 R-09-10/ also investigated the stability of the main tunnel and deposition tunnel intersections if the deposition tunnel was not orthogonal to the main tunnel. Eriksson et al. /2009 R-09-10/ concluded that the effect of a skewed intersection was marginal and focused additional stress analyses (discussed below) to geometries with orthogonal intersections.

/Eriksson et al. 2008/ also explored the impact of the cross section geometry of the main tunnel on tangential stress concentrations. The tunnel geometry was modified using a flatter roof by increasing the distance from the roof to the spring line by 0.2 m and 0.4 m. They concluded that a flatter roof profile increases the tangential stress stresses on the springline area while reducing the tangential stresses in the roof. Such a profile change could be applied if spalling was encountered in the roof of the main tunnel during construction. In all cases analysed the tangential stress concentrations were below the spalling strength and as expected the lowest tangential stresses occur when the tunnel is aligned to the maximum horizontal stress.

The stress concentrations around deposition holes were also evaluated using three-dimensional elastic analyses. These analyses were carried out using both the "most likely" and "possible maximum" stress model while all the analyses discussed above were carried out using the "most likely" stress model (see Table 3-5). Only the results from the "possible maximum" stress model are reported here as those results represent the worst case. It is important to realise that the "possible maximum" stress model was developed from actual in situ measurements made at the Äspö HRL at a depth of 450 m and therefore may be more representative of the actual stresses at the repository depth than those given by the "most likely" stress model which is based on borehole measurements. Figure 6-7 shows the five cases that were examined to evaluate the maximum tangential stress on the deposition hole. The orientation of the deposition tunnel relative to the maximum horizontal stress ranged from 0 (parallel) to 90 degrees (perpendicular).

Figure 6-8 summarises the maximum tangential stress with depth along the deposition hole when the deposition tunnel is aligned at various orientations relative to the maximum horizontal stress. Also shown in Figure 6-8 is the spalling criterion established by /Andersson 2007/ from the APSE Experiment and the spalling criterion based on the laboratory crack initiation stress (CI) from Table 3-2. It is clear from Figure 6-8 that there is a potential for spalling in some rock types if the deposition tunnel is aligned at a large angle to the maximum horizontal stress. To reduce the potential for spalling in the deposition holes the deposition tunnels should be aligned at angles less than 30 degrees to the orientation of the maximum horizontal stress (Figure 6-8). The effect of stress induced spalling on the deposition tunnels due to the thermally induced stresses is a potential concern for long term safety and is not analysed in this report.

The stress analyses in Figure 6-8 shows there is a potential for spalling in some of the weaker rock types at Laxemar (low CI values) if the deposition tunnels are aligned at large angles to the maximum horizontal stress. However the construction experience, pertaining to the excavation of underground openings from the nearby Äspö HRL, summarised by /Carlsson and Christiansson 2007/ shows that stress induced spalling was not observed at the Äspö HRL at 450 m depth in deposition holes drilled from the Prototype Repository tunnel, which is at a large angle to the maximum horizontal stress. It should be noted that the main rock type (Äspö Diorite) at the Äspö HRL has a larger crack initiation stress (125 MPa) than most of the major rock types encountered at Laxemar (88–104 MPa in Table 3-2) and as shown in Figure 6-8 the maximum tangential stress from the "possible maximum" stress model does not exceed 120 MPa. Hence the results in Figure 6-8 actually support the construction experience at the Äspö HRL reported by /Carlsson and Christiansson 2007/.

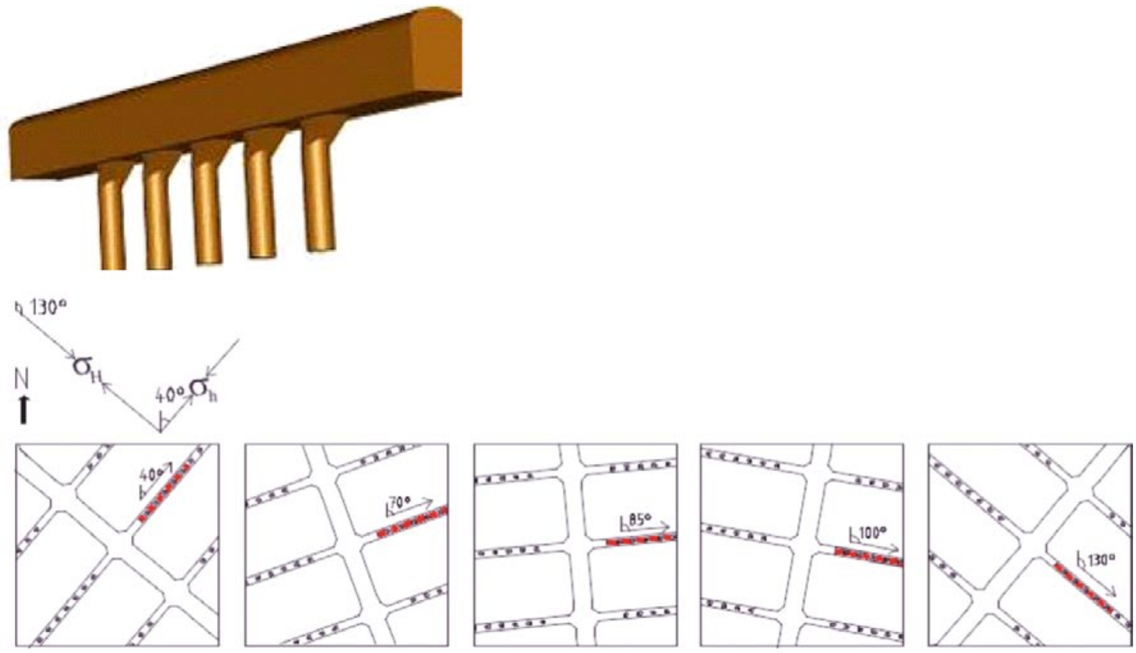



*Figure 6-7.* *Illustration of the five cases examined to assess the tangential stress concentrations around a deposition hole. The orientation of the deposition tunnel relative to the orientation of the maximum horizontal was varied from 0 to 90 degrees. Compiled from /Eriksson et al. 2009/.*

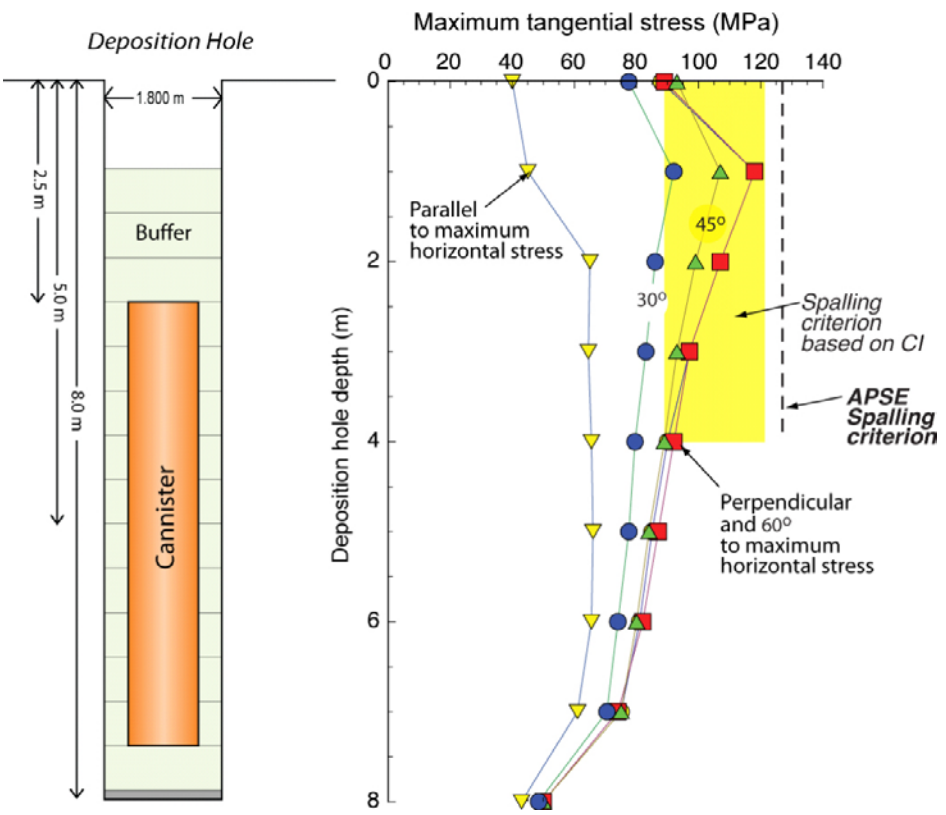



*Figure 6-8.* *Maximum tangential stress from three-dimensional elastic analyses versus deposition hole depth for the "possible maximum" stress model. Also shown the spalling criterion developed by /Andersson 2007/ from the APSE Experiment at Äspö HRL and the spalling criterion based on Crack Initiation (CI) from laboratory tests.*

## 6.2 Support measures

The system behaviour was assessed by /Eriksson et al. 2009/ using analytical methods, rock characterisation methods (the Q-system), numerical methods, and comparative studies based on experience from previous underground projects in the Oskarshamn area (cf. Section 6.1). /Eriksson et al. 2009/ evaluated the system behaviour and concluded that all the tunnels in the repository except for the deposition tunnels were to be supported with shotcrete. In order to simplify the number of support types /Eriksson et al. 2009/ proposed the support types in Table 6-4. The distribution of these support types are summarised in Table 6-5 and the estimated support quantities are provided in Table 6-6.

The results from the various analyses used by /Eriksson et al. 2009/, all indicate that conventional underground support measures would be sufficient to ensure that the performance of the underground openings was acceptable. These results are supported by the underground experiences from Clab and Äspö Hard Rock Laboratory where most of the underground openings are supported with spot bolts or are unsupported /Carlsson and Christiansson 2007/.

## 6.3 Summary

The analyses of the ground behaviour indicate that stable openings can be readily achieved using traditional tunnel support systems. In addition, experience from nearby underground openings at the Äspö HRL to depth of 450 m suggest a good quality rock mass can be expected and that structurally-controlled wedges will be the dominant ground behaviour /Carlsson and Christiansson 2007/. Figure 6-9 illustrates the quality of a tunnel excavation that was achieved using controlled blasting at the Äspö HRL at a depth of 450 m. This tunnel has a cross sectional area of about 20 $m^2$ was excavated approximately perpendicular to the maximum horizontal stress. No stress-induced spalling was observed in this tunnel or any of the underground openings at Äspö HRL. As illustrated by Figure 6-9 even at the 450 m depth spot bolting was usually adequate to provide a stable safe opening.

Regardless of the support required to achieve stable openings, temporary support will be applied to ensure worker safety during construction. The extent and type of this temporary support will be decided during the Detailed Design. Table 6-7 contains the estimated quantities of the material used for ground support in each function area for this design step.

**Table 6-4. Summary of support types (ST) used in design D2.**

| Support type | Description | Ground types | Ground behaviour |
|---|---|---|---|
| ST1 | Fibre-reinforced shotcrete 30 mm in roof + uppermost 1 m of walls.<br>Spotbolting: 1 bolt / 50 $m^2$ in roof and walls(ø 25 mm, length 3 m). | GT1 | GB1, GB2A |
| ST2 | Fibre-reinforced shotcrete 50 mm in roof + uppermost 1 m of walls.<br>Spot bolting: 1 bolt / 50 $m^2$ in walls (ø 25 mm,length 3 m).<br>Systematic bolting: c/c 2 m in roof (ø 25 mm,length 3 m). | GT2<br>GT3 | GB1, GB2A, GB3B |
| ST3 | Fibre-reinforced shotcrete 75 mm in roof + uppermost 1 m of walls.<br>Spot bolting: 1 bolt / 50 $m^2$ in walls (ø 25 mm,length 3 m).<br>Systematic bolting: c/c 1 m in roof (ø 25 mm,length 3 m). | GT4 | GB1, GB2A, GB3B |
| ST4 | Concrete lining | GT4 | GB2B, GB3B |
| ST Deposition Tunnel | Wire mesh in roof + uppermost 1 m of walls for GT2 and GT3.<br>Spot bolting: 1 bolt / 50 $m^2$ in roof and walls (ø 25 mm, length 3 m). | GT1, GT2, GT3 | GB1, GB2A |
| ST Cavern | Fibre-reinforced shotcrete 50 mm in roof + uppermost 1 m of walls.<br>Spot bolting: 1 bolt / 50 $m^2$ in walls (ø 25 mm,length 3 m).<br>Systematic bolting: c/c 2 m in roof (ø 25 mm,length 3 m). | GT1, GT2<br>GT3 | GB1, GB2A |

**Table 6-5. Distribution (length, m) of support type for each functional area.**

| | ST1 [m] | ST2 [m] | ST3 [m] | STC [m] | STD [m] |
|---|---|---|---|---|---|
| **Ramp** | | | | | |
| Tunnel (5.5 m wide)[1] | 2,785 | 1,763 | 1 | – | – |
| Tunnel (6.0 m wide) | – | 123 | – | – | – |
| Tunnel (6.5 m wide) | 284 | 412 | 11 | – | – |
| Tunnel (7.0 m wide) | 104 | 94 | 32 | – | – |
| Passing places and niche (8.0 m wide) | 148 | 63 | – | – | – |
| Niche (10.0 m wide) | 17 | 7 | – | – | – |
| **Ventilation** | | | | | |
| Shaft (ø 1.5 m) | 176 | 76 | – | – | – |
| Shaft (ø 2.5 m) | 343 | 147 | – | – | – |
| Shaft (ø 3.5 m) | 343 | 147 | – | – | – |
| Shaft (ø 4.5 m) | 17 | 8 | – | – | – |
| Tunnel (4.0 m wide) | 565 | 242 | – | – | – |
| Tunnel (8.0 m wide) | 22 | 10 | – | – | – |
| **Central Area** | | | | | |
| Skip shaft (ø 5.0 m) | 391 | 167 | – | – | – |
| Elevator shaft (ø 6.0 m) | 374 | 160 | – | – | – |
| Silo (ø 9.5 m) | 15 | 7 | – | – | – |
| Tunnel (3.0 m wide) | 94 | 40 | – | – | – |
| Tunnel (4.0 m wide) | 373 | 160 | – | – | – |
| Tunnel (5.0 m wide) | 33 | 14 | – | – | – |
| Tunnel (7.0 m wide) | 653 | 280 | – | – | – |
| Halls (13.0 m wide) $n = 5$ | – | – | – | 290 | – |
| Halls (15.0 m wide) $n = 3$ | – | – | – | 186 | – |
| Crushing hall (10.0 m wide) | – | – | – | 22 | – |
| Vehicle hall (16.0 m wide) | – | – | – | 65 | – |
| Service hall (12 m wide) | – | – | – | 20 | – |
| **Deposition Area** | | | | | |
| Ventilation shafts SA01 and SA02 (ø 3.0 m) | 757 | 270 | 2 | – | – |
| Main tunnel (10.0 m wide) | 5,141 | 2,727 | 15 | – | – |
| Transport tunnel (7.0 m wide) | 1,054 | 5,754 | 45 | – | – |
| Deposition tunnel (4.2 m wide) | – | – | – | – | 89,408 |

**Table 6-6. Compilation of reinforcement amounts for different facility parts of the repository.**

| | No of bolts[1] | Bolts/m | Shotcrete [$m^3$] | Wire mesh [$m^2$] |
|---|---|---|---|---|
| **Ramp** | | | | |
| Tunnel (5.5 m wide)[2] | 4,104 | 0.90 | 1,522 | – |
| Tunnel (6.0 m wide) | 243 | 1.97 | 57 | – |
| Tunnel (6.5 m wide) | 1,033 | 1.46 | 291 | – |
| Tunnel (7.0 m wide) | 507 | 2.20 | 104 | – |
| Passing places and niche (8.0 m wide) | 203 | 0.96 | 84 | – |
| Niche (10.0 m wide) | 30 | 1.25 | 11 | – |
| **Ventilation** | | | | |
| Shaft (ø 1.5 m) | 24 | 0.09 | 43 | – |
| Shaft (ø 2.5 m) | 77 | 0.16 | 139 | – |
| Shaft (ø 3.5 m) | 108 | 0.22 | 194 | – |
| Shaft (ø 4.5 m) | 7 | 0.28 | 13 | – |
| Tunnel (4.0 m wide) | 473 | 0.59 | 207 | – |
| Tunnel (8.0 m wide) | 30 | 0.94 | 12 | – |
| **Central Area** | | | | |
| Skip shaft (ø 5.0 m) | 175 | 0.31 | 315 | – |
| Elevator shaft (ø 6.0 m) | 201 | 0.38 | 362 | – |
| Silo (ø 9.5 m) | 13 | 0.60 | 24 | – |
| Tunnel (3.0 m wide) | 61 | 0.45 | 29 | – |
| Tunnel (4.0 m wide) | 311 | 0.58 | 135 | – |
| Tunnel (5.0 m wide) | 33 | 0.71 | 15 | – |
| Tunnel (7.0 m wide) | 830 | 0.89 | 347 | – |
| Halls (13.0 m wide) $n = 5$ | 1,157 | 3.99 | 247 | – |
| Halls (15.0 m wide) $n = 3$ | 878 | 4.72 | 174 | – |
| Crushing hall (10.0 m wide) | 76 | 3.45 | 15 | – |
| Vehicle hall (16.0 m wide) | 313 | 4.82 | 67 | – |
| Service hall (12 m wide) | 74 | 3.70 | 16 | – |
| **Deposition Area** | | | | |
| Ventilation shafts SA01 and SA02 (ø 3.0 m) | 194 | 0.19 | 343 | – |
| Main tunnel (10.0 m wide) | 10,368 | 1.32 | 3,900 | – |
| Transport tunnel (7.0 m wide) | 13,547 | 1.98 | 3,333 | – |
| Deposition tunnel (4.2 m wide) | 22,422 | 0.25 | – | 224,094 |
| Total | 57,492 | - | 11,999 | 224,094 |


**Figure 6-9.** *A tunnel with a cross sectional area of 20 m² was excavated in 2009 at 450 depth in the Äspö HRL. The tunnel is aligned perpendicular to the major horizontal stress and only required spot-bolting support to provide a stable safe opening.*

**Table 6-7. Compilation of the material and quantities used for ground support in each functional area.**

| Subsidiary material | kg/m³ | Ramp/access | | Central Area, including ventilation | | Deposition Area, including SA01 and SA02 | |
|---|---|---|---|---|---|---|---|
| | | [ton] | [m³] | [ton] | [m³] | [ton] | [m³] |
| **Rock Bolt s** | | | | | | | |
| Rock bolts (l=3 m, d=25 mm) | 4 | 73 | | 58 | | 558 | |
| Wire mesh (1,7 kg/m²) | | | | | | 381 | |
| Fixing bolts (29,329 pcs) | | | | | | 112 | |
| **Rock Bolt Grout** | | | | | | | |
| Cement | 340 | 40 | 19 | 31 | 15 | 301 | 143 |
| Silica | 226.7 | 26 | 13 | 21 | 10 | 200 | 95 |
| Water | 266.6 | 31 | 31 | 25 | 25 | 236 | 236 |
| Glennium 51 | 4 | 0.5 | 0.5 | 0.4 | 0.4 | 3.5 | 3.5 |
| Quarts filler | 1,324 | 154 | 77 | 122 | 61 | 1,171 | 585 |
| **Shotcrete** | | | | | | | |
| Water | 158 | 327 | 327 | 372 | 372 | 1,197 | 1,197 |
| Ordinary Portland cement CEM I 42.5 | 210 | 435 | 207 | 494 | 235 | 1,591 | 758 |
| Silica fume | 140 | 290 | 138 | 329 | 157 | 1,061 | 505 |
| Coarse aggregate (5–11) | 552 | 1,143 | 672 | 1,299 | 764 | 4,182 | 2,460 |
| Natural sand (0–5) | 1,025 | 2,122 | 1,248 | 2,412 | 1,419 | 7,765 | 4,568 |
| Quarts filler (0–0,25) or Limestone filler (0–0,5) | 250 | 518 | 259 | 588 | 294 | 1,894 | 947 |
| Superplasticiser "Glennium 51" from Degussa | 3 | 6.2 | 6.2 | 7.1 | 7.1 | 23 | 23 |
| Air entraining agent "Sika AER S" | 2.5 | 5.2 | 5.2 | 5.9 | 5.9 | 19 | 19 |
| Accelerator "Sigunit" from Sika or AF 2000 from Rescon | 7% | 0.1 | 0.1 | 0.2 | 0.2 | 0.5 | 0.5 |
| Steel fibres | 70 | 145 | 48 | 165 | 55 | 530 | 177 |

# 7 Groundwater control and grouting

The SDM Site for Laxemar /SKB 2009a/ divided the bedrock groundwater system into hydraulic domains where the flow is controlled by: (1) the characteristics of the deformation zones, and (2) the connected fracture network through the rock mass. The underground excavations required for the Final Repository will start at the ground surface and penetrate these hydraulic domains (i.e. HRD_C, HRD_W and HRD_EW007). Therefore an estimate of the potential groundwater inflows is required to establish: (1) whether or not the expected inflows meet the limits specified in UDP/D2 /SKB 2007/, (2) the potential control measures that may be required to reduce the groundwater inflows to acceptable levels, and (3) the potential drawdown that may occur around the underground excavations, particularly those that penetrate the ground surface.

An assessment of the grouting measures required to control the inflows to the specified levels was carried out by /Brantberger and Jansson 2009/ using the grouting technology specified in UDP/D2, i.e. application of a low-PH grout recipe using proven grouting technology.

## 7.1 Inflow estimates

The hydraulic domains at Laxemar have been divided into three depth intervals (0–150 m, 150–400 m and 400–650 m) (see Table 3-6). Within these three-depth intervals the frequency of flowing fractures ranges from 0.5 to 0.8, 0.07 to 0.5 and 0.06 to 0.2, respectively, with transmissivity values for individual fractures at these depths ranging from about $10^{-5}$ to $10^{-10}$ $m^2/s$. This implies that the groundwater flow system at Laxemar is complex and heterogeneous and that differences between the three domains are subtle. These characteristics also suggest that it is unlikely that significant rock volumes exist at repository level that are free of water-bearing fractures.

The assessment of inflow for each functional area was determined using the hydrogeological characteristics for the hydraulic domains and for deformation zones given in SER /SKB 2008a/. The assessment of water inflow has been made using both analytical and numerical calculation methods.

According to /Bergman and Nord 1982/ the water inflow into a circular tunnel can be estimated using:

$$Q_t = \frac{2 \cdot \pi \cdot K \cdot H \cdot L}{\ln\left(\frac{2 \cdot H}{r_t}\right) + \left(\frac{K}{K_g} - 1\right) \cdot \ln\left(1 + \frac{t}{r_t}\right) + \xi} \qquad \text{Eq 7–1}$$

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)

$t$= thickness of grouted zone (m)

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

$r_t$= tunnel radius (m)

$\xi$ = skin factor inside seal (dimensionless)

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

If a deformation zone is evaluated, hydraulic conductivity in Equation 7-1 can be replace by transmissivity ($T$, $m^2/s$) where $T$= $K$ $L$. Equation 7-1 can be applied to both a non-grouted and a grouted circular tunnel, and to approximate the inflows for other tunnel geometries /Brantberger and Janson 2009/.

The acceptable inflows of water to the various underground openings are:

- Deposition holes: 0.1 l/min.
- Deposition tunnels: 5 l/min, 300 m (1.7 l/min per 100 m of tunnel length); point leakage 1 l/min.
- Shaft and ramp: 10 l/min, 100 m.
- Other underground openings: 10 l/min, 100 m.

As shown in Table 7-1 the estimated inflows to some areas of the facility will exceed this inflow criterion. The results are based on average hydraulic conductivities for the rock mass.

**Table 7-1. Estimated water inflow to various excavations before grouting using Equation 7-1 /Brantberger and Janson 2009/.**

| Underground opening | Inflow per 100 m, (l/min) 10-percentile / median / 90-percentile |
|---|---|
| **Ramp (depth 0–500 m)** | |
| Depth 0–150 m | 100 |
| Depth 150–400 m | 34 |
| Depth 400–500 m | 2 / 11/ 44 |
| Zone NE107A (– 75 m) | 250 |
| Zone NE107A (–180 m) | 290 |
| **Shafts inside ramp (depth 0–500 m)** | |
| Domain HRD_C (0–150 m) | 65 |
| Domain HRD_C (150–400 m) | 26 |
| Domain HRD_C (400–500 m) | 1.6 / 9 / 36 |
| Rock caverns (depth 500 m), | 2,5 /14 / 55 |
| **Deposition tunnels (depth 500 m)** | |
| Domain HRD_C | 2,1 / 12 / 47 |
| Domain HRD_W | 0,1 / 9,4 / 235 |
| Domain HRD_EW007 | 47 / 71 / 120 |
| Zones, < 3km | 4.0 / 11 / 95 |
| **Transport/main tunnels (depth 500 m)** | |
| Domain HRD_C | 2,2 / 12 / 49 |
| Domain HRD_W | 0,1 / 9.8 / 245 |
| Domain HRD_EW007 | 49 / 75 / 125 |
| Zone NS059A, passage in transport tunnel | 270 |
| Zone NE107A, passage in transport tunnel | 70 |
| Zones < 3 km | Min.: 4.3, Median: 12, Max.: 105 |
| **Ventilation air shafts** | |
| Domain HRD_C (0–150 m) | 65 |
| Domain HRD_C (150–400 m) | 27 |
| Domain HRD_C (400–500 m) | 1,7 / 9,3 / 37 |
| Domain HRD_W (0–150 m) | 87 |
| Domain HRD_W (150–400 m) | 33 |
| Domain HRD_W (400–500 m) | 0,1 / 7,4 / 185 |
| Zone klx11_dz11 (400 m) | 12 |

## 7.2 Grouting strategy

### 7.2.1 Overview

Grouting technology has evolved considerably over the past 20 years. The following section presents general guidelines related to; i.e. fan geometry, grout, execution, equipment and control measures that will need to be optimised during the execution of the work. The following guidelines, based on analytical calculations and on experiences from construction projects, have been proposed by /Brantberger and Janson 2009/, however, these general guidelines may need to be revised during the detailed design:

- Test drilling and grouting trials are required during the initial phase of ramp and shaft construction and during the initial phase of main, transport and deposition tunnel construction (depth 500 m). This drilling and grouting will be carried out to establish the means and methods required for effectively sealing the excavations.
- 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 inflow criteria. Moreover, post-grouting will be necessary to seal point leakage following excavation.
- Systematic investigation and probe drillings are essential in selecting appropriate grouting methods and grout types. Investigation drilling is required to identify the deformation zones and probe drilling is required to plan the detailed grouting procedure.
- For the vertical raise bored shafts grouting from the surface in deep boreholes may be required.
- The skip shaft can be grouted from the shaft bottom during shaft excavation. Curtain grouting from the surface may provide better conditions for shaft sinking. Less time may then be needed for grouting from the shaft bottom, which will accelerate shaft advance.
- Grouting using cement based grout is considered "proven technology", in accordance with /SKB 2007/, and shall be used where possible. Today, grouting with silica sol at great depth cannot be considered "proven technology" but it may be required to meet the sealing criteria. Grouting methodology using silica sol at depths of 450 m are currently under development and demonstration by SKB /Funehag 2008/.
- For deposition tunnels the expected strategy is as follows:
  - o Selective grouting in HRD_C and HRD_W.
  - o Systematic grouting in HRD_EW007.
  - o Post-grouting if point leakage in the tunnel is >1 l/min.

### 7.2.2 Grouting types

The grouting measures are classed into three grouting types defined as:

- Grouting type 1 (GrT1): Selective grouting.
- Grouting type 2 (GrT2): Systematic grouting.
- Grouting type 3 (GrT3): Systematic grouting including special measures according to; 3A: grouting in water-bearing zones at high pressure, 3B: grouting with silica sol /Funehag 2008/.

Each grouting type defines the pre-grouting requirements, such as type of grout, fan geometry and sequencing. It should be noted that the development of grouting technology using silica sol is being developed by SKB. At the time of this report preliminary findings from research conducted at Äspö HRL /Emmelin et al. 2007, Funehag 2008/ warranted its inclusion as an alternative grouting material for application in Grouting type 3.

### 7.2.3 Grouts and grouting equipments

The grout is selected primarily based on the estimated hydraulic fracture aperture, i.e. cement based grouts for "larger" apertures and silica sol for "smaller" apertures. For design step D2 "Plug grout" is normally used as hole-filling grout but can also be used in extreme situations 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 fracture domains/zones and depths. Silica sol is required to penetrate the thinnest fractures that contribute to the inflow at depth. Therefore Silica sol is proposed as the main grout in deposition tunnels and as supplement in other tunnels and rock caverns and also for post-grouting of point leakage.

Experience indicates that mixing low pH cement-based grouts requires advanced mixing equipment to provide batch consistency. All equipment, for example hoses and couplings, must be designed for the total pressures that apply at the repository depth. Blow-out preventors' might be required at repository depth, if high flows are expected.

### 7.2.4 Grouting fan geometries

Grout fan geometry depends on geometry/profile of the individual underground openings. Typically, pre-grouting is done with a cone-shaped fan of boreholes from the tunnel face (Figure 7-1). A length of up to approximately 20 m for a pre-grouting fan is commonly applied to achieve practical drilling precision and efficiency. Experience shows that an overlap of 5 m between the fans, and an angle that provides grout up to 5 m outside the tunnel contour results in an adequate geometry for most tunnel grouting situations. A typical fan for pre-grouting require a hole spacing of approximately 0.8 m at the collar on the tunnel face. This would require at least some 30 boreholes for pre-grouting for transport and main tunnels /Brantberger and Janson 2009/. A fan with 23 holes is proposed for a deposition tunnel. Extensive drilling for grouting outside the contour in deposition tunnels will not be accepted. Boreholes for pre-grouting inside the deposition tunnel contour will require at least 25% less boreholes. On the other hand, the effort to achieve the design grout spread outside the tunnel contour will require longer pumping times using this technique.

It shall also be noted that the grouting fans also include control holes drilled from the centre of the tunnel face parallel to the tunnel axis. Hydraulic tests should be made in these holes to verify the sealing efficiency after grouting.

### 7.2.5 Summary of preliminary grouting measures

Table 7-2 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.

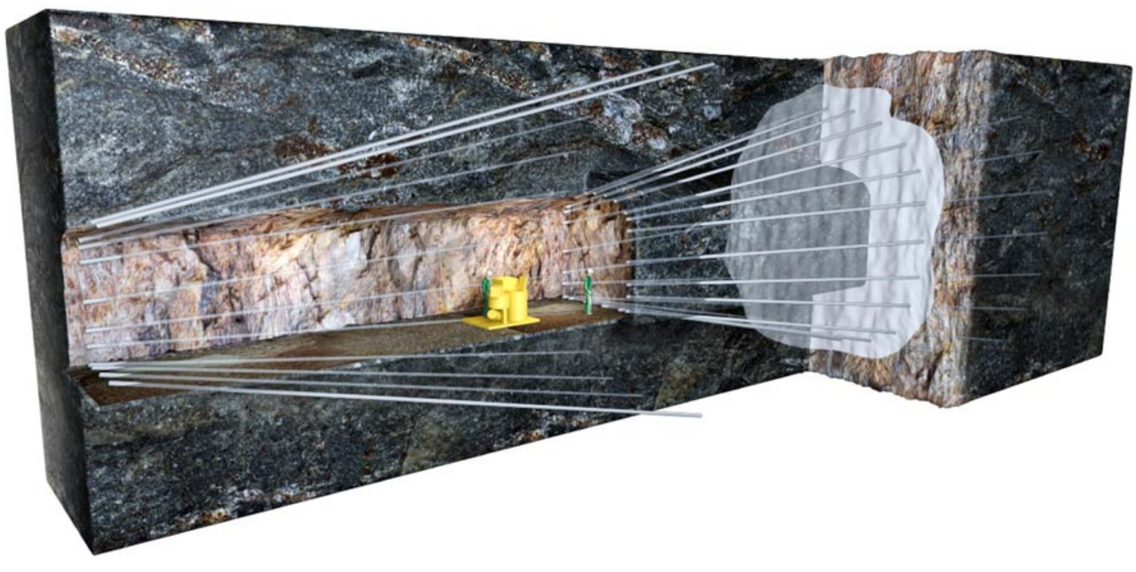


***Figure 7-1.*** *Geometry of a typical fan for pre-grouting.*

**Table 7-2. Summary of selected grouting types, GrT, and principles for grouting in different functional areas /Brantberger and Janson, 2009/.**

| 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. |

## 7.3 Grouting of functional areas

### 7.3.1 Ramp

Grouting in the ramp will be made down to a depth of 400 m, and especially between 0–150 m where more extensive grouting will be required, see Figure 7-2. The grouting types are selective grouting, GrT 1, and systematic grouting, GrT2. When passing highly conductive zones the grouting could be extensive and require special measures, GrT3a.

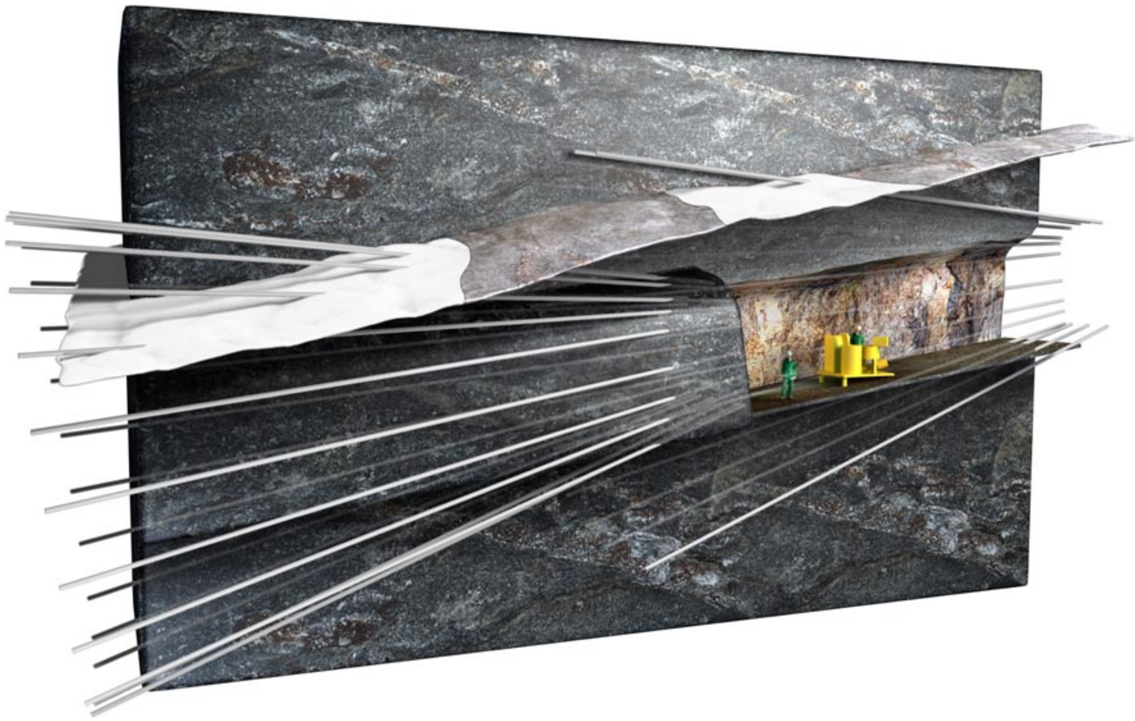


*Figure 7-2. Illustration from above of angled grouting holes in bottom of the fan.*

### 7.3.2 Shafts

The shafts will be done in two different ways, partly by shaft sinking (skip shaft) and partly by expanding the shafts using raise-drilling technique (elevator and ventilation shafts). With regard to the uncertainty concerning fulfilment of requirements on inflow in the drilled shafts, methods for post-grouting ought to be compiled for use when needed 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 down 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 grout fan designs for tunnels can also be used for shafts, see Figure 7-3. This type of grouting is sometimes denoted as "cover grouting". Furthermore, it is suggested that some of the curtain grouting holes are extended down to 500 m to reduce the risk of serious and uncontrolled leakage of water in the shaft sinking.

#### *Elevator and ventilation shafts through the Central Area*

Initial sealing will begin by grouting deep holes 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. Furthermore, the shafts down to the Central Area will be accessible from the ramp every 100 metres, which is an advantage with, regard to grouting because the work can be done in 100-metre stages. The principle for grouting is that the grouting holes are drilled about 25 m deep and with hole spacing depending on the shaft diameter. Hydraulic tests are then made, grouting with cement-based grouts and renewed drilling of grouted holes and subsequent hydraulic tests. If the desired sealing effect is achieved, a new stage of about 25 m is drilled; otherwise the grouting procedure is repeated. The boreholes in grouting round 2 are located between the holes in round 1 and processed in a similar way as for round 1 in stages of 25 metres. Figure 7-4 presents the principle for grouting the lift and ventilation shafts in the Central Area.

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

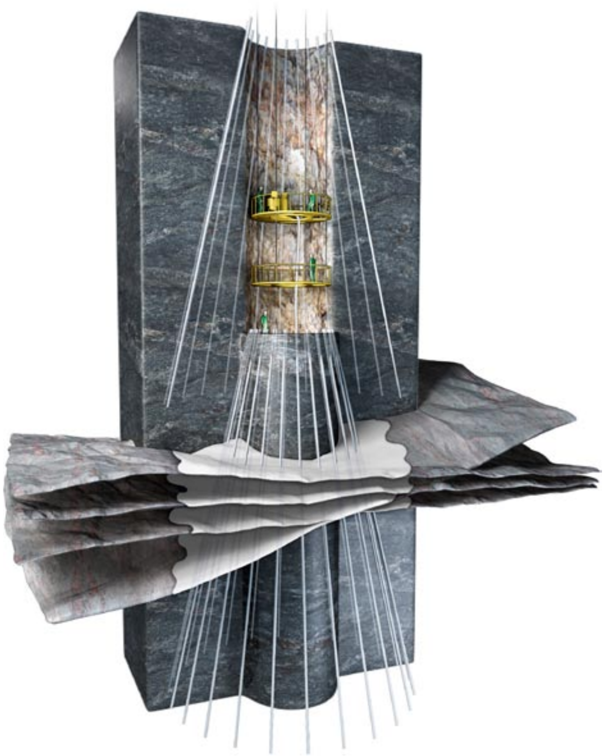


***Figure 7-3.*** *Principle of grouting in skip shaft.*

***Figure 7-4.*** *Principle for grouting in boreholes around lift and ventilation shafts.*

### 7.3.3 Central Area

The Central Area consists of a number of different tunnels and rock caverns. The unique geometries in the Central Area compared to other functional areas are the large rock caverns. The size of the rock caverns, from about 95 to 255 $m^2$ cross section, and sequence of excavation, whole section/divided stope/gallery and bench, will influence the geometry of the grouting fan but it should be possible to follow the guidelines presented previously.

### 7.3.4 Deposition Area

#### *Deposition tunnels*

The grouted rock mass around a deposition tunnel must have an average hydraulic conductivity of $1\times10^{-10}$ m/s in order to meet the inflow criteria specified in /SKB 2007/. The appearance of the grouting fan, in principle follows that of a conventional fan, with length of holes about 20 m. The biggest difference is that grouting holes are to be drilled inside the tunnel contour when systematic grouting is expected. The exception will be in deformation zones were holes outside the contour are allowed. The final location of grouting holes in wall and roof will depend on the geometry of the deformation zone and also on the orientation of the fractures within the deformation zone. Holes inside the tunnel contour reduce the number of holes needed but at the same time the requirement for grout penetration length is increased (>5 m). Grouting holes inside the contour may also restrict the possibility ability to adapt the direction of the borehole relative to the fracture orientation.

In domain HRD_EW007 systematic pre-grouting with silica sol will be needed in almost 90% of the deposition tunnels. In hydraulic domains HRD_C and HRD_W, 60% and 40%, respectively, will need grouting with silica sol. For domain HRD_EW007 systematic probe drilling will need to be carried out in possible deposition hole locations. Grouting of these holes will in some cases be necessary, and criteria for grouting must be established in the detailed design.

#### *Exhaust ventilation shafts in the Deposition Area*

The exhaust shafts in the Deposition Area will be made using raise-drilling technique in the same way as for the lift and exhaust ventilation shafts in the Central Area. Grouting in these shafts will be carried out before raise drilling begins. The grouting is carried out in long, vertical boreholes, which are drilled in a ring round the shafts. Several grouting holes may be about 500 m deep, which puts strict demands on drilling equipment and handling of grout. The requirement on 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. 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 requirements specification 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.

#### *Intersection with deformation zones*

Two steeply dipping deformation zones NS059A and NE107A are > 3km and will intersect transport tunnels (see /Leander et al. 2009/). The transmissivity values for NS059A and NE107A at deposition level are estimated to be $1.1\times10^{-5}$ $m^2$/s and $2.8\times10^{-6}$ $m^2$/s respectively and the thickness of the zones are about 50 m and 35 m, according to SER /SKB 2008a/. Deformation zone NE107A also crosses the ramp at about Elevation –75 m and about –180 m with transmissivity values $5.2\times10^{-5}$ $m^2$/s and $2.9\times10^{-5}$ $m^2$/s respectively, according to SER /SKB 2008a/.

The shorter deformation zone klx11_dz11 intersects the outer shaft at about depth level 400 m. The zone has an estimated transmissivity of $7.5\times10^{-7}$ $m^2$/s at this depth.

## 7.4 Application of the Observational Method for grouting

Prior to probe drilling, a procedure will be needed to determine whether or not grouting is required. A decision methodology based on results from hydraulic tests in probe holes was developed by /Funehag 2008/. This methodology whether grouting is needed by evaluating the probability that the median value from the hydraulic tests will be lower than the critical transmissivity value specified to meet the inflow requirements. An example of the application of this Observational Method to the grout selection process for the deposition tunnels in each hydraulic domain is summarised in Table 7-3. While Table 7-3 is illustrative only it highlights the need in the initial stages of the Deposition Area development to establish the grout selection methodology.

## 7.5 Estimated amounts of grouting material

Based on the analyses in this section, Table 7-4 summarises the amount of grouting material 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 grout that remains in the rock mass after blasting is presented in Table 7-5. The amounts presented are rounded off to the nearest 10 $m^3$.

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

- "Plug grout" is used both for filling of tight holes and also 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, eg, a first grouting round in rock mass of high hydraulic conductivity.
- "Injection grout" is the cement grout that is used primarily.
- Silica sol is used primarily in grouting type 3B and for complementary grouting in the other grouting types and also for post-grouting.

The amount of grout materials was calculated using the grout recipes given in /Brantberger and Janson 2009/ and the estimated proportion of Grout types. Table 7-6 presents the estimated tunnel lengths with and without grouting, for each functional areas, and the estimated grout take. Table 7-7 presents the estimated amounts of grout materials remaining in the rock mass.

**Table 7-3. Summary of a possible process for choosing grouting types and adjusting grouting measures /Brantberger and Janson 2009/.**

| 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 |

**Table 7-4. Summary of total amounts of grout injected before blasting for different functional areas /Brantberger and Janson 2009/.**

| Functional areas/ underground openings | Drilling number of holes/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:<br>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) | 57,730/1,154,670 | 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) | 66,330/1,326,670 | 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 V1 | Curtain grouting: 25/12,500 | 25–95 | 10/20/70/– |
| Exhaust shaft V2 (including klx11_dz11) | Curtain grouting: 25/12,500 | 40–155 | 10/20/70/– |

**Table 7-5. Summary of amounts of grout remaining in the rock mass after blasting for different functional areas /Brantberger and Janson 2009/.**

| Functional areas/ underground openings | Drilling number of holes/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<br>Curtain grouting: 75/37,500 | 25–115 | 10/20/50/20 |
| **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 V1 | Curtain grouting: 25/12,500 | 20–85 | 10/20/70/- |
| Exhaust shaft V2 (including klx11_dz11) | Curtain grouting: 25/12,500 | 35–135 | 10/20/70/- |

**Table 7-6. Estimated lengths with and without grouting and grout take /Brantberger and Janson 2009/.**

| Functional areas/ underground openings | Length (m) | Grout take ($m^3/m$) |
|---|---|---|
| **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 7-7. Estimated quantities of grout materials and drilling that remain in the rock mass after excavation of the different underground openings /Brantberger and Janson 2009/.**

| 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 |
| Chemical grouting | 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 in /Brantberger and Janson 2009/.

An accurate prediction of inflows to underground openings and an estimate of the grout quantities required to limits these inflows remains a challenging task. While various techniques have been used to estimate the anticipated grout quantities in this section considerable uncertainty remains. /Carlsson and Christiansson 2007/ compiled the systematic grouting experience from the Clab 2 facility. In Clab 2 the grout take for the rock cavern at shallow depth was 0.17 $m^3$/m while the grout take for the largest rock caverns in Laxemar at a depth of 500 m is estimated to be about 0.35 $m^3$/m. The difference between the Clab 2 experience and forecast for rock caverns in the Central Area is about a factor of two, corresponding to porosity differences between 0.4 ($2\cdot10^{-10}$ m/s) for Clab and 0.8 ($5\cdot10^{-9}$ m/s) for Laxemar. Similar observations can be made when comparing the results from the cement-based grouting trial that was carried out in Äspö HRL in 2003 with forecasts in HRD_EW007. According to /Emmelin et al. 2007/ the grout take at Äspö HRL was about 0.13 $m^3$/m while 0.18 $m^3$/m is expected for deposition tunnels in domain HRD_EW007. The hydraulic conductivity of the rock mass containing the deposition tunnels in domain HRD_EW007 is similar to the hydraulic conductivity of the rock mass in the grouting trial. While uncertainty in the predicted grout takes remains, there is little doubt that extensive grouting will be required in large portions of Laxemar's underground facility and especially in domain HRD_EW007.

## 7.6 Groundwater inflow and drawdown

The construction of the Repository Access (ramp and shafts, and the ventilation shafts) will likely result in a groundwater drawdown around those excavations. In addition, where excavations intersect water bearing deformation zones, there is also the potential for additional drawdown areas to develop. The very strict groundwater inflows allowed in the repository will significantly limit the area extent and depth of these potential drawdowns. At this stage there is not sufficient geotechnical information to predict the nature of these site-specific drawdowns and this will have to be addressed in the detailed design.

The grouting strategy outlined in this chapter will be used to minimise the groundwater inflow and environmental impact. Groundwater pressure monitoring in boreholes in soil and rock as well as measurement of inflow will increase the understanding for groundwater response to the underground excavations. It is likely that local infiltration could be the most realistic measure to mitigate risk for environmental impact.

## 7.7 Summary

The expected inflows and the grouting requirements to minimise those inflows at Laxemar were analysed using analytical and numerical techniques. Those analyses indicate:

1. 90% of the deposition tunnels in hydraulic domain HRD_EW007,
   40% of the deposition tunnels in HRD_C and
   60% of the deposition tunnels in HRD_W will require systematic grouting.
2. Cement based grouting alone will not be sufficient to reduce the inflows to acceptable limits.
3. Silica sol grouting will be required to reach the inflow criteria currently specified in /SKB 2007/.
4. Observational Methods will have to be developed to aid in the selection of the most efficient grouting technology.

# 8 Uncertainty and risk in design D2

## 8.1 Strategy

Geotechnical engineering for underground design is fundamentally about managing risk. /Stille et al. 2003/ summarised risk assessment concepts using a general framework for managing risk and uncertainty. Regardless of how risk is managed, in all cases risk assessment requires identification of the hazard and quantifying the risk associated with each hazard (Figure 8-1). With risk defined as the (mathematical) product of the probability of occurrence of an undesired event and of the event's assessed consequence, risk can, in principle, be calculated. The full potential of risk analysis is best met with the establishment of acceptable risk criteria and relating consequences to cost/benefit analysis provides a simpler basis for evaluating acceptable risk. The link between risk and benefit must be balanced and within the context of geotechnical engineering the risks are usually reduced to an acceptable standard by the best practical means. The risks discussed in this section are the risks that the reference design D2 will need modifying. These design-related risks should not be confused with the risks described and assessed in the long term safety assessment.

The confidence in the underground design and of the repository layout described in D2 resides primarily in the confidence in the site descriptive model for Laxemar given in SDM Site /SKB 2009a/ and in a specific confidence assessment of the SDM /SKB 2008a/. This site descriptive model was used as the bases for developing the design and layout. To assess the adequacy of the design, values were assigned to key design parameters. The values assigned to each parameter were based on the data provided in SDM-Site Laxemar and these values were specified in SER/SKB 2008b/. The design was first evaluated using the "most likely" value and a deterministic design based on this value was carried out. It should be noted that this "most likely" value is not an optimistic or pessimistic value. It represents an estimate of the value developed for a parameter during the site investigation phase that is consistent with interpretation given in the Site Descriptive Model. In some cases it can represent the mean value while in others, and especially when the design concerns issues of importance for long term safety, it can be a conservative estimate that is either lower or greater than the mean value because of the uncertainty associated with the mean value. However, in keeping with the philosophy of the Observational Method, a range of values that represented conceivable best and worst case conditions were also provided for the various design scenarios considered. The range in values was provided when it was judged that a change in this value may significantly impact the design. For example, the scenario for the potential loss of deposition positions (i.e. degree of utilisation) was evaluated using various DFN approaches. In some cases alternative DFN models were used to evaluate the sensitivity and robustness of the design. For such situations a probability-based approach was used to explore the likely outcome.

Quantitative risk assessment using probability functions are appropriate when the scenario being assessed is well constrained. However there are scenarios that may impact the design that cannot be assessed using quantitative analyses, but the impact of these scenarios on the design or layout must still be evaluated. These scenarios were evaluated using qualitative analyses using the approach for Failure Modes and Effects Analyses (FMEA, Figure 8-2). According to /Rausand and Høyland 2004/ FMEA is a technique used to identify, prioritise, and resolve potential problems in a system before they occur. FMEA is usually performed during the conceptual and initial design phases of the system in order to assure that all potential failure modes have been considered and the proper provisions have been made to eliminate these failures. The primary function of FMEA is to assist in selecting design alternatives to:

- Ensure that all conceivable failure modes and their effects on operational success of the system have been considered.
- List potential failures and identify the severity of their effects.
- Develop early criteria for construction and operational planning.
- Provide historical documentation for future reference to aid in design decision making as field conditions are revealed.
- Provide a basis for operational planning.
- Provide a basis for future quantitative construction and operational risk analyses.

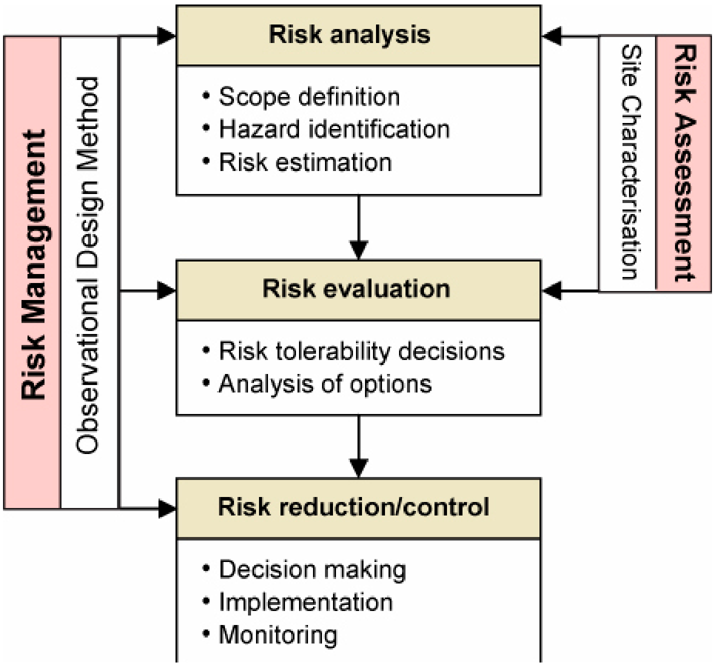



***Figure 8-1.*** *Illustration of the risk management process and its linkage to the Observation Design Method and Site Characterisation, modified from /IEC 1995/.*

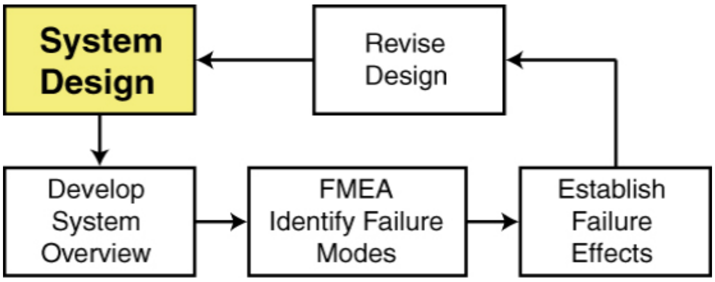



***Figure 8-2.*** *Illustration of the role of Failure Modes and Effects Analyses in the design process, modified from /Rausand and Høyland 2004/.*

While FMEA can be conducted using quantitative approaches, it is mainly a qualitative analysis tool that utilises risk matrixes to rank the relative failure modes /Rausand and Høyland 2004/. Regardless of the type of analyses, quantitative or qualitative, used to evaluate a failure scenario, three steps are required: (1) the identification of a hazard, (2) an assessment of the likelihood that the hazard will be encountered, and (3) an assessment of the consequence of the hazard. These steps are similar to those described in the Observational Method.

Within the context of the reference design D2 there are two general categories of risk:

1. The risk that the design methodology is not appropriate for the problem being analysed, and
2. The risk that the input used for the design is wrong.

It must be remembered that the primary goal of the design is to provide a constructible layout for 6,000 canisters using modern day construction technology. This section describes the risk reduction techniques and strategy that have been utilised during the course of design step D2 to ensure that this primary goal can be achieved.

## 8.2 Uncertainty in the design methodology

The methodology used to establish the design and layout is based on "best practise" augmented with state-of-the-art approaches for specific problems, e.g. thermal dimensioning. While every effort is made to develop a robust design, there are uncertainties associated with the design methodologies. The methodologies used to develop the design and layout can be grouped into five broad categories:

1. Stability of underground openings.
2. Thermal dimensioning of the repository.
3. Assessment of loss of deposition positions.
4. Assessment of inflow potential.
5. Assessment of spalling potential.

The uncertainties associated with each of these design methodologies are discussed below.

### 8.2.1 Design methodology

***Stability of underground openings***

Assessing the stability of underground openings is routinely carried out in civil and mining engineering for excavations constructed to depths of 3,000 m. Empirical, analytical and numerical methodologies are well established and routinely used in the assessment of the stability. All these approaches have been used in assessing the stability of the underground openings in the reference design and therefore the confidence in the output from the design dealing with underground stability is ranked very high.

***Thermal dimensioning of the repository***

There is essentially no experience with heating large volumes of rock at the scale required for a repository. However, there is experience with heating smaller volumes of rock at underground research facilities and individual cavern projects. The nuclear waste industry has been conducting thermal experiments over the past 30 years, e.g. the Prototype repository /Sundberg et al. 2005/. While there is not experience with heating large volumes of rock, the analytical and numerical techniques used to predict heat transfer are well established and the smaller scale experiments have validated the approaches used for the design. The confidence in the thermal dimensioning used for the layout is considered acceptable. To augment the lack of large-scale thermal experience, the thermal design parameters were evaluated using various techniques and a wide range of thermal properties were used in establishing the design. There is also ample opportunity during the operation of the repository to optimise the thermal dimensioning of the layout using temperature measurements.

***Assessment of loss of deposition positions due to long fractures***

In order to mitigate the impact of potential future earthquakes deposition positions are selected such that they do not intersect too long fractures. Theoretically fractures with radii larger than 60 m to 150 m should be avoided, depending on the location of the deposition hole position in relation to the deformation zones, but since fracture sizes may be very hard to measure more robust criteria are needed. Currently, deposition positions must satisfy the *Extended Full Perimeter Intersection* (EFPC) criterion originally suggested by /Munier 2006/ and later modified (see Design Premises Long Term Safety /SKB 2009b/. The resulting potential loss of deposition positions was assessed using the geological Discrete Fracture Network (DFN) developed for the site. DFNs are a relatively new stochastic method for describing the discrete fracturing that occurs in rocks. There is substantial uncertainty in the robustness of DFN models for predicting the discrete fractures at the repository depth based entirely on surface mapping and borehole logging. Furthermore, the EFPC criterion unnecessarily rejects many positions, which encounter long fractures less than the theoretical thresholds of 60 to 150 m. However, there is little doubt that some deposition positions may be rejected due to long discrete fractures and therefore an allowance for this loss must be assessed. The current design applied the EFPC criterion using a variety of DFN models to estimate the possible range in the loss of deposition positions. While there is a lack of confidence in the DFN approach used to describe the fracture network and while it is known that the EFPC criterion is unnecessarily conservative, there is confidence that the number of deposition positions that may be lost due to long fractures will be within the values used in the reference design. The reasons for this confidence are discussed in Section 8.4.

### *Assessment of inflow potential*

The methodologies used to estimate groundwater inflow into underground excavations are well established in hydrogeology. Analytical and numerical methods have been used to estimate the expected range of inflows for the reference design. The inputs for these methods were calibrated to in situ hydrogeology measurements for the site and hence there is confidence that these input values capture the hydraulic heterogeneity of the site.

In order to avoid piping erosion of the buffer, only deposition holes with limited inflows can be used. The current criterion, /Design Premises Long Term Safety, SKB 2009b/ is that the total volume of water flowing into a deposition hole, for the time between when the buffer is exposed to inflowing water and saturation, should be limited to ensure that no more than 100 kg of the initially deposited buffer material may be lost due to piping/erosion. This implies, according to the present knowledge, that this total volume of water flowing into an accepted deposition hole must be less than 150 $m^3$. There are various means of meeting this criterion, and it is judged to be met if deposition holes with inflows less than 0.1 L/min are avoided. Such inflows could only occur if the total transmissivity of fractures intersecting the deposition hole is larger than $4\times10^{-9}$ $m^2/s$ /Smith et al. 2007, Appendix B2/. Possible Observation Design approaches for meeting this criterion are:

a. Reject potential deposition position with inflows exceeding the inflow criterion of 0.1 L/min.

b. Reduce the inflows using grouting techniques to meet the criterion. However, the latter method is of questionable benefit for deposition holes, since the grout will not be stable in the long term.

c. Artificial wetting of the tunnel, which would decrease the saturation time, may be considered – as long as these actions are compatible with the design premises.

If the inflows exceed the allowable inflows in deposition tunnels or other parts of the repository, mitigative measures will be required to reduce the inflows to acceptable levels. It is anticipated /SKB 2007/ that this inflow reduction can be achieved using cement-based grouting. Grouting practice has been developed for traditional civil engineering projects where the quantities of grout and the type of material used to grout may not be strictly controlled. In a repository environment the current reference method for grouting will be limited to cement-based grouting and therefore the options available for controlling the groundwater inflows using grouting are limited, particularly if the water bearing fractures have relatively small apertures with channelized flow. In the current design traditional methods as well as new technologies have been used to estimate the grout quantities required to reduce the inflows to acceptable levels. However, there is uncertainty if these grouting methods are appropriate for the very low inflows specified. Grouting demonstration trials are currently in progress at the Äspö HRL but the final results were not available at the time of this writing and hence there is a low confidence in the grouting methods and the level of effort required to control these very low inflows.

The site investigations have shown that frequency of transmissive fractures at Laxemar is relatively high and heterogeneous. The hydrogeological DFN model was developed from findings and calibrated to the in situ measurements. Therefore there is reasonable confidence in the range of the transmissivity values and their distribution, at the repository level, developed from the hydrogeological DFN model. As a consequence the amount of grouting required to meet the specified inflow criterion will be substantial at Laxemar.

### *Assessment of spalling potential*

Rock mass spalling was described by /Terzaghi 1946/ as "popping rock" :

*"The term popping rock refers to rock formations from which thin slabs of rock are suddenly detached after the rock has been exposed in a quarry or a tunnel. Popping normally occurs only in hard rocks in an intact state. In tunnels the slabs are popped off either from the sides or from the roof of the tunnel. Popping has been encountered only in hard and brittle "rocks. It has invariably been found that the detached slabs do not fit the surface from which they popped."*

Spalling is commonly encountered in deep excavations in the mining and civil engineering projects and the process is well understood. The uncertainty in the prediction of the initiation of spalling in a typical repository rock mass was significantly reduced by the results from the Äspö Pillar Stability Experiment /Andersson 2007/. The methodology used to assess the spalling potential for the design was based on the findings reported by /Andersson 2007/ and the empirical methods developed for the mining industry. Because the methodology is empirically based, full scale deposition hole experiments similar to those conducted by /Andersson 2007/ may be needed at the repository level to confirm the design assumptions.

### 8.2.2 Constraints and assumptions impacting design

There are three primary design constraints that significantly influence the repository layout: (1) deformations zones requiring a respect distance, (2) minor deformation zones and (3) thermal rock mass properties and the approach used in the thermal dimensioning.

1. The reference layout includes provision for all the deterministic deformation zones identified in the site descriptive model. In the current layout there are no deposition positions in any of these deformation zones. In addition there is a respect distance of 100 m on either side of the borders of these deformation zones longer than 3 km and there are no deposition positions placed in this respect distance. It should be noted that the deformation zones at Laxemar are characteristically classed, as very good quality rock mass and excluding these deformation zones may be considered conservative. Utilising the very good quality rock mass portions of these deformation zones would reduce the footprint area of the repository.
2. For design stage D2, the minimum centre-to-centre spacing for the deposition tunnels is 40 m and the minimum centre-to-centre spacing for the deposition holes is 9 m in RSMA (2.25 W/m·K), 10.5 m in RSMM (2.05 W/m·K) and 8.1 m in RSMD (2.4 W/m·K). This spacing is based on the estimated thermal characteristics assigned to the rock and the criterion that the temperature in the buffer shall not exceed 100°C.

Figure 8-3 shows the relationship between deposition tunnel and deposition hole spacing for various thermal conductivities that meet this criterion. The reference D2 design uses a thermal conductivity values based on the tails of the distributions. As shown in Figure 8-3, the optimum spacing for the tunnels and deposition holes that satisfy 100°C temperature criterion depends on the thermal conductivity value that is considered representative of the rock mass. Optimizing the tunnel and the canister spacing for the thermal properties used in D2 would reduce the footprint area by approximately 25%. Thus there is an opportunity to optimise the deposition tunnel spacing once the thermal properties of the rock mass at the repository level are verified, that could result in a significantly reduced repository footprint.

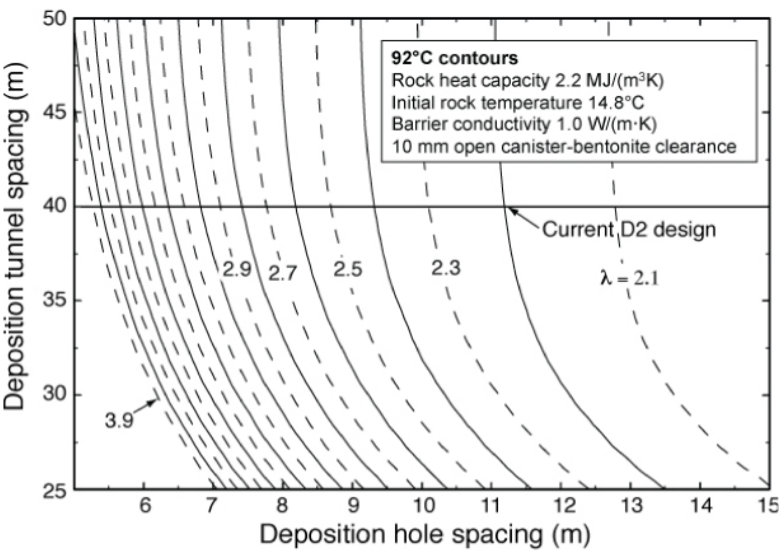



*Figure 8-3. Approximate deposition hole (canister) spacing versus deposition tunnel spacing for different thermal conductivity (λ) values at Laxemar /SKB 2008a/.*

## 8.3 Impact of uncertainty in site conditions on design

Site characterisation is one means of reducing risk to acceptable levels, yet routine site investigations can lead to wrong conclusions if the findings are not interpreted correctly. An essential step in a site characterisation program is the development of a geological model that captures the geological complexity of the site and is used as the basis for interpreting the findings from the site investigation. Geological complexity can exist at all scales. The Site Descriptive Model (SDM) should capture this complexity but this complexity may or may not impact the project design. Geological complexity implies that the geological description of the site, i.e. lithology domains and structural domains, varies spatially. The extent of this complexity and its potential impact on design dictates the site characterisation requirements, not the complexity alone. In this section the key findings from the site characterisation that impact the underground design are identified and the associated uncertainties described in the site descriptive model are assessed.

As described in Section 1 the repository design has been an iterative and stepwise process during the Site Investigations phase. The identification of hazards and uncertainties that may impact the design was also carried out during this period as an iterative stepwise process. As set out in the /SKB 2000a/ the investigation and evaluation of site was continued until the reliability of the site description reached sufficient confidence to conduct safety assessment and repository engineering. At the end of the site investigations the confidence in the site descriptive model was formally assessed in /SKB 2008b/. A brief summary of the remaining uncertainties discussed in /SKB 2008b/ is provided below in Table 8-1.

It should be noted that the uncertainty assessment of the SDM-Site in /SKB 2008b/ also discusses uncertainties in hydrogeochemical and transport conditions of the site. These issues are judged to be of no geotechnical significance for the underground design and layout.

### 8.3.1 Likelihood

The uncertainties, referred to as geohazards in this section, remaining in the Site Descriptive model and described in Table 8-1 may or may not impact the reference design D2. In order to evaluate the potential impact of a geohazard, the first step is an assessment of the likelihood of occurrence. The terminology for the likelihood of occurrence is expressed in terms of assessing the risk that the description of the site provided in the Site Descriptive model is incorrect. The four likelihood descriptors are described in Table 8-2 and range from “Extremely Unlikely” to “Very Likely”. These four categories for likelihood are in keeping with those recommended for qualitative risk analyses, e.g. /Australian Geomechanics Society 2000, Vose 2008/. The descriptor “Extremely Unlikely” in the context of the geohazards derived from the site uncertainties described in /SKB 2008b/ implies that there is simply no evidence from the site investigations to support the occurrence of the geohazard within the target area used for the design, while the descriptor “Very Likely” implies the geohazard is expected to occur. An evaluation of the site uncertainties was assessed in /SKB 2008b/ and the likelihood of the geohazard was independently assessed by an Advisory Expert Team (Figure 1-2). ). It was the latter that were responsible for the formulation of the geohazards and for the assigned likelihood of their occurrence.

A summary of the geohazards that are evaluated for their potential impact on the underground design and layout are listed in Table 8-3. The geohazards in Table 8-3 are grouped according to geology, hydrogeology, rock mechanics and in situ stress, and thermal properties. In addition to the lists of geohazards, the likelihood of its occurrence is also given based on the uncertainty description given in /SKB 2008b/. The likelihood descriptors were assigned based on the notion that the occurrence of the geohazard would be widespread throughout the target area and would therefore impact the entire repository design. The likelihood of the local occurrence of such geohazards was not evaluated, as a localised occurrence is not expected to cause a change to the overall repository design.

**Table 8-1. Summary of the uncertainties in Laxemar Site Descriptive Model at the end of the site investigations /SKB 2008b/.**

*Geological Uncertainties (*section 3.2 of R-08-101 /SKB 2008b/)

1. *Confidence and adequacy in the subdivision of Rock Domains in the potential repository volume.* The uncertainty relates to the location of the rock domain boundaries at depth between the boreholes, in particular for the boundary between RSMM01 and RSMA01, since there is no sharp contact between the Ävrö granite (RSMA01) and the Ävrö quartz monzodiorite (RSMM01), including the appearance of diorite/gabbro that also characterizes RSMM01.
2. *Alteration of intact rock*. The uncertainty relates to the spatial distribution and what is the true effect (difference) on e.g. thermal and rock mechanical properties between what is classified as fresh, faint, weak, medium and strong alteration (based only on qualitative inspection of the drill core during the mapping).
3. *Occurrence, geometry, character and properties of deformation zones, with trace length >3 km, inside the potential repository volume.* Overall there is high confidence in existence and location of the larger, layout determining deformation zones, but due to heterogeneity there is relatively high uncertainty in their character and physical properties.
4. *Occurrence of subhorisontal zones inside the potential repository volume.* Gently dipping deformation zones exist, but they are not modelled deterministically. At the elevations of interest they are considered to lie within the local minor deformation zone size range (i.e. less than 1 km) and are part of the DFN model.
5. *Size distribution of minor deformation zones (MDZ)*. MDZ will occur inside repository volume, and are modelled statistically as part of the geological and hydrogeological DFN models. However, the uncertainty in the size distribution for the MDZ is large.
6. *Geological DFN model inside the potential repository volume*. Size is the single largest uncertainty in the geological DFN model, whereas orientation, intensity, and spatial model are all well-treated and well-constrained. The size of sub-horizontal fractures is most uncertain, see point 4 and 5.

*Rock Mechanics model Uncertainties (*section 3.3 of /SKB 2008b, R-08-101/)

1. *In situ state of stress.* There is a good understanding of stress orientation. A relatively homogenous state of stress is expected in the focussed area/volume, but magnitude and orientation are expected to be locally influenced by structures, but the full extent of this local variability is uncertain. There is a very high confidence in the upper stress limit
2. *Intact rock mechanical properties.* There is a fairly large expected variation in strength, due to mineralogical variation/grain size distribution, within each rock type. The uncertainty in the proportions of different rock types in the M and A rock domain is fairly large, and thus the total expected strength distribution, on rock domain basis, is also fairly uncertain.

*Thermal model Uncertainties (*section 3.3 of /SKB 2008b, R-08-101/)

1. *True spatial variability of low conductive rock.* There are uncertainties in the lower percentiles of the modelled thermal conductivity distributions for rock domains RSMA01, RSMM01 and RSMD01. The thermal models are judged to slightly underestimate the lower tail of the thermal conductivity distribution.
2. *Geometrical bounds on different thermal subdomains*. Properties of Thermal subdomains are identified and modelled, but not geometrically modelled.
3. *Anisotropy in thermal conductivity* Thermal anisotropy exists and it is linked to foliation, but the magnitude of the anisotropy is uncertain.
4. *In situ temperature*. The quality approved borehole logging data indicates only small variation in temperature at repository depth, but this conclusion is based on limited data from four boreholes.

*Hydrogeological Uncertainties* (section 3.4 of /SKB 2008b, R-08-101/)

1. *Hydraulic properties of the rock mass (HRD) inside the potential repository volume.* There is a fair amount of data from rock domains and the fracture domains N, EW007, NE005, C, W ). There is a clear depth dependency and a basis for separating the data into different hydraulic rock domains (HRD), but the spatial variation within any division is rather high. The PFL-f statistics become more uncertain at greater depth as there are few boreholes within a domain and not all of them are drilled to greater depths. There is also great uncertainty in the orientation bias correction, since current data that stems from steeply dipping boreholes and the conductive fractures a steeply dipping too. The true inflow distribution and the true need for grouting in the underground constructions can only be fully known during construction and as assessed by pilot holes and probe holes during the tunnel excavation.
2. *The hydraulic properties of the deterministic deformation zones*, their spatial variability, anisotropy and scaling inside the target volume are uncertain.
3. *Hydraulic boundary conditions at regional scale are uncertain*. This is judged to have little importance for the flow conditions at the site, especially during excavation

**Table 8-2. Qualitative likelihood of occurrence terminology used to assess the geohazard risk to the underground design and layout.**

| Level | Descriptor |
|---|---|
| L1 | Extremely unlikely |
| L2 | Unlikely |
| L3 | Likely |
| L4 | Very likely |

**Table 8-3. Catalogue of geohazards in Laxemar evaluated during the Design step D2.**

| | **Geohazard: Geology model (The overall heterogeneity in the geological model is under estimated)** | |
|---|---|---|
| **Identifier** | **Descriptor** | **Likelihood** |
| G1 | Distribution of rock types deviates from the design value | Likely |
| G2a | Rock domain boundaries deviates from those used in the design | Very Likely |
| G2b | Deformation zone boundaries deviates from those used in the design | Very Likely |
| G3 | Frequency of long fractures exceeds the most likely values predicted by geoDFN Model and used in the design | Unlikely |
| G4 | New Deformation zones between 1 km and 3 km long trace length | Likely |
| G5 | New Deformation Zones requiring Respect Distance | Extremely Unlikely |
| G6 | Rock alteration (reduced rock quality) not well defined spatially | Likely |
| G7 | Width of MDZ<1 km exceeds the estimated value of 10 m | Likely |
| | **Geohazard: Hydrogeology model** | |
| **Identifier** | **Descriptor** | **Likelihood** |
| H1 | Frequency of water bearing fractures or Minor Deformation Zones (MDZ, <1 km) in the access area exceeds the most likely values used in the design | Likely |
| H2 | Transmissivity and complexity of deformation zones and MDZ(<1 km) at the repository level underestimated | Unlikely |
| H3 | Frequency of discrete flowing fractures or MDZ(<1 km), with flows unsuitable for deposition holes or deposition tunnels, is as estimated or greater | Likely |
| | **Geohazard: Rock Mechanics/In situ stress model** | |
| **Identifier** | **Descriptor** | **Likelihood** |
| R1a | Strength and Deformation Properties of the major and minor deformation zones less than the design values | Unlikely |
| R1b | Spatial variability of intact rock strength (Crack initiation strength) underestimated | Likely |
| R2 | Orientation of Shmax varies more than ±15 deg | Unlikely |
| R3 | Horizontal magnitudes exceed “most-likely” model but not the ”Unlikely maximum model” | Unlikely |
| R4 | Horizontal stress magnitudes exceed the “Unlikely maximum” model | Extremely unlikely |
| | **Geohazard: Thermal model** | |
| **Identifier** | **Descriptor** | **Likelihood** |
| T1 | Geometrical distribution of thermal rock domains deviates from the design value | Likely |
| T2 | The lower tail of the thermal conductivity distribution in the up-scaled model in the different rock domains is less than the design value | Unlikely |

## 8.3.2 Consequence

Having identified the geohazards and its likelihood, the next step requires an assessment of the consequence of the geohazard on the reference design and layout. The consequence of a geohazard occurring was assessed in qualitative terms, ranging from Insignificant to Major (see Table 8-4). The consequences for each of these consequence categories have been assessed according to the three functional areas of the repository: (1) Construction of the Repository Access, (2) Construction of the Central Area, and (3) Layout of the Repository, including loss of deposition positions. These consequences are described below.

### *Repository Access*

The quantitative consequence categories for Repository Access have been assessed using the planned Construction Schedule. The Repository Access is expected to take 5 years to complete. Delays in construction schedules can often be corrected by different construction procedures and are not classed as a “Major consequence” An increase to the construction schedule that occurs because of unforeseen site conditions is classed as a “Major Consequence” because it is caused by errors in the Site Descriptive Model.

### *Central Area*

The quantitative consequence categories for the Central Area have been assessed using the layout and stability of the caverns. The Central Area requires the construction of Caverns that vary in cross sectional area from 95 $m^2$ to 255 $m^2$. The stability of these caverns, which must remain functional for the life of the repository, is therefore important and has been judged during the design not to be a significant issue for the rock mass at Laxemar. If a concrete lining is required to provide a stable opening then the consequence is classed as "Moderate" as the support proposed during the design was underestimated. The position of these caverns must be determined prior to the commencement of the construction of the Repository Access. Hence changing the location of the Central Area outside the Building Permit Area, because of unforeseen geological conditions, is classed as a "Major Consequence" as it again reflects errors in the Site Descriptive Model. Modifying the orientation of the Caverns is classed as an "Insignificant Consequence".

The consequence categories for the Central Area could also have been established in terms of construction schedule (Project delays). However, opening more headings and applying more resources can easily correct a construction schedule for a series of caverns. While underestimating cavern support, which would also affect schedule, implies lack of understanding of site conditions, which is considered a more serious consequence.

### *Repository Area*

The Repository reference layout was developed to provide space capacity for 6,000 canisters. For design purposes the loss of deposition positions is expressed as a percentage or the percentage utilisation. For example, if the loss of deposition positions is expected to be 30% (70% utilisation), then the number of deposition positions required to accommodate this loss is 8,571 (8,571–6,000=2,571), i.e. 2,571 extra deposition positions will be required to meet the 6,000 canister requirement. The reasons for rejecting a deposition position are discussed below.

There are three safety related reasons for rejecting a potential deposition hole position:

1. During a future earthquake, shearing of a deposition hole may detrimentally impact the canister. According to the Design Premises Long Term Safety /SKB 2009b/: *Deposition holes are not allowed to be placed closer than 100 m to deformation zones with trace length longer than 3 km. Deposition holes should, as far as reasonably possible, be selected such that they do not have potential for shear larger than the canister can withstand. To achieve this, the EFPC criterion should be applied in selecting deposition hole positions.* The EFPC criterion /Munier 2006/ implies that canister positions intersected by fractures intersecting the full perimeter of the deposition tunnel or fractures intersecting five or more deposition holes should be rejected (see Section 8.2.1).
2. The potential groundwater flow to the deposition hole may results in unacceptable buffer erosion as described in Section 8.2.1. For the purposes of this preliminary design buffer erosion will not occur if deposition hole positions with inflows less than 0.1 L/min are avoided. Instead, it should be noted that most – if not all – of "high flow" positions are likely to be screened out by the EFPC criterion. In short, this means that the flow criterion would only marginally increase the loss of canister positions.
3. If there is too much spalling in the deposition holes, the hole geometry may be unsuitable. According to the Design Premises Long Term Safety /SKB 2009b/, *the initially place buffer mass should have a saturated buffer density of less than 2,050 kg/$m^3$ to prevent too high shear impact on the canister, and higher than 1,950 kg/$m^3$, to ensure a swelling pressure of 2 MPa.* According to /SKB 2010/, 5 cm of deposition hole overbreak is considered acceptable. Larger overbreak would need to be filled with, for instance, pieces of bentonite or with bentonite pellets before or during installation of the bentonite buffer.

The loss of deposition positions for each of these criteria in the D2 reference design is discussed in the following sections.

### 8.3.3 Potential loss of deposition positions

#### *Due to Long fractures*

The loss of deposition positions due to long fractures was evaluated using statistical approaches due to the stochastic and uncertain nature of the site description. SER /2009/ described the discrete fracture network (DFN) input in order to calculate the expected loss of deposition hole positions (or degree-of-utilisation) using various alternative DFN-models and the criterion given by /Munier 2006/. A value of 20% was proposed in SER /SKB 2008a/ as the "most likely" loss of deposition positions due to long fractures.

#### *Due to inflows*

The Laxemar SER/2009, concluded that acceptable deposition hole positions may not be located where the total transmissivity of intersecting fractures are larger than $4{\cdot}10^{-9}$ $m^2/s$ /Smith et al. 2007, Appendix B2/. As can be seen in Table 3-8 *(tabulated values showing distribution of* $T > 4{\cdot}10^{-9}$ $m^2/s$ *for each HRD)*, the potential loss of deposition hole positions are ~22% in HRD W, ~31% in HRD C and ~62% in HRD EW007. However, these positions are likely to coincide with the positions likely to be screened out by the EFPC criterion. In short, this means that the flow criterion dominates the cause to loss of depositions positions. The most likely situation is that very few additional deposition holes will be lost due to their intersection with long fractures, which are not already lost due to high inflows. The consequences for the viability of the Laxemar site is potentially major; the analyses suggest that approximately 35% of all positions could be lost due to high inflows. Furthermore, deposition within hydraulic domain HRD_EW007 is probably not advisable, since that part of the repository would require continuous grouting. If HRD_EW007 is discarded the potential loss is even higher than 35%.

#### *Due to spalling*

The in situ stress magnitudes are low relative to the uniaxial strength of the rock. Spalling is not anticipated for these stress magnitudes. This notion is supported by the experience with the underground openings to a depth of 450 m at the Äspö Hard Rock Laboratory. Nonetheless to reduce the potential for spalling the deposition tunnels are aligned between and 0 and 30 degree to the maximum horizontal stress.

No deposition positions are expected to be loss due to spalling for the most likely stress model.

### 8.3.4 Summary of consequences

The consequence of a geohazard occurring has been assessed in qualitative terms, ranging from Insignificant to Major (Table 8-4). In Table 8-4 the consequences are summarised for the three functional areas of the repository: (1) Construction of the Repository Access, (2) Construction of the Central Area, and (3) Layout of the Repository, including loss of deposition positions.

The Consequence categories for the loss of deposition positions were developed based on the degree of utilisation. A loss of deposition positions greater than the 2,031 currently provided for in the D2 layout is classed as a "Major Consequence" as a loss >2,000 would require additional rock volumes for deposition. A loss of deposition positions between 1,500 and 2,000 is the maximum loss expected in the current design and this is classed as a "Moderate" consequence. A loss of deposition positions between 1,000 and 1,500 is the expected loss in the current design based on current DFN models and this is classed as a "Minor" consequence. A loss of deposition positions <1,000 is less than expected using the most likely DFN model and is therefore classed as an "Insignificant" consequence for the layout.

It should be clear from the quantitative descriptions of consequence in Table 8-4 that the consequences associated with the loss of deposition positions should not be considered equivalent to delays in the construction schedule (Repository Access) or stability of caverns (Central Area). Delays in construction schedules or changes in underground support requirements are relatively minor consequences and can generally be resolved, while it is assumed that the space deemed unsuitable for placing the waste cannot be fixed using simple engineering solutions, without significant design changes.

**Table 8-4. Qualitative consequence terminology used to assess the geohazard risk to the underground design and layout.**

| Consequence Level | Descriptor | Repository Access (Construction Schedule Delays) | Notes: |
|---|---|---|---|
| C1 | Insignificant | <1 year | The reference design anticipated a 5 year time period for the construction of the access ramp and ventilation. The consequences are evaluated against that reference schedule. |
| C2 | Minor | 1–2 years | |
| C3 | Moderate | >2–3 years | |
| C4 | Major | > 3 year+<br>Delay caused by unforeseen site conditions | |

| Consequence Level | Descriptor | Central Area (Construction schedule, cavern stability and location) | Notes: |
|---|---|---|---|
| C1 | Insignificant | Orientation adjustment | Large caverns have been constructed at Clab Facility at shallow depth and at ÄspöHRL at depths of approximately 420 m in similar rock quality to that expected at Laxemar. The consequences are evaluated against the performance experience for those facilities which has been reported in /Carlson and Christiansson 2007/. |
| C2 | Minor | Cavern shape modified | |
| C3 | Moderate | Concrete lining needed for stability | |
| C4 | Major | Central Area moved requiring land outside the building permit | |

| Consequence Level | Descriptor | Repository Area (Loss of deposition hole positions) | Notes: |
|---|---|---|---|
| C1 | Insignificant | <1,000<br>(Less than expected) | The current layout has a gross capacity of 8,031 deposition positions. This capacity takes into account the loss of deposition positions from all geohazards. Exceeding the gross capacity of the site would have major consequences. |
| C2 | Minor | 1,000–1,500<br>(Expected loss) | |
| C3 | Moderate | 1,500–2,000<br>(Maximum loss with current design) | |
| C4 | Major | >2,000<br>larger loss than current design<br>(exceeds gross capacity) | |

## 8.4 Qualitative risk assessment of site uncertainties on design

Risk assessment in the design process can be defined as the combination of the two basic components: (1) input uncertainties and (2) possible consequences. As there are many facets of these components, there is often a broad perspective on risk, reflecting for example that there might be different assessments of uncertainties, as well as different views on how these uncertainties should be dealt with. Qualitative risk assessment uses descriptive word form to describe the magnitude of the potential consequences and the likelihood that those consequences will occur. The risk assessment is the process of making a decision recommendation on whether existing risks are tolerable and present risk control measures are adequate, and if not, whether new risk control measures need to be developed. Qualitative risk assessment is subject to limitations as the risk is judged. However, despite its limitations the FMEA approach for risk assessment is a well-established process for assessing the safety of systems during the design stage /Rausand and Høyland 2004/. The method is inductive; for each component of the system, we investigate what happens if the geohazard occurs. The method represents a systematic analysis of the components of the design to identify all significant failure modes and to see how important they are for implementing the reference design. Only one component is considered at a time, and the other components are then assumed to function as designed. One of the primary functions of the FMEA at this stage is to identify elements of the design that may need to be modified if the design assumptions are proven to lie outside those used for the reference design. This aspect of FMEA encompasses the requirements of the Observational Method. During the next design step quantitative approaches may be required combined with event tree analyses for particular scenarios.

### 8.4.1 Qualitative risk matrix

A risk matrix is a simple method of presenting the results from an FMEA analysis that expresses the likelihood-consequence analyses for each geohazard (Figure 8-4). This type of binning of the various geohazards evaluated provides a means of ranking the hazards and visualizing the results /Vick 2002/. Such a figure provides a matrix for identifying the geohazards that require additional investigation/analyses during the next design step. It also provides an effective means for identifying the issues that need careful attention and planning during construction as part of the Observational Method. It must be remembered that because of the stepwise process used during the design many of the issues that were identified during the early stages of the design were resolved as additional site information was obtained and the Site Descriptive model updated. Hence for the risk matrix in Figure 8-4 only two categories of risk are identified:

(1) Risk Class N/A – where the risks to the design are considered Negligible and/or Acceptable, and

(2) Risk Class DM – where the risks to the design are such that if the geohazard occurs the design may need modification and therefore mitigative measures and monitoring plans must be developed.

It must be remembered that the risks described here are the risks that the reference design presented in D2 will need significant modifying. This could occur if additional site information collected during the next design step and/or during construction changes the parameters used in the design D2 for a particular geohazard. These design–related risks should not be associated with the risks described and assessed in the Safety Assessment. Risk Category N/A in Figure 8-4 reflects minor changes to design D2 and these changes are within the anticipated ranges used to establish the reference design D2. Risk Category DM reflects major changes that may be necessary to the reference design because of major changes to the site descriptive model in SDM Site. These "Major Consequences" are such that the mitigating actions may require a re-assessment of the long term safety consequences. Risk Category DM indicates that the design should develop migitative plans in case the geohazard scenario evaluated occurs. A geohazard scenario that is assigned Risk Category DM also suggests that additional site information should be collected as soon as practical to resolve the uncertainty with the geohazard.

The geohazards listed in Table 8-1 that could impact the three functional areas of the repository have been evaluated using the risk matrix approach described above. Table 8-5 provides a summary of the risk ratings for the three functional areas and theses risk ratings are discussed in the following sections.

### 8.4.2 Repository Access

The Repository Access must be constructed before the Central Area can be prepared and before deposition can commence. As noted previously the consequences of the geohazards for the Repository Access were evaluated in terms of schedule delays. The consequences of geohazards G1 G2a, G2b, G4, G5, G6, G7 G3, R1a, R1b, R2, R3, R4, G4 and R4, while having varying classes

| Likelihood | | | | | |
|---|---|---|---|---|---|
| | Very likely | N/A | N/A | DM | DM |
| | Likely | N/A | N/A | N/A | DM |
| | Unlikely | N/A | N/A | N/A | DM |
| | Extremely Unlikely | N/A | N/A | N/A | N/A |
| | | Insignificant | Minor | Moderate | Major |
| | | Consequence | | | |

*Figure 8-4. Illustration of the binning approach used to highlight risks using the qualitative Likelihood and Consequence risk matrix.*

of likelihood, were assessed as insignificant (<1 year schedule delay). The primary concern for the construction of the Repository Access is delays due to water inflows and the associated grouting. The likelihood of geohazard H1 (Frequency of water bearing fractures or minor deformation zones (MDZ<1 km) is underestimated) is ranked as "Likely". Because the number of boreholes is limited in the access area, it is likely that the number of water-bearing features, which will require grouting, is underestimated and could result in moderate construction delays to Repository Access.

The detailed design will need to focus additional site investigation on the hydrogeology conditions for the Repository Access and utilise probe holes during construction, as part of the Observational Method, to minimise the potential impact of water bearing fractures and MDZ.

**Table 8-5. Likelihood and consequence results for the geohazards listed in Table 8-2 for each functional area in Laxemar.**

| Repository Access | | | | | |
|---|---|---|---|---|---|
| Likelihood | Very likely | G2a,G2b, | | | |
| | Likely | G1,G4G6,G7 R1b | | H1 | |
| | Unlikely | G3,R1a,R2,R3 | | | |
| | Extremely Unlikely | G5,R4 | | | |
| | | **Insignificant** (<1 year) | **Minor** (1–2 year) | **Moderate** (2–3 Years) | **Major** (>3 Years+) |
| | | **Consequences (Schedule Delays)** | | | |

| Central Area | | | | | |
|---|---|---|---|---|---|
| Likelihood | Very likely | G2a | G2b | | |
| | Likely | G1,R1b | G4,G6,G7 | | |
| | Unlikely | G3,H2,R2 | R1a,R3 | | |
| | Extremely Unlikely | R5 | R4 | G5 | |
| | | **Insignificant** (Orientation adjustment) | **Minor** (Cavern Shape Modified) | **Moderate** (Concrete lining needed for stability) | **Major** (CA moved requir-ing land outside the building permit) |
| | | **Consequences (Stability & Location)** | | | |

| Repository Area | | | | | |
|---|---|---|---|---|---|
| Likelihood | Very likely | G2a,G2b | | | |
| | Likely | G1,G4,G6,R1b,T1 | | | H3 |
| | Unlikely | R1a,R2,R3 | G7,H2,T2 | G3 | |
| | Extremely Unlikely | | R4 | | G5 |
| | | **Insignificant** (<1,000) | **Minor** (1,000–1,500) (Most likely) | **Moderate** (1,500–2,000) | **Major** (>2,000) |
| | | **Consequences (Loss of deposition positions)** | | | |

### 8.4.3 Central Area

The primary concern for the Central Area is the stability of the caverns that vary in cross sectional area from 95 $m^2$ to 255 $m^2$. The consequence of geological geohazards G1, G2a and G3, the hydrogeology geohazard H2, and the rock mechanics geohazard R1b and R2 on the stability of the caverns is considered "insignificant". Because the transport shaft will be completed in the vicinity of the caverns the rock mass conditions will be known prior to cavern construction. The instrumentation that will be installed as part of the Observational Method during shaft and ramp construction (see section 8.5) can be used to assess the need for adjustments in orientations of the caverns relative to the maximum horizontal stress (R2).

The consequences of the geological geohazards G2b, G4, G6 and G7, and the rock mechanics geohazard R1a, R3 and R4 are classed as "Minor". This implies that these geohazards can be handled by modifying the cavern profile combined with orientation adjustment. The construction experience at Clab facility and Äspö Hard Rock Laboratory in similar rock quality suggests that large caverns can be excavated with minimum support.

The likelihood of geological geohazards G5 (new deformation zones requiring respect distance) is classed as "Extremely unlikely" because there is no evidence from the Site investigations that such a zone could exist in the target volume. Should such a new deformation zone be intersected in the largest caverns, a concrete lining may be needed for cavern stability, and hence the consequence is classed as "Moderate".

The location of the Central Area is dictated by the location of the transport shaft. Additional geological information will be available from the site investigations for the detailed design for the Repository Access. In addition, the shaft excavation will also provide additional information on rock mass quality prior to the construction of the Central Area. Hence much of the uncertainty associated with the geohazards noted here could be resolved prior to construction of the Central Area.

### 8.4.4 Repository area

The primary consequence for the repository area is a loss of deposition positions greater than that used for the reference design. The current design can accommodate 8,031 deposition positions, i.e. 2,031 beyond the 6,000 deposition holes required for the reference design layout. The geohazards G1 (distribution of rock types), G2a (geological boundaries), G2b (Deformation zone boundaries) and G4 (New deformation zones between 1 km and 3 km long trace length) have varying degrees of likelihood but the consequences of these geological geohazards are considered "insignificant", that is the loss of deposition positions due to these geohazards is expected to be less than 1,000. Similar consequences have been assigned to R1a (Strength and deformation properties of deformation zones), R2 (Orientation of maximum horizontal stress) and R3 (Horizontal stress magnitudes exceed the most likely model but not the unlikely maximum model), although all these geohazards are classed as "unlikely".

The consequences of geohazard G7 (Width of MDZ < 1 km underestimated), H2 (Transmissivity and complexity of deformation zones and MDZ underestimated) and T2 (Thermal conductivity of rock mass is less than the design value) are classed as minor (1,000–1,500) as this number of deposition positions is considered the most likely values based on the design parameters. The geohazard R4 (Horizontal stress magnitudes exceed the unlikely maximum) is considered "extremely unlikely" as there is no evidence from the site investigations in the Oskarshamn region that such stress magnitudes occur. In addition because of the different rock domains, should these magnitudes occur they are likely to be limited to the best quality rock mass, which makes up about 1/3 of the repository footprint. Hence the consequence of R4 is also classed as minor.

The Consequence of the geology geohazard G3 (Frequency of long fractures) is classed as Moderate (1,500–2,000) but is considered "Unlikely" to occur. There is reasonable confidence in the geological DFN model for the Laxemar area as there are considerable surface bedrock exposures in the area and some calibration of the geological DFN model was carried out using the underground openings at the Äspö HRL. Furthermore, the current EPFC criterion /SKB 2009b, Design Premises Long Term Safety/ is probably overly restrictive. Improved means of characterising the size of large fractures, would allow a more appropriate criterion and a smaller loss of deposition positions.

The occurrence of geohazard G5 (New deformation zones requiring a respect distance) is considered extremely unlikely. There is simply no evidence from the site descriptive model that such a deformation zone could exist. Should it occur the consequence is classed as “major” since it implies a significant change in the SDM Site that would have a major impact on the layout. Should these conditions occur alternative design arrangements would have to be evaluated.

The occurrence of geohazard H3 (frequency of inflows unsuitable for deposition holes or deposition tunnels, are as estimated or higher) is considered “likely”. The modelled transmissivity distribution of water bearing fractures intersecting the deposition holes are based on the hydrogeological Discrete Fracture Network (hydro-DFN) /Rhen et al. 2008/. The hydro-DFN model is calibrated to hydrogeology measurements for the site and hence there is reasonable confidence in the estimated transmissivity distribution. However, the modelling approach may slightly underestimate the number of intersections, which motivates the chosen likelihood category for geohazard H3. The consequence for geohazard H3 is classed as “major” as a loss of more than 2,000 deposition positions would require evaluation of alternative design arrangements.

## 8.5 Implementing the Observational Method

It was shown in Figure 8-1 that the Observational Method was comparable to a risk management process that is well suited for managing the uncertainties associated with the design and construction of a geological repository. The Observational Method has two caveats: (1) one must be able to define an action plan for every possible adverse condition based on current site understanding; and (2) the method cannot be used if a predictive model for the behaviour cannot be developed, i.e. it is necessary to establish a model that can calculate the parameters that will subsequently be monitored during construction. This is not a trivial problem as often we can measure what we cannot calculate and vice versa. This means that the monitoring plan that will be used to verify the design assumptions in the reference design D2 must be chosen very carefully with a good understanding of the significance to the problem.

The reference design has thus far addressed the first two requirements of the Observational Method: (1) **acceptable limits of behaviour,** and **(2) evaluated the range of possible behaviour**. In the following sections the remaining 3 elements of the Observational Method: (1) a **plan for monitoring the behaviour**, (2) the **response time of the monitoring** and (3) a **plan of contingency actions** are discussed.

### 8.5.1 Monitoring requirements

Table 8-3 lists the geohazards that were evaluated during the course of the reference design D2 and Table 8-5 rated the likelihood and consequences of these geohazards. While many of the geohazards did not impact the design, the monitoring program that will be established for the construction of the repository must evaluate all the geohazards listed in Table 8-3 regardless of their likelihood.

Table 8-6 lists the parameters that should be measured during the construction of the repository to establish if the design assumptions are valid. Table 8-6 lists the geohazard, the parameter that will be assessed and a general description of the technology that can be used to conduct the assessment. Also included are the general locations where the measurements should be carried out and a suggested frequency for those measurements. It should be noted that the parameters listed in Table 8-2 are only intended to address the uncertainties noted in the design-risk assessment. Other parameters that may be monitored to meet the objectives of the detailed site investigations, e.g. serving additional needs for future long term safety assessments, are described in /SKB 2009b/ and not discussed in this report.

A detailed design will be carried out prior to start of repository construction. During this detailed design detailed monitoring plans will need to be developed for each of the remaining design uncertainties. In addition, the detailed design will need site-specific investigations for the Repository Access shafts and ramps and for the Central Area. These investigations will also provide additional geotechnical information that will reduce the uncertainties associated with some of these geohazards in those areas.

**Table 8-6. A list of parameters that should be measured/quantified during repository development to assess the uncertainty in the design assumptions. An assessment of these parameters must be carried out as part of the Observational Method.**

| Uncertainty descriptor | Parameter(s) to be assessed | Technology to be used | Location and Frequency of assessment |
|---|---|---|---|
| **Geohazard: Geology model** | | | |
| G1: Distribution of rock types not correct | Geological Descriptors | Borehole logging & Tunnel mapping | All investigative boreholes and underground excavations regardless of locations |
| G2a: Rock domain boundaries deviate from those used in the design | Geological Descriptors | Borehole logging & Tunnel mapping | All investigative boreholes and underground excavations regardless of locations |
| G2b: Deformation zone boundaries deviate from those used in the design | Geological Descriptors | Borehole logging & Tunnel mapping | All investigative boreholes and underground excavations regardless of locations |
| G3: Frequency of long fractures greater than that predicted by DFN model | Geological Descriptors | Borehole logging & Tunnel mapping | All investigative boreholes and underground excavations regardless of locations |
| G4: New Deformation zones between 1 km and 3 km long | Geological Descriptors | Regional modelling | All investigative boreholes and underground excavations regardless of locations |
| G5: New Major Deformation Zones requiring Respect Distance | Geological Descriptors | Regional Modelling | All investigative boreholes and underground excavations regardless of locations |
| G6: Rock alteration (reduced rock quality) not well defined spatially | Geological Descriptors | Regional Modelling | All investigative boreholes and underground excavations regardless of locations |
| G7: Width of MDZ<1 km is under estimated | Geological Descriptors | Regional Modelling | All investigative boreholes and underground excavations regardless of locations |
| **Geohazard: Hydrogeology model** | | | |
| H1: Frequency of water bearing fractures or MDZ underestimated | Water Inflow to excavations | Exploratory and Probe hole drilling | All investigative boreholes, probeholes and excavations |
| H2: Transmissivity and complexity of deformation zones at repository level underestimated | Water Inflow to excavations | Exploratory and Probe hole drilling | All investigative exploratory boreholes, probeholes and excavations at repository elevation |
| H3: Frequency of discrete flowing fractures, with flows unsuitable for deposition holes or deposition tunnels, under estimated | Water Inflow to excavations | Exploratory and Probe hole drilling | All investigative exploratory boreholes, probeholes and excavations in the repository area |
| **Geohazard: Rock Mechanics/In situ stress model** | | | |
| R1a: Properties of the major and minor deformation zones | Kn/Ks | Convergence | To be carried out at repository depth. Detailed plans will have to be developed |
| R1b: Spatial variability of intact strength | Geological Descriptors and Lab testing | Regional Modelling | All investigative boreholes and underground excavations regardless of locations |
| R2: Orientation of Shmax | Shmax | Convergence | To be carried out in shafts below 100 m depth. The frequency of measurements must be sufficient to establish the orientation variability at the repository elevation. |
| R3/R4: Horizontal stress magnitudes | Shmax/ Shmin | Convergence/ Overcoring | Measurements to be conducted in the shaft and ramp access below 200 m depth. Frequency of measurements must be sufficient to confirm design assumption at repository elevation |
| **Geohazard: Thermal model** | | | |
| T1: Distribution of thermal rock types not representative | Rock type | Sampling | Measurements to be conducted in the Central Area and repository area. Frequency of measurements must be sufficient to confirm design assumption |
| T2: Up-scaled thermal conductivity is less than the design value | Rock type | Sampling | Measurements to be conducted in the Central Area and repository area. Frequency of measurements must be sufficient to confirm design assumption |

### 8.5.2 Response time and contingency design plans

As noted in the Observational Method, once monitoring plans are in place the final stages are: (1) the response time for the monitoring and (2) a plan for contingency action for design alternatives, should the monitoring program indicate that the in situ conditions are outside the range of values used for the design. Figure 8-5 shows the tentative schedule for the construction of the underground excavations that will provide Repository Access and the Central Area. Also shown in Figure 8-5 are the geohazards listed in Table 8-2 and the monitoring period available to establish if design alternatives are required. It should be noted that the uncertainty associated with the stress magnitudes and orientations could be resolved by the end of shaft construction and provide ample time to develop alternative layout plans if required. Hence it is important the contract documents for shaft sinking incorporate plans that will provide for convergence measurements and/or stress measurements during sinking operations.

The detailed plans for contingency action, i.e. alternative designs, must be developed during the next design step.

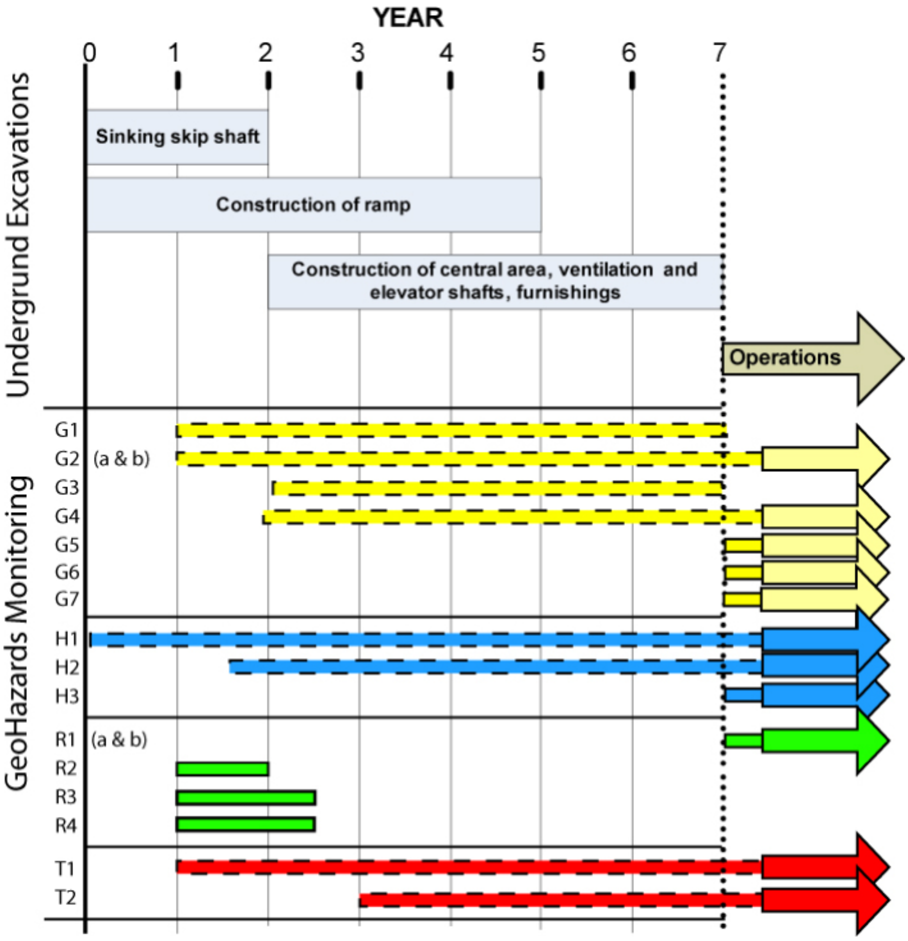



*Figure 8-5. Tentative schedule for the construction of the Access ramp and shaft, and the Central Facility. Also shown are the monitoring periods for the geohazards listed in Table 8-2.*

## 8.6 Summary

The primary objective of design D2 is to provide a constructible repository that has space for 6,000 canisters and meets the long term safety requirements. The purpose of this chapter was to evaluate site uncertainties (geohazards) and assess the risk associated with these uncertainties in achieving this primary design objective. The site uncertainties were established from the summary of the uncertainties in the Site Descriptive Model at the end of the site investigations /SKB 2009a/. The likelihood that these uncertainties would occur was ranked from “Extremely unlikely” to “Very likely”, and the consequences of the uncertainties were assessed for each for the three functional areas in the repository. The consequences were evaluated using the planned construction schedule for the Repository Access, the stability of the caverns for the Central Area and the loss of deposition positions for the Repository layout. It should be noted that the consequences associated with the loss of deposition positions are not considered equivalent to delays in the construction schedule (Repository Access) or stability of caverns (Central Area). Delays in construction schedules or changes in underground support requirements are relatively minor consequences and can generally be resolved, while it is assumed that the space deemed unsuitable for placing the waste cannot be fixed using simple engineering solutions, without significant design changes.

The most significant uncertainties at Laxemar are related to the amount of grouting required to meet the inflow criteria in deposition tunnels and the potential loss of deposition positions due to inflows. It is uncertain if traditional grouting methods alone can be used to reach the sealing efficiency specified for the deposition tunnels. Reaching the sufficient efficiency is likely to require reliance on new technologies that were recently tested at the Äspö HRL. In addition, the utilization of the Deposition Area HRD EW007 will be low as there is currently no proven method for sealing inflows to deposition holes that complies with the design premises for long term safety. In summary, because of the expected distribution of the transmissive fractures, there is low confidence in the ability of traditional grouting methods to provide the relatively dry environment required for deposition. Alternative design arrangements would have to be evaluated to ascertain one of the primary objectives of design D2, i.e. to provide a repository that has space for 6,000 canisters.

It is clear from the analyses in this chapter that no amount of surface investigations or sophisticated design analyses will remove the design risk and uncertainties to zero. It is simply not achievable for an underground project at the scale of a nuclear waste repository. Using most-likely and/or conservative design parameters in conjunction with various design tools, combined with uncertainty and risk analyses, suggests that a repository may be constructed that will meet all safety requirements, but it is very likely that the capacity of the layout will be less than 6,000 canisters, unless additional rock volumes for deposition is utilized. Based on most-likely estimates for the design parameters the capacity of the layout could be reduced to approximately 4,000 canisters.

# 9 Conclusions

The Complete Site investigations for the Laxemar site were completed in 2008 and the findings summarised in SDM Site /SKB 2009a/. During the site investigation, several studies and design steps were carried out to ensure that sufficient space was available for the deposition requirements within the local model volume. The findings from design Step D2, the subject of this report, for the design and layout for the underground facility including the access ramp, shafts, rock caverns in a Central Area, transport tunnels, and deposition tunnels, are summarised below.

## 9.1 General

1. The design and layout of the repository have been carried out using the principles of the Observational Method.
2. A 6,000-canister layout has been developed at a maximum Elevation of –500 m. The layout has a gross capacity of 8,031 deposition positions, which provides for a loss of 2,031 deposition positions (approximately 25%).
3. This layout incorporates all the deterministic deformation zones and the respect distance for the deformation zones longer than 3 km. No deposition positions are located in these zones.
4. The behaviour of the underground openings is expected to be similar to the performance of other underground openings in the Scandinavian shield at similar depths. The dominant mode of instability is expected to be either:
   a. Structurally controlled wedge failure and/or
   b. Stress-induced spalling.

   Stability of the openings will be achieved with traditional underground rock support and by orienting the openings relative to the maximum horizontal stress.
5. The layout of the repository area has the deposition tunnels aligned <30 degrees relative to the maximum horizontal stress. The orientation reduces the potential for spalling for the deposition tunnels and the deposition holes. Spalling is not anticipated in the deposition tunnels at this orientation or in the deposition holes.
6. The layout design separated the deposition and construction activities using the separation by linear development. The linear development method is considered to provide the greatest excavation flexibility and meet all operational requirements.
7. Summary of the layout dimensions and volumes:

| **Description** | **Quantity** | |
|---|---|---|
| Layout area ($km^2$) | 5.7 | |
| Repository Elevation (m) | –500 | |
| Gross capacity (deposition positions) | 8,031 | |
| Repository level: | **Volume** (…×$10^3$ $m^3$) | **Length** **(km)** |
| Transport tunnels | 230 | 6.5 |
| Main tunnels | 515 | 8.6 |
| Deposition tunnels | 1,824 | 95 |
| Deposition holes (6,000) | 115 | 48 |
| Central Area and access (…×$10^3$ $m^3$) | 324 | |
| **Total** | **3,008** | **158.1** |

## 9.2 Current Design Constraints

1. The location of the surface facilities was limited to the southeast part of the local model volume and maintained a distance of 200 m from the existing 400 kV transmission lines.
2. The layout was constrained to the Local Model Volume that was investigated and reported in SDM Site Laxemar /SKB 2009a/. Other suitable volumes may exist adjacent to the boundaries of the Local Model Volume but these have not been considered in these studies.

   The space for the repository has been determined using the thermal dimensioning guidelines developed for the site. The centre-to-centre spacing for the deposition tunnels was set at 40-m and the canister spacing was selected that met the thermal guidelines using the low tail of the thermal property distribution. The current canister spacing is 9 m in rock domain RSMA01, 10.5 m in RSMM01 and 8.1 m in RSMD01. These rather wide canister spacing reflect the lack of quartz in the rock domains. This approach does not optimise the footprint of the repository.
3. No deposition positions were located within the 100-m Respect Distance allocated to major deformation zones (>3 km trace length). In addition, no deposition positions were located in deterministic deformation zones <3 km in trace length.

## 9.3 Expected site conditions

### *Repository Access*

The excavations for the Repository Access will be located in hydraulic domain HRD_C. These excavations will encounter the greatest frequency of highly conductive water bearing fractures (0.564/m) between 0 and 150 m, requiring extensive grouting and probe drilling.

The excavation of the Repository Access ramp and shaft(s) will result in a groundwater drawdown. Detailed geotechnical information is required to assess if grouting would be sufficient to prevent negative environmental consequences. Alternative measures may need to be evaluated in the next design step as a means of meeting the environmental objectives if grouting is shown to be inadequate.

### *Central Area*

The caverns and tunnels for the Central Area will be excavated at the approximate depth of the repository. At this depth the rock mass is expected to encounter water bearing fractures spaced approximately 0.107/m. The cavern orientations have been aligned parallel to the maximum horizontal stress to minimise the tangential stresses on the excavations.

### *Repository Area*

There is a number of steeply dipping deterministic deformation zones that intersect the repository. Five of these require a Respect Distance of 100 m. These cross cutting geometric constraints significantly impact the layout and development of the facility, and the utilization of available space.

The rock mass at the repository horizon is expected to contain water bearing fractures spaced approximately 1 every 5 to 15 m and the distribution of transmissivity values for these fractures is expected to range from $<10^{-9}$ to $>10^{-6}$ $tm^2/s$. These transmissivity values will produce groundwater inflows that will exceed the allowable inflow to a deposition tunnel; requiring extensive groundwater control measures to control/reduce these inflows.

The orientation of the deposition tunnels have been aligned parallel to the maximum horizontal stress to minimize the tangential stresses on the deposition tunnel and deposition hole excavations. The main tunnels will be approximately perpendicular to the maximum horizontal stress. If spalling is encountered the shape of the main tunnel can be used to reduce the extent of spalling.

#### *Ground Control*

The results from the analyses all indicate that conventional underground support measures would be sufficient to ensure that the performances of the underground openings are acceptable. The estimated amount of support is on average very low because of the good quality rock mass anticipated.
This conclusion is also supported by the experience at the Clab Facility and the Äspö Hard Rock Laboratory.

#### *Groundwater Control and Grouting*

The results from the analyses indicate that conventional cement grouting measures will generally not be sufficient to meet the inflow criterion. New grouting technologies, e.g. silica sol will be required in most circumstances to meet the inflow criterion. Comprehensive grouting works are expected at all depths and in the deposition tunnels. Groundwater-control measures, e.g. cut-off grout curtain, may be required before commencement of the excavation works for ramp and shafts. Such options should be evaluated during the Detailed Design.

## 9.4 Uncertainty in site conditions impacting design

1. The technical risk assessment presented in Chapter 8 concludes that the available gross capacity of about 8,000 deposition positions cannot be judged sufficient to host a repository with 6,000 deposited canister, without significant design changes. The water inflows to many deposition holes are judged to be higher than the accepted values. The problems are worse at the hydraulic domain (HRD_EW007), which in the current repository layout accounts for about 2,000 positions, but the loss of positions are high (20–30%) also in other domains. This risk can primarily be handled by avoiding HRD_EW007 and by revising the thermal dimensioning such that the remaining area is used more efficiently, although at the expense of more deposition tunnels to be constructed. It may also be necessary to utilize additional rock volumes for deposition. Even considering these design changes there will be a substantial need for grouting and special methodology will be needed in order to reach sufficiently small inflows. Such methodology has recently been developed by SKB, but it remains to be developed into an industrial scale.
2. The in situ stress conditions at the depth of the repository are not expected to be sufficient to cause spalling on the deposition holes using either the “most likely” or “possible maximum” stress models.
3. There is a general lack of confidence in predicted loss of deposition positions using the geological DFN for the repository Level. There is greater confidence in the hydrogeological-DFN model since it is calibrated on a multitude of hydraulic tests carried out at the potential repository level. The hydrogeological DFN model supports the overall geological and hydrogeological model for the site and indicates the at the repository level the frequency of water bearing fractures is approximately 1 water bearing fracture every 5 to 15 m. In hydraulic domain HRD_EW007 the frequency is higher. This implies that comprehensive grouting may be required for the deposition tunnels.

## 9.5 Implementing the Observational Method in the next design step

1. A preliminary implementation plan for the Observational Method is outlined that illustrates how uncertainty in the design parameters could be reduced during construction of the repository. During the next design step these plans must be fully developed using “means and methods” statements that clearly describe:
   a. What will be measured and how.
   b. Location of measurements.
   c. Frequency of observations.
   d. Interpretation and reporting of results.

## 9.6 Feed-back to future design, safety assessment and site investigations

1. A site investigation plan will be needed for the access shaft and ramps to finalise the location of these openings. This investigation should focus on the geotechnical information needed for detailed design. The results from that investigation should be used to conduct a detailed FMEA analysis of the geological uncertainties (tunnel and shafts, stability and seepage) impacting the Repository Access.
2. The need for groundwater control measures during and after ramp/shaft construction should be assessed. Alternative solutions such as a grout curtain cut-off and/or measures to preserve the groundwater table using surface infiltration techniques should be evaluated.
3. Monitoring plans, as part of the Observational Method, should be developed during detailed design to reduce the uncertainty in the stress magnitudes and orientations at the repository level and executed during the construction of the access ramp and shaft.
4. Establish design rules for establishing the width of deformation zones <3,000 m that must be avoided for deposition positions.
5. Evaluation of the thermal guidelines/constraints for the site should be carried out. Optimising the thermal design by reducing the deposition tunnel spacing and increasing the canister spacing, using the current thermal design properties, has the potential to reduce the footprint of the repository by approximately 30%.
6. The current layout will develop the depositional area located in hydraulic domain HRD_EW007 at the end of the repository life. The greatest potential for the loss of deposition positions occurs in this region. The detailed design should ensure that the HRD_EW007 Deposition Area is constructed last to allow ample time for optimising the layout.
7. The proposed development of the repository advances away from the Central Area towards the eastern boundary. This means that the waste will be placed around the Central Area first. The temperatures generated by the waste increase rapidly in the first 10 years and peak around 50 years. This implies that the Central Area caverns and the permanent access excavations will be subjected to thermal loads that have not been evaluated in this design. In the Detailed design the effect of thermal loading on all permanent excavations should be evaluated.

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# Appendix A

## Typical drawings for the repository underground excavations

### Table of Contents

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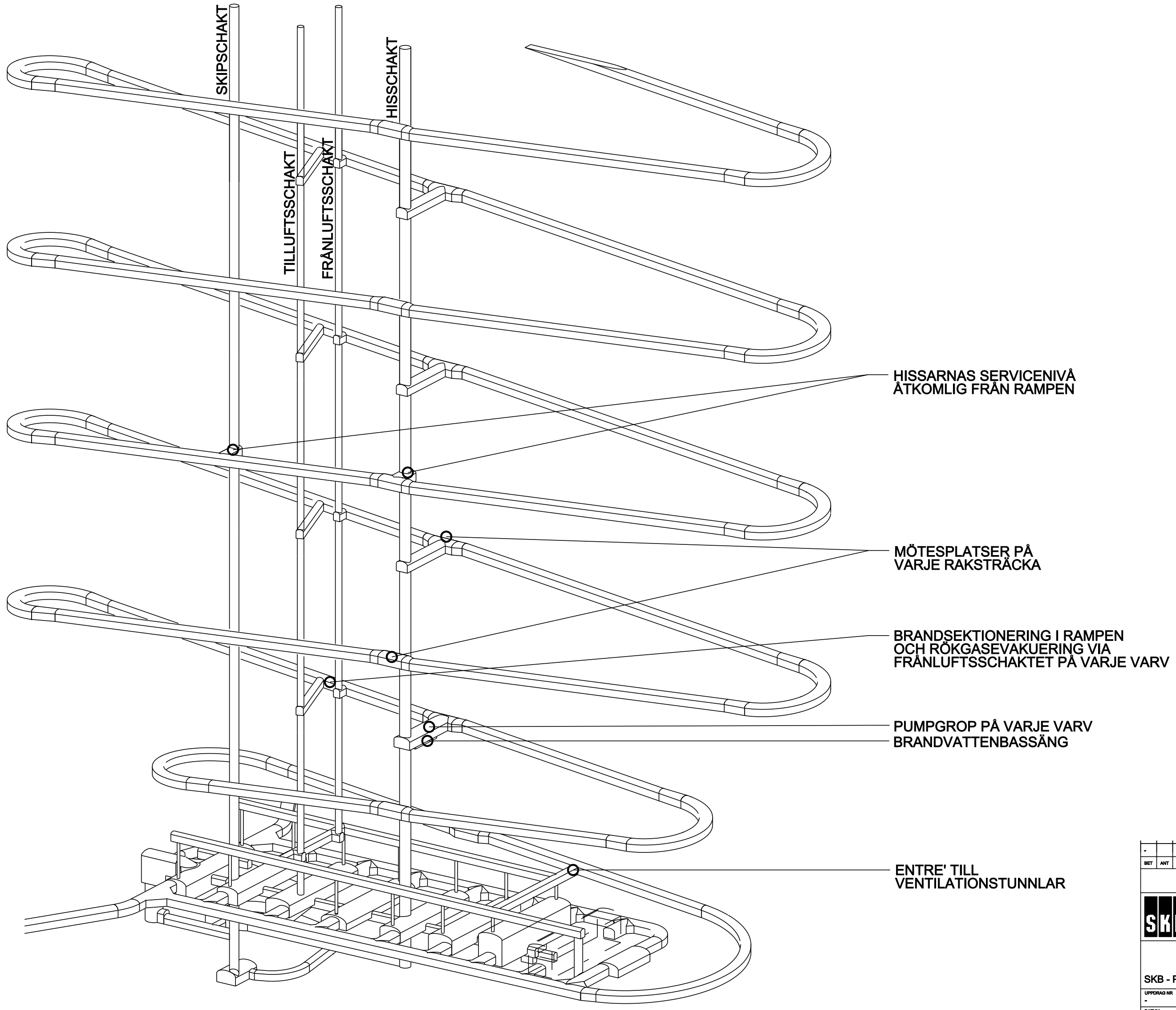


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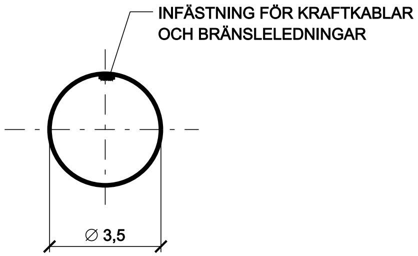


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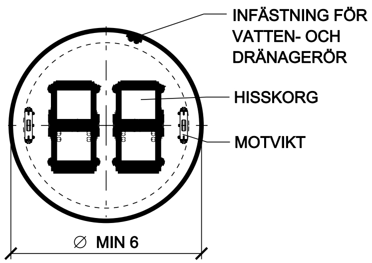


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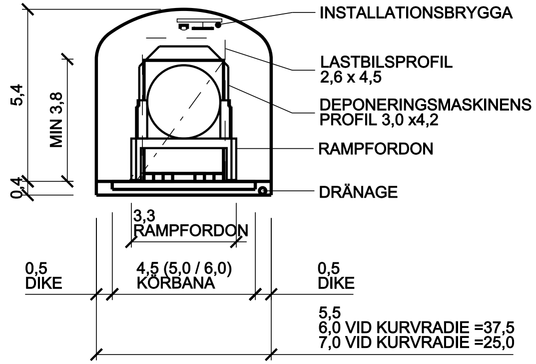


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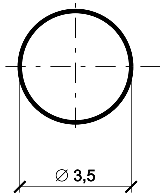


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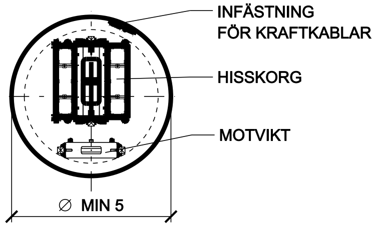


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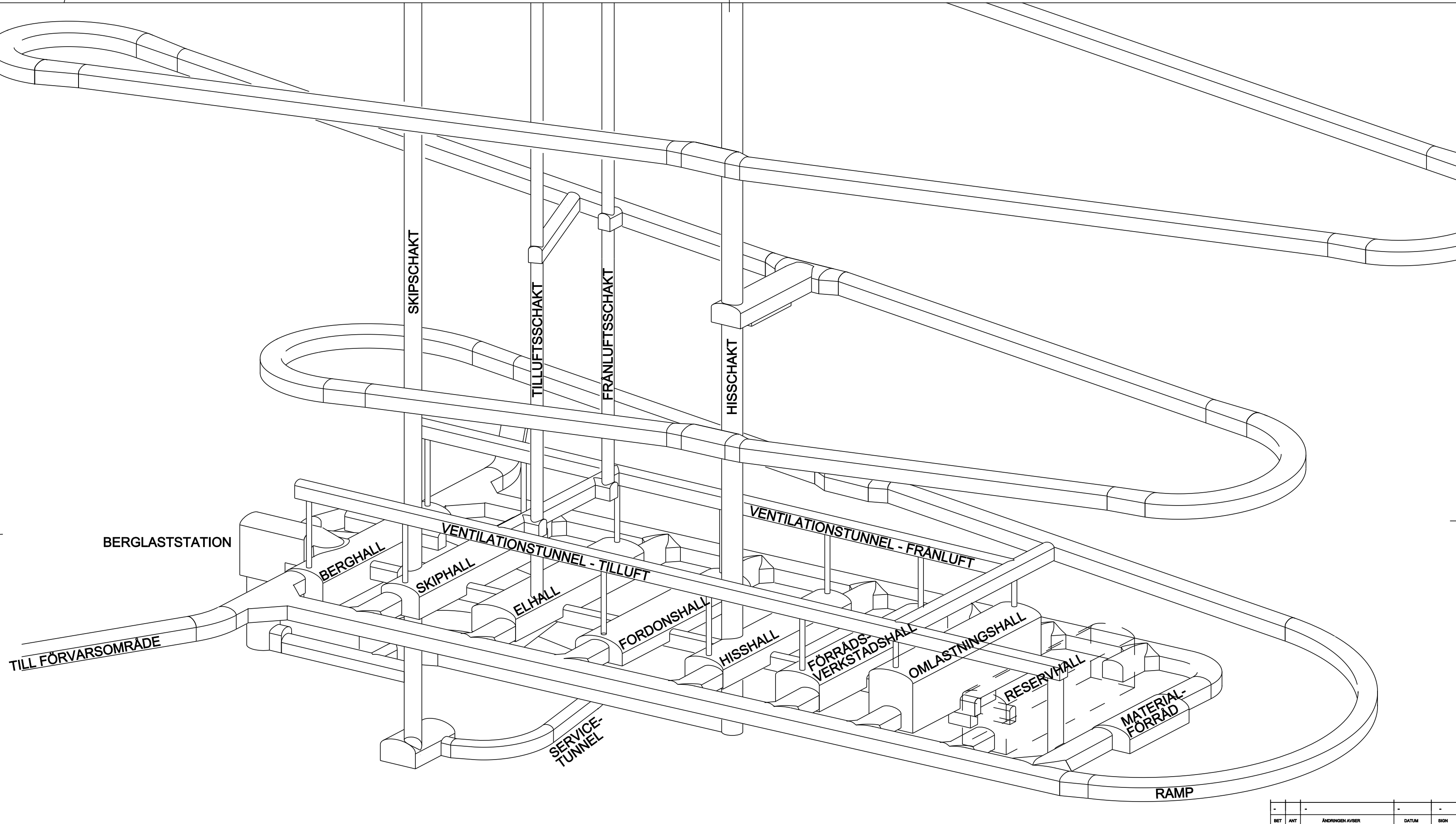


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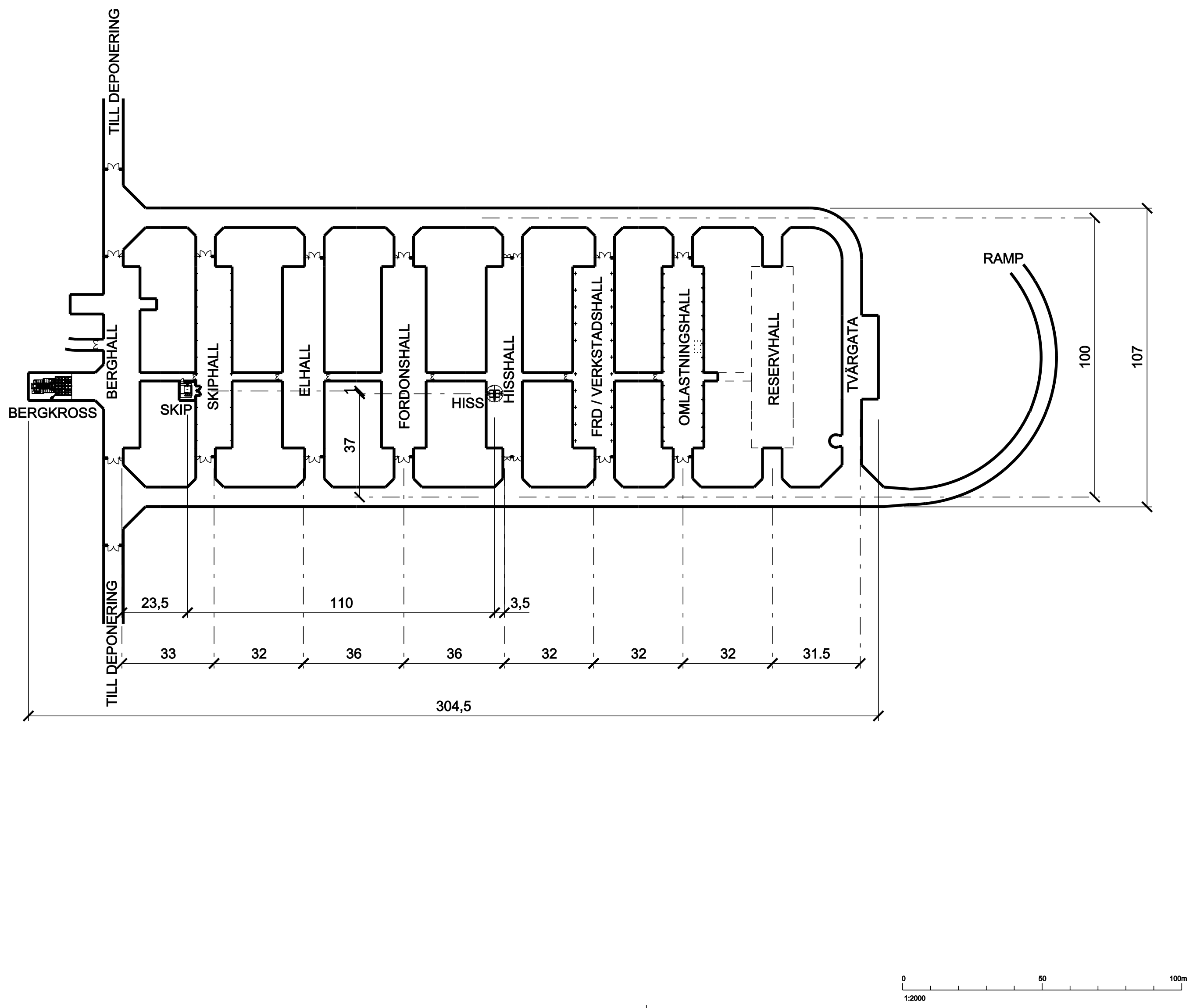


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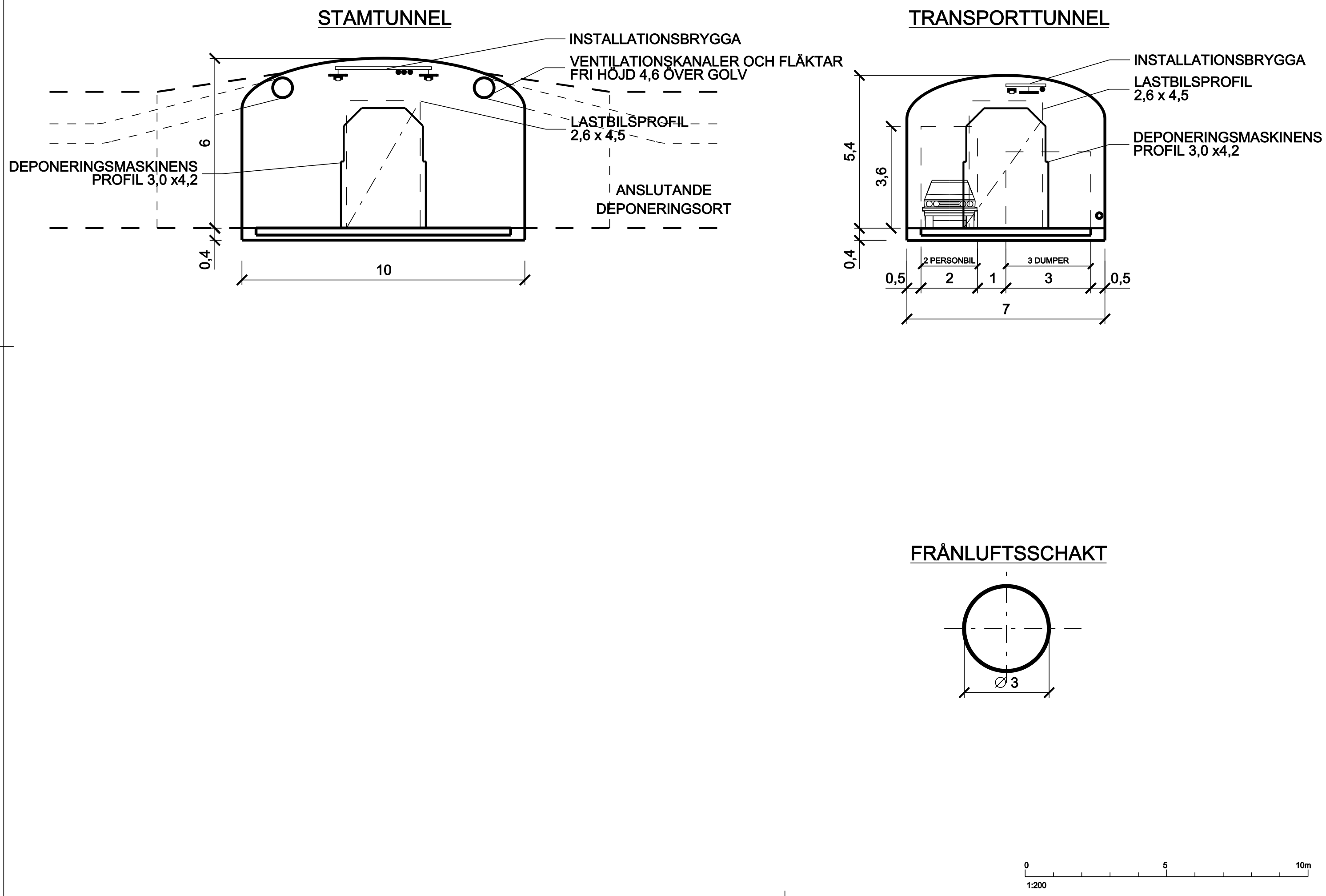


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|---|---|---|---|---|
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SKB Slutförvar för använt kärnbränsle

SKB - PROJEKTERING SLUTFÖRVAR

| UPPDRAG NR | RITAD | GRANSKAD |
|---|---|---|
| - | SWECOOS | - |
| DATUM | GODKÄND | |
| 2009-06-25 | - | |

REFERENSUTFORMNING
FÖRVARSOMRÅDE
PROFILER AV TUNNLAR OCH SCHAKT

| SKALA | NUMMER | BET |
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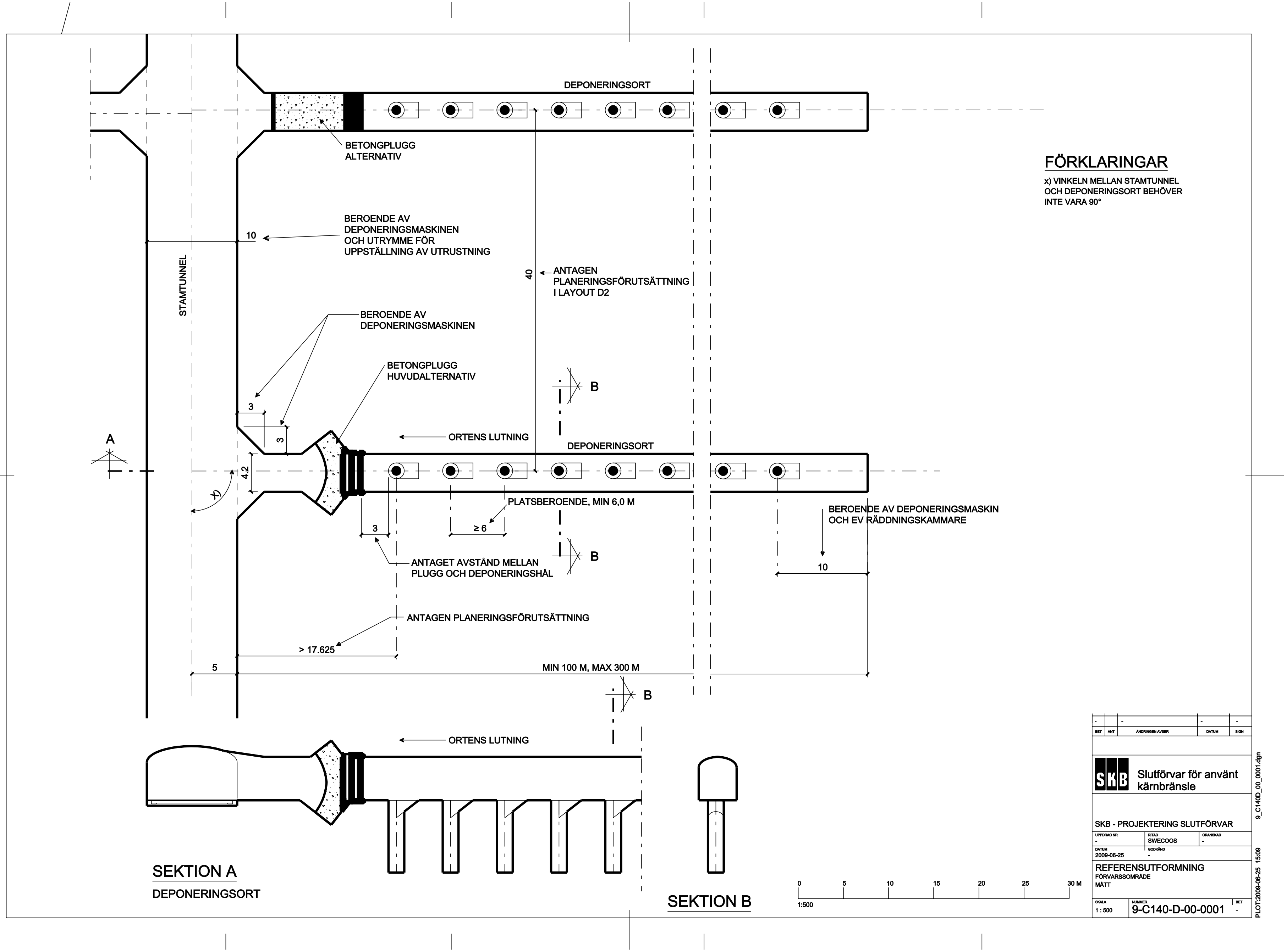

DEPONERINGSORT
BETONGPLUGG
ALTERNATIV
FÖRKLARINGAR
x) VINKELN MELLAN STAMTUNNEL
OCH DEPONERINGSORT BEHÖVER
INTE VARA 90°
10
BEROENDE AV
DEPONERINGSMASKINEN
OCH UTRYMME FÖR
UPPSTÄLLNING AV UTRUSTNING
STAMTUNNEL
40
ANTAGEN
PLANERINGSFÖRUTSÄTTNING
I LAYOUT D2
BEROENDE AV
DEPONERINGSMASKINEN
BETONGPLUGG
HUVUDALTERNATIV
B
3
3
ORTENS LUTNING
A
DEPONERINGSORT
4.2
x)
PLATSBEROENDE, MIN 6,0 M
BEROENDE AV DEPONERINGSMASKIN
OCH EV RÄDDNINGSKAMMARE
3
≥ 6
B
ANTAGET AVSTÅND MELLAN
PLUGG OCH DEPONERINGSHÅL
10
ANTAGEN PLANERINGSFÖRUTSÄTTNING
> 17.625
5
MIN 100 M, MAX 300 M
B
ORTENS LUTNING
SEKTION A
DEPONERINGSORT
SEKTION B
0
5
10
15
20
25
30 M
1:500
BET
ANT
ÄNDRINGEN AVSER
DATUM
SIGN
SKB
Slutförvar för använt
kärnbränsle
SKB - PROJEKTERING SLUTFÖRVAR
UPPDRAG NR
RITAD
SWECOOS
GRANSKAD
DATUM
2009-06-25
GODKÄND
REFERENSUTFORMNING
FÖRVARSSOMRÅDE
MÅTT
SKALA
1 : 500
NUMMER
9-C140-D-00-0001
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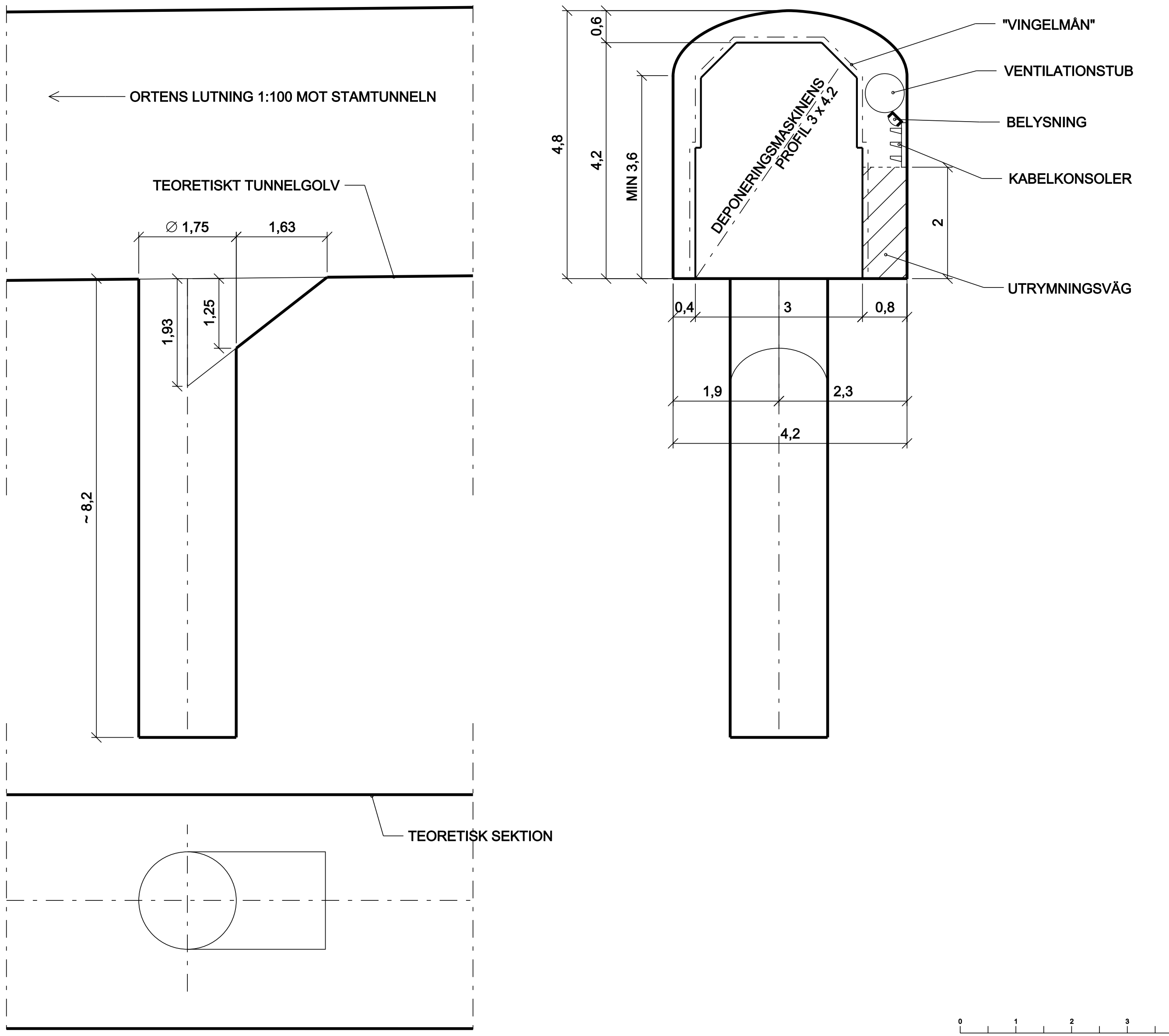
A7




## FÖRESKRIFTER

DEPONERINGSORTENS EXCENTRICITET FÅR EJ SPEGELVÄNDAS SETT FRÅN STAMTUNNELN

0 1 2 3 4 5 M
1:100

| BET | ANT | ÄNDRINGEN AVSER | DATUM | SIGN |
|---|---|---|---|---|
| - | - | - | - | - |

SKB Slutförvar för använt kärnbränsle

SKB - PROJEKTERING SLUTFÖRVAR

| UPPDRAG NR | RITAD | GRANSKAD |
|---|---|---|
| - | SWECOOS | - |
| DATUM | GODKÄND | |
| 2009-06-25 | - | |

REFERENSUTFORMNING
FÖRVARSOMRÅDE
DEPONERINGSORTER OCH DEPONERINGSHÅL
MÅTT

| SKALA | NUMMER | BET |
|---|---|---|
| 1 : 100 | 9-C140-D-00-0021 | - |

9_C140D_00_0021.dgn
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REF: 9_140CAB00_0001.dgn

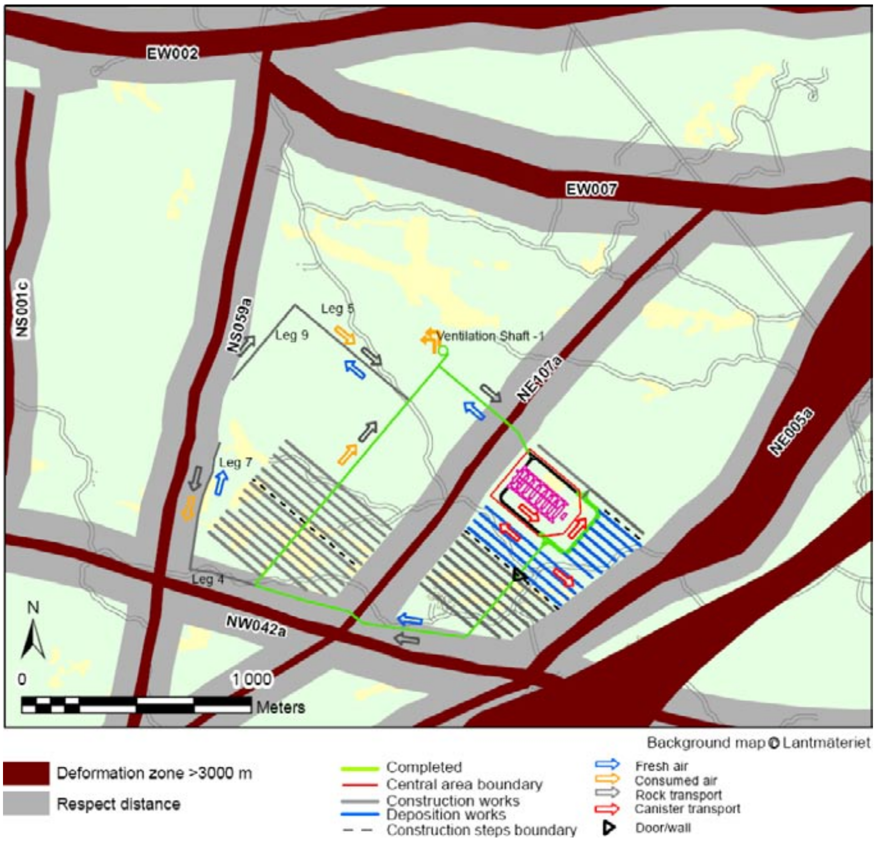
A8

# Appendix B

## A 40-year Construction & Deposition Development Plan

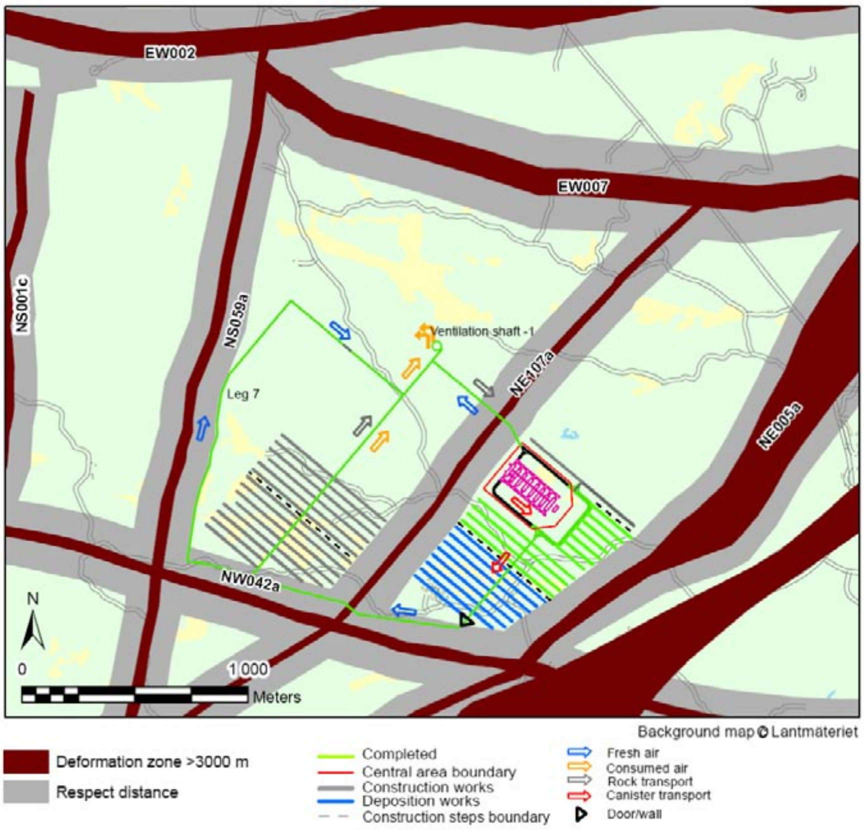
### List of Figures

**B2**

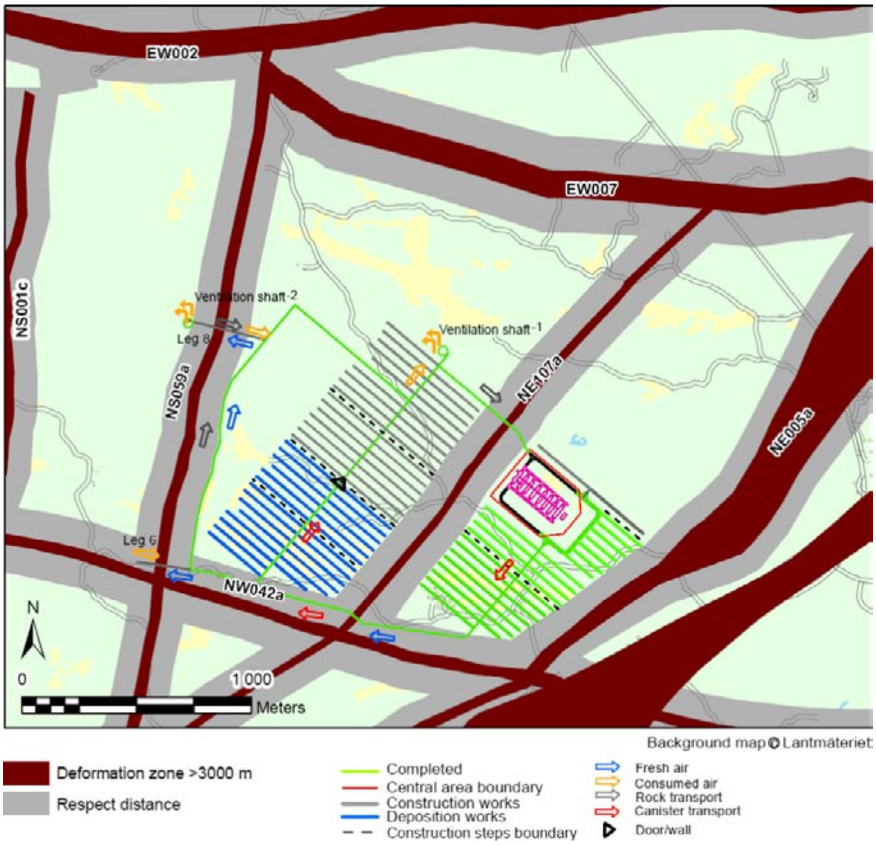



*Construction plan, year 0–9.*

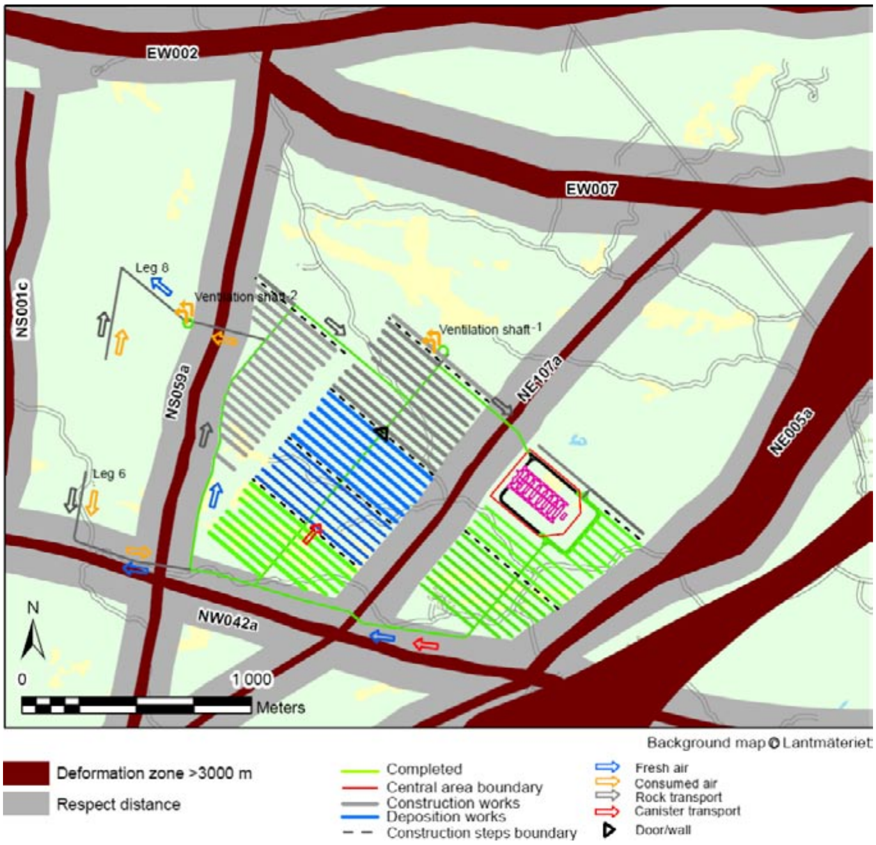
*The first canister is deposited year 6.*




*Construction plan, year 9–11.*

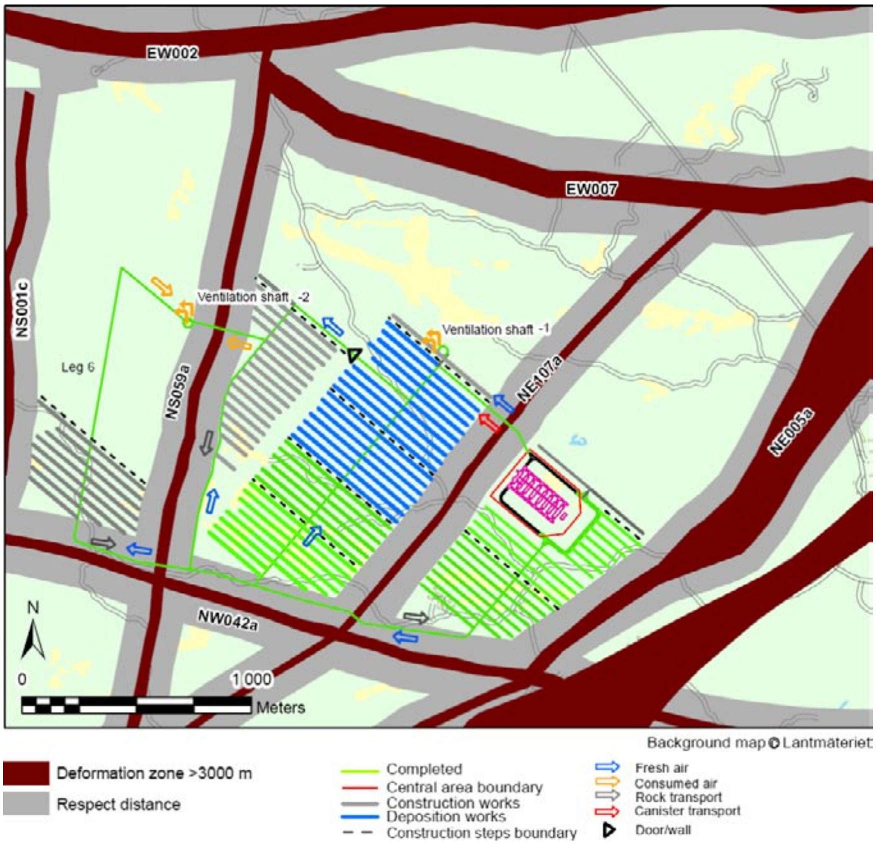



*Construction plan, year 11–13.*

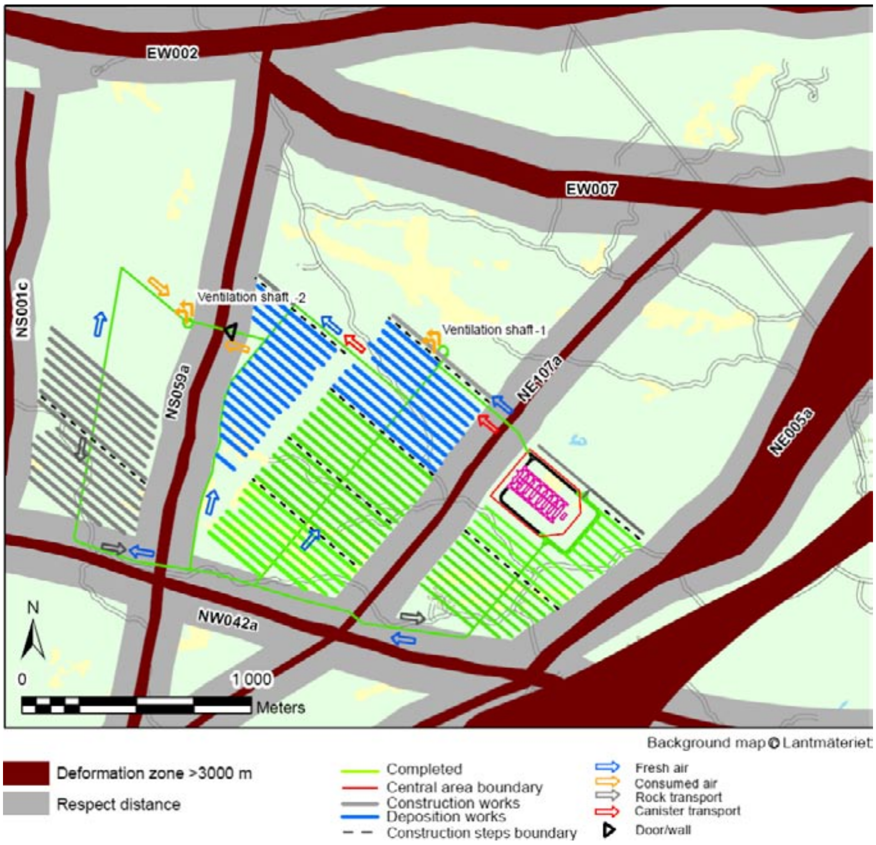



*Construction plan, year 13–15.*

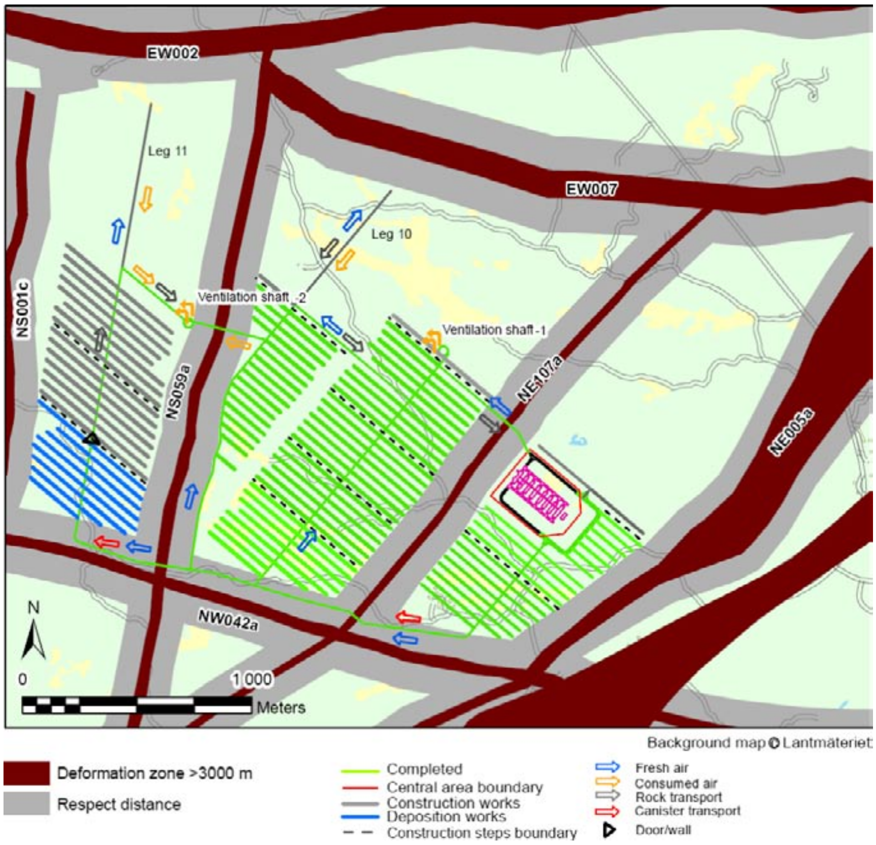
## B4



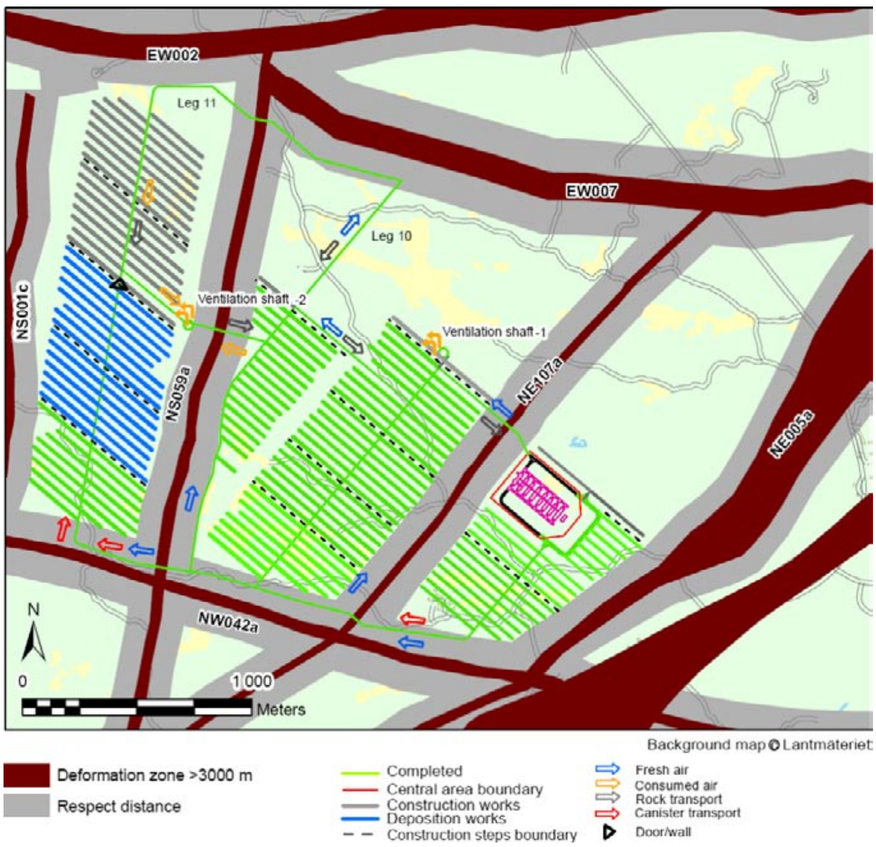

*Construction plan, year 15–17.*




*Construction plan, year 17–19*

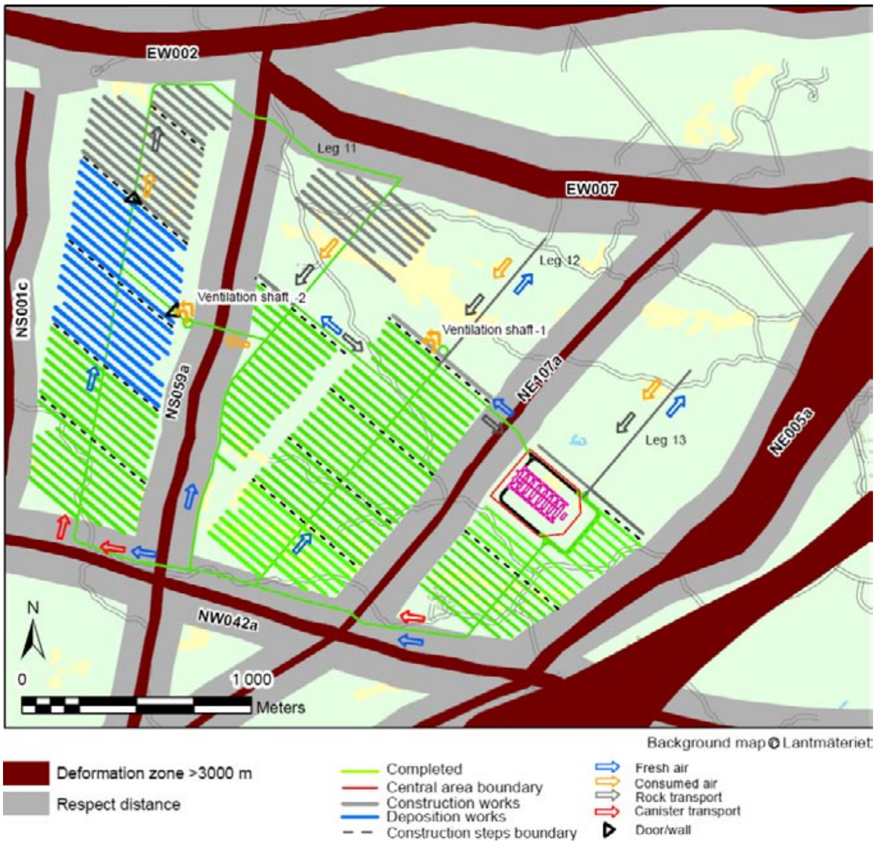



*Construction plan, year 19–21*

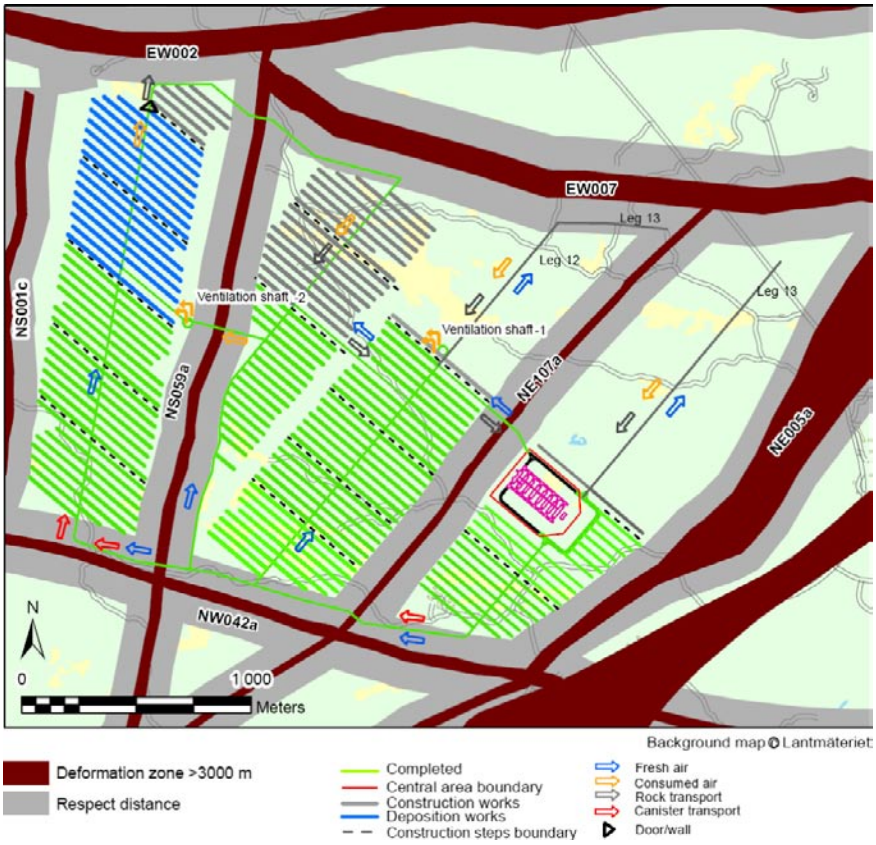



*Construction plan, year 21–23*

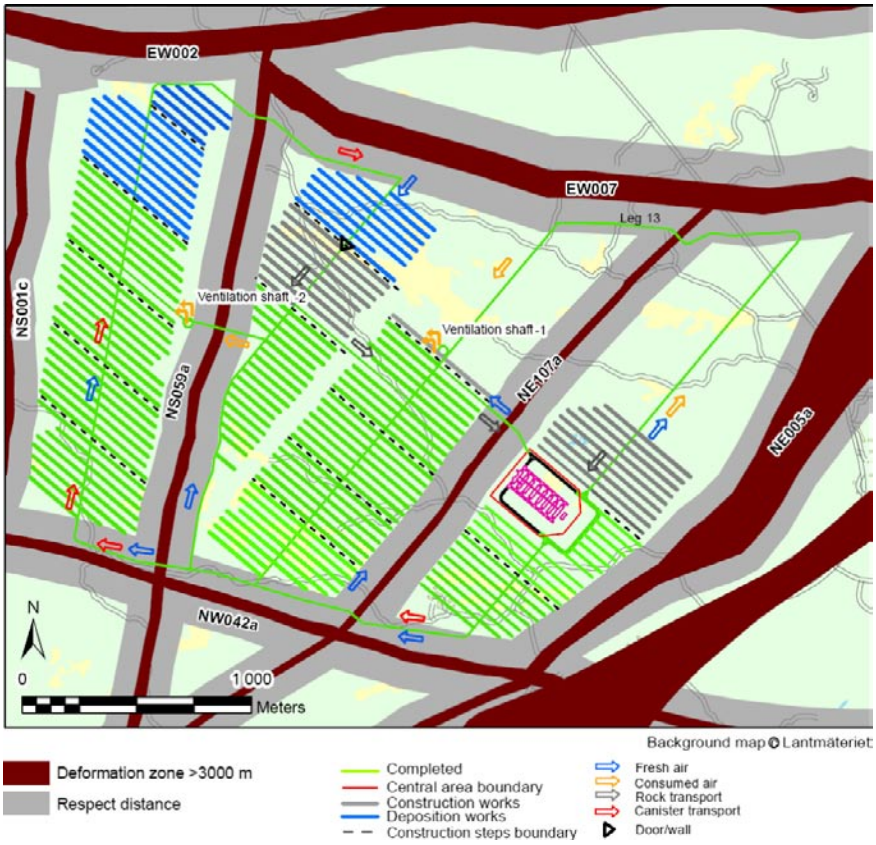
**B6**



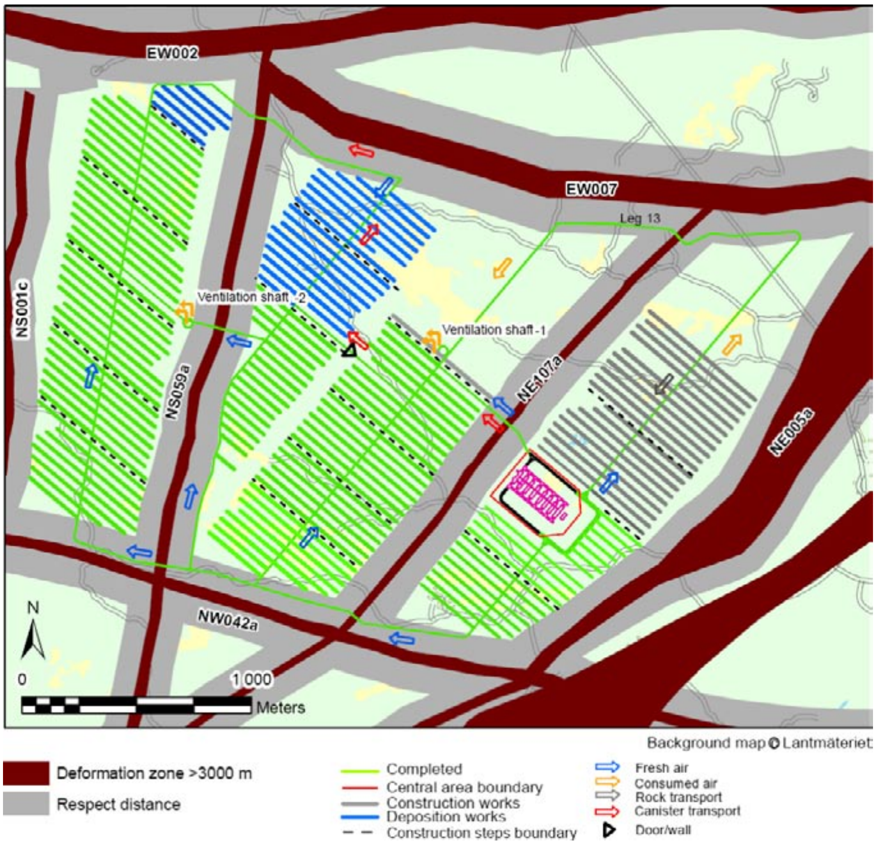

*Construction plan, year 23–25*




*Construction plan, year 25–27*

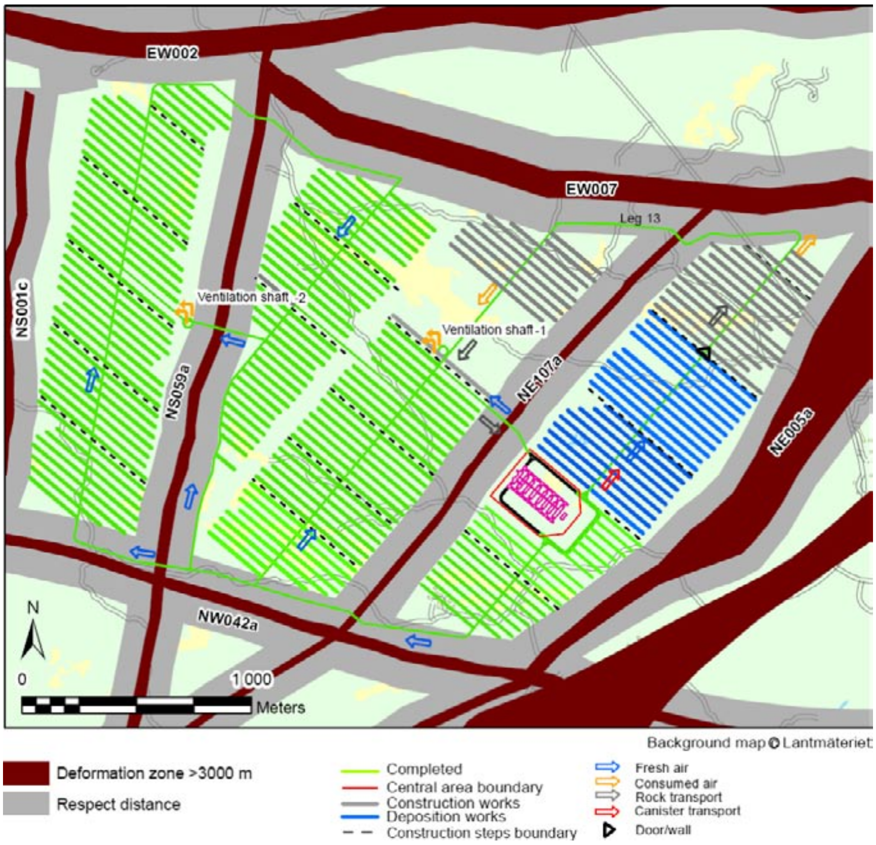



*Construction plan, year 27–29*

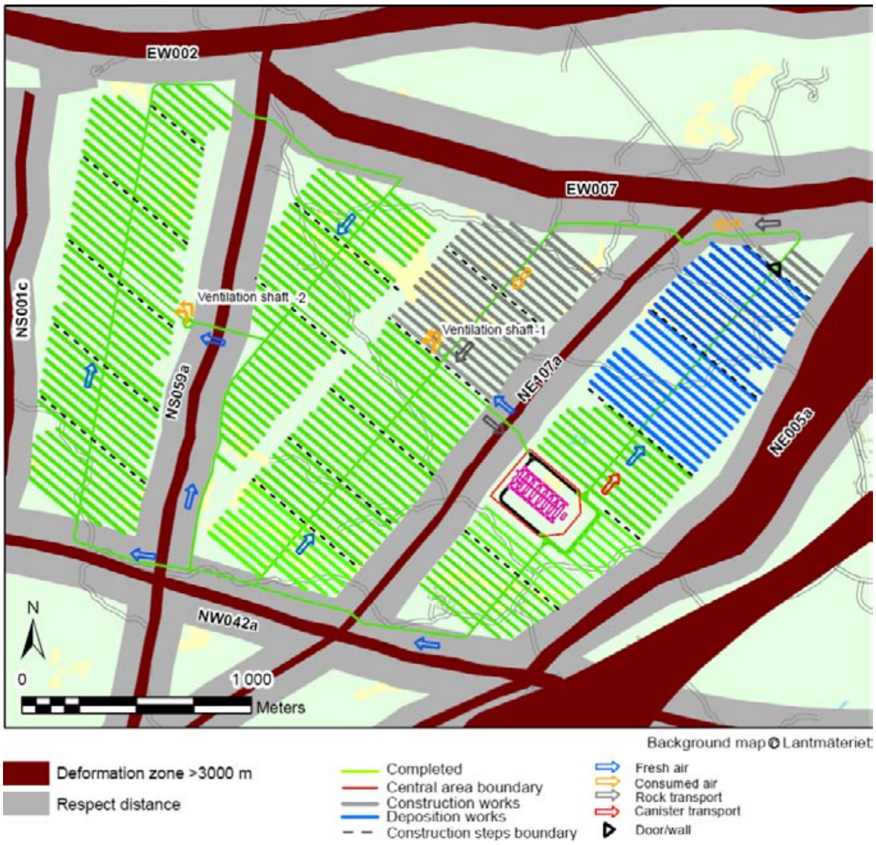



*Construction plan, year 29–31*

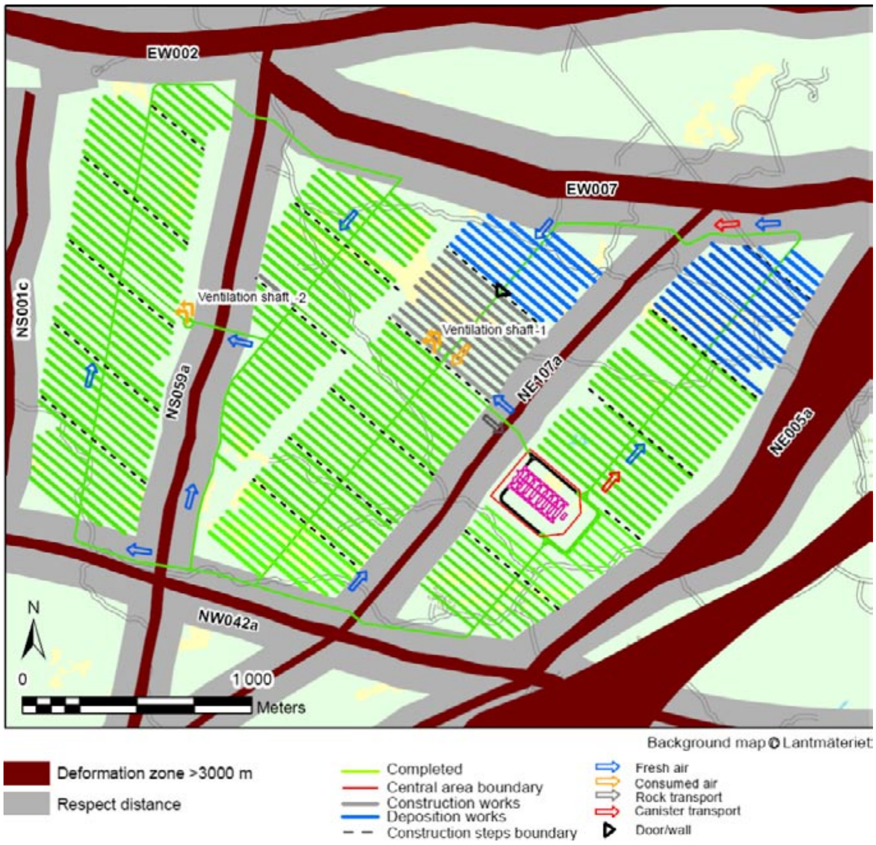
## B8



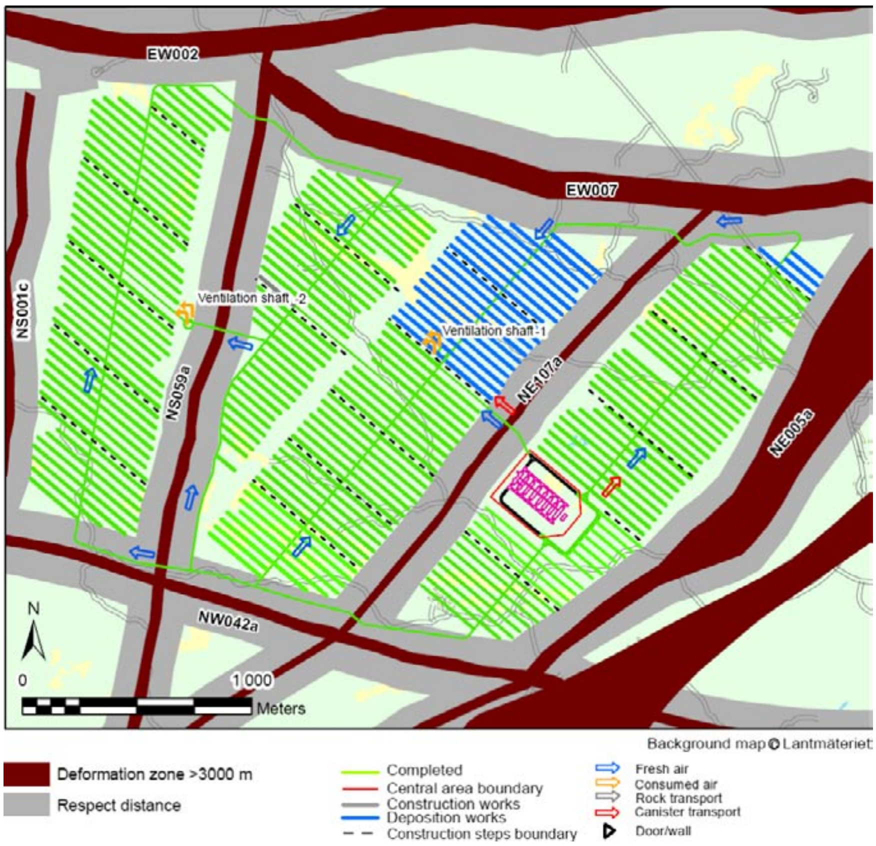

*Construction plan, year 31–33*




*Construction plan, year 33–35*

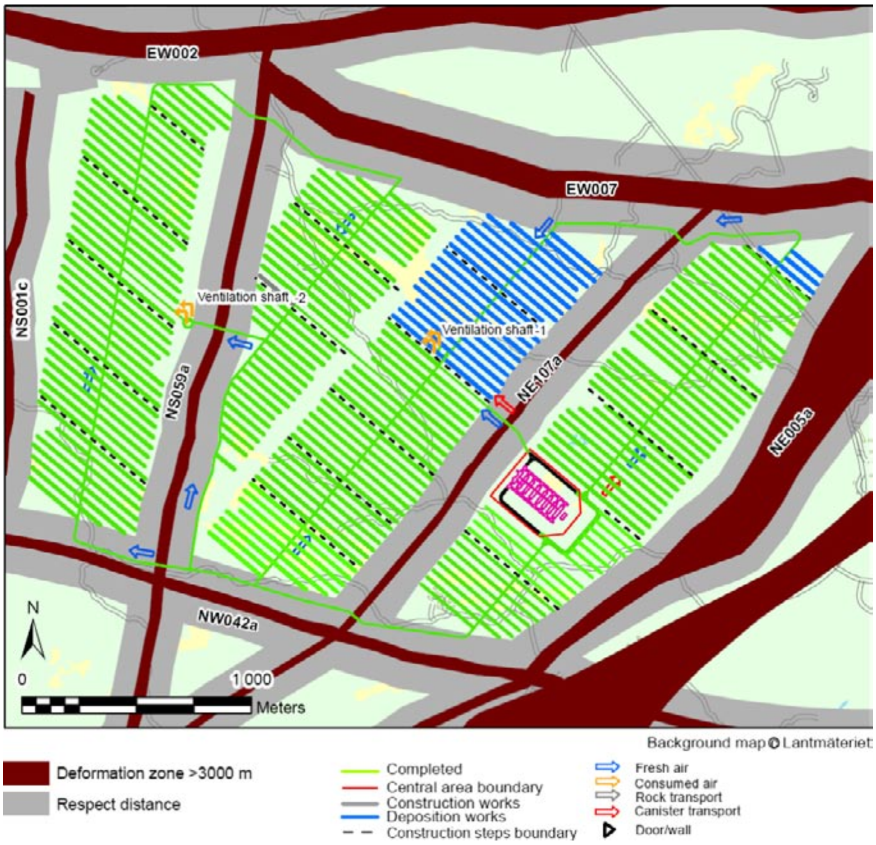



*Construction plan, year 35–37*




*Construction plan, year 37–39*

## B10




*Construction plan, year 40*