R-08-116

Underground design Forsmark Layout D2

Svensk Kärnbränslehantering AB

July 2009

Svensk Kärnbränslehantering AB
Swedish Nuclear Fuel
and Waste Management Co

Box 250, SE-101 24 Stockholm
Phone +46 8 459 84 00

ISSN 1402-3091
SKB Rapport R-08-116

Underground design Forsmark
Layout D2

Svensk Kärnbränslehantering AB

July 2009

Summary

The candidate area for site investigations at Forsmark is situated within the north-western part of an ancient and geologically-stable tectonic lens. The lens is approximately 25 km long and extends along the Uppland coast from northwest of the Forsmark nuclear power plant towards Öregrund in the southeast. The 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 2007.

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 tectonic lens at a depth of approximately 470 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 Forsmark 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 requires an area of 3.6 km^2, and the total rock volume to be excavated is $2.2 \cdot 10^6$ m^3 using a total tunnel length of approximately 72 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 tunnels placed in any of these zones. The layout has a gross capacity of 7,818 deposition-hole positions, which provides for a loss of deposition-hole positions of approximately 23% (1,818). The 1,818 extra deposition-hole positions are expected to be sufficient to accommodate all losses due to unacceptable water inflows and intersection of long fractures.

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 either structurally controlled wedge failure and/or 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. 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 Forsmark SFR Facility and other underground excavations at the Forsmark Nuclear Power Plant. The layout of the repository area has the deposition tunnels aligned $< 30^\circ$ 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 be relatively massive with few widely spaced water bearing fractures (0.005/m). Groundwater inflows are not expected to be a significant issue at repository level. Results from grouting analyses indicate that conventional grouting measures will generally be sufficient to meet the inflow criterion. However, in some situations the aperture of the fracture could be so low that reaching the required sealing efficiency may not be practical with cement-based grouts; other sealing technologies 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. 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 was that none of the consequences from these uncertainties would render the repository unsuitable for the purpose intended. However, 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.
- In situ stress magnitudes and orientations at repository level.
- Spatial dimensions of 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 där platsundersökningarna i Forsmark genomförts är beläget i den nordvästliga delen av en geologisk stabil tektonisk urbergslins. Linsen är cirka 25 km lång och sträcker sig längs Upplandskusten från ett område nordväst om Forsmarks kärnkraftverk i sydöstlig riktning till Öregrund. Platsundersökningen av kandidatområdet har utförts i etapperna, inledande (IPLU) och kompletta (KPLU) platsundersökningar. Undersökningarna påbörjades 2002 och avslutades under 2007.

Under och parallellt med platsundersökningarna genomfördes ett antal studier och tre projekteringssteg (D0, D1 och D2) för att säkerhetsställa, att tillräckligt utrymme fanns tillgängligt för en layout omfattande 6 000 kapselpositioner inom den tektoniska linsen och på ett djup av cirka 470 m. Riktlinjer för layouten angavs i *Underground Design Premises/D2* (UDP/D2) och parametrar och restriktioner för designen av undermarksanläggningen redovisades i *Forsmark 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. Området som layouten omfattar är 3.6 km^2, och den totala uttagna bergvolymen uppgår till cirka $2{,}2 \cdot 10^6$ m^3. Den totala tunnellängden är cirka 72 km.

Layouten innefattar samtliga deterministiska deformationszoner och respektavstånd för deformationszoner längre än 3 000 m. Inga deponeringstunnlar är placerade i dessa zoner. Layouten har en bruttokapacitet av 7 818 kapselpositioner, vilket möjliggör ett kapselbortfall på cirka 23 % (1 818). Dessa extra 1 818 kapselpositioner förväntas vara tillräckliga för att ersätta bortfall på grund av oacceptabla vatteninflöden och kontakt med långa sprickor.

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 uppnås genom att tillämpa traditionell bergförstärkning och genom att orientera utrymmena i förhållande till största horisontella spänningen. Förstärkningsmängden bedöms vara låg som en följd av bergmassans förväntade mycket goda kvalitet. Denna slutsats stöds också av de erfarenheter, som finns dokumenterade från undermarksutrymmena för Forsmarks kärnkraftverk och SFR. 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 förväntas vara relativt massiv med få vattenförande sprickor med stort sprickavstånd (0.005/m). Grundvatteninflödet på förvarsdjup förväntas bli mycket litet. Resultaten från injekteringsanalyserna indikerar, att konventionella injekteringsmetoder i allmänhet kommer att vara tillräckliga för att möta inflödeskriterierna. Däremot kan tillämpning av annan injekteringsteknik behöva användas lokalt på förvarsnivån och då främst i sprickor med liten sprickvidd.

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. I likhet med alla förundersökningar finns osäkerheter i tolkningen av geologisk information från borrhål. Dessa osäkerheter har identifierats, och inverkan av dessa osäkerheter på den nuvarande designen har utvärderats genom tillämpning av riskanalysmetoder. Den genomförda riskbedömningen visar, att ingen av konsekvenserna av dessa osäkerheter skulle leda till att förvaret är olämpligt för dess avsedda syfte. Flera osäkerheter har däremot identifierats, som skulle erbjuda 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.
- In situ spänningsmagnituder och spänningsorientering på förvarsnivå.
- Rumslig fördelning av 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 objective 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 Forsmark were completed in 2007 (Figure 1-2). The investigations were carried out according to the guidelines provided in /SKB 2000a, SKB 2000b/ and the findings from these investigations were used to develop a site descriptive model (SDM) for the site. A SDM is an integrated model for geology, thermal properties, rock mechanics, hydrogeology, hydrogeochemistry, 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.

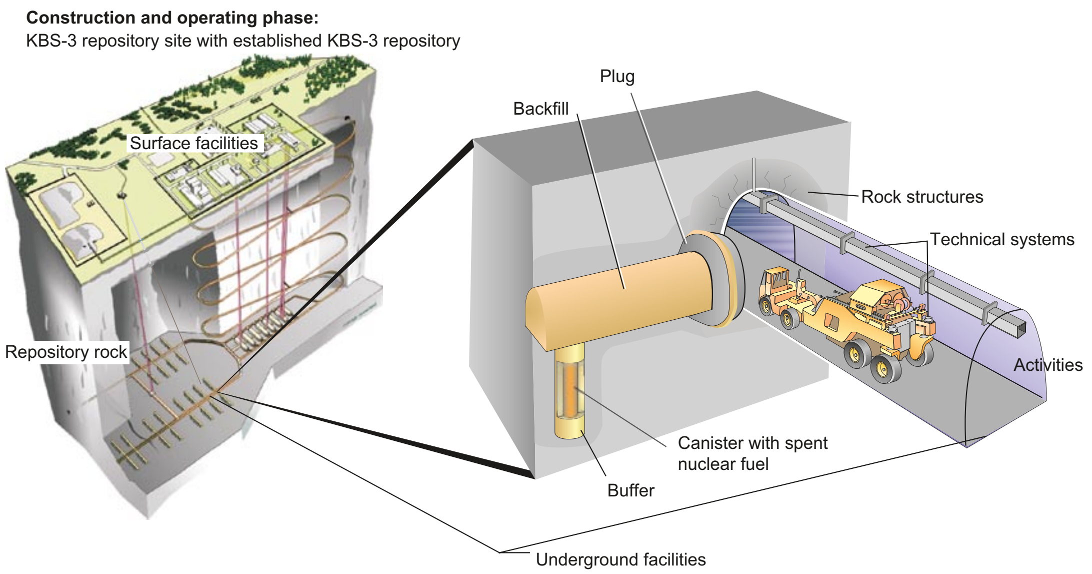


***Figure 1-1.** General three dimensional overview of three major underground functional areas of the Final Repository, (Access area, Central area and Deposition area). The location of the surface facilities is also shown.*

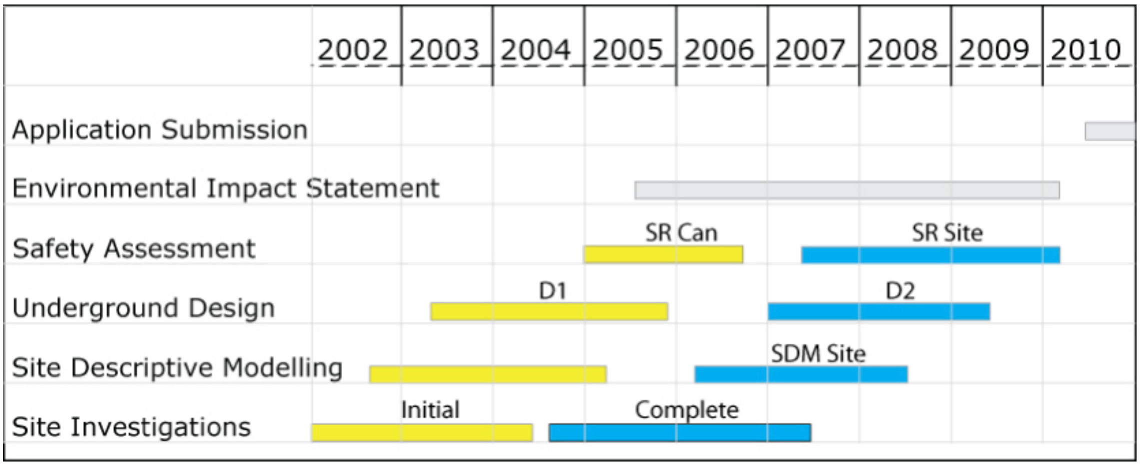


Figure 1-2. *Schedule for the design of the Final Repository Project up to the submission of the application.*

1.1 Site investigations

The candidate area for site investigations at Forsmark is situated within the northwestern part of an ancient and geologically-stable tectonic lens. The lens is approximately 25 km long and extends along the Uppland coast from northwest of the Forsmark nuclear power plant southeastwards to Öregrund (Figure 1-3).

The goal of the site investigation phase was to obtain sufficient information to enable application for permission to site and build a final repository for spent nuclear fuel /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.
- Allow comparisons with other investigated sites.
- Serve as a basis for adaptation of the deep 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). These commenced in 2002 and were completed in 2007 and are described below. The locations of the drill sites used for both the Initial and Complete site investigations and the boundary of the candidate area are given in Figure 1-4 and the general topography of the site can be seen in Figure 1-5.

Initial Site Investigations (ISI)

The initial site investigation stage (ISI) investigations at Forsmark focused on characterising conditions at depth within the tectonic lens with a given 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 given number of deep investigation boreholes on the site to determine whether the site is suitable for complete site investigations.

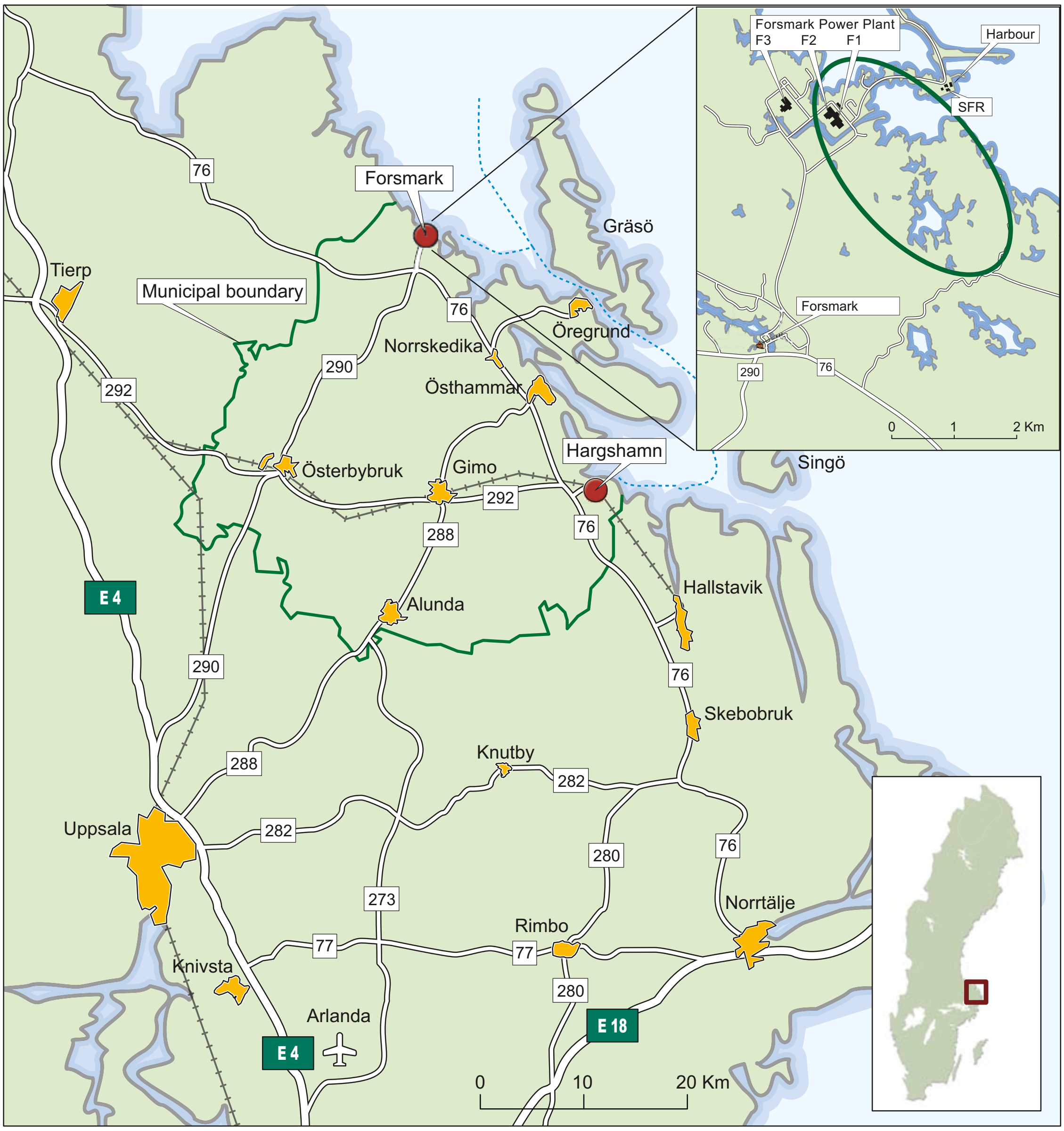


Figure 1-3. *Map of Sweden showing the location of Forsmark. The green ovaloid in the insert figure approximates the location of the tectonic lens and the target area during the site investigations.*

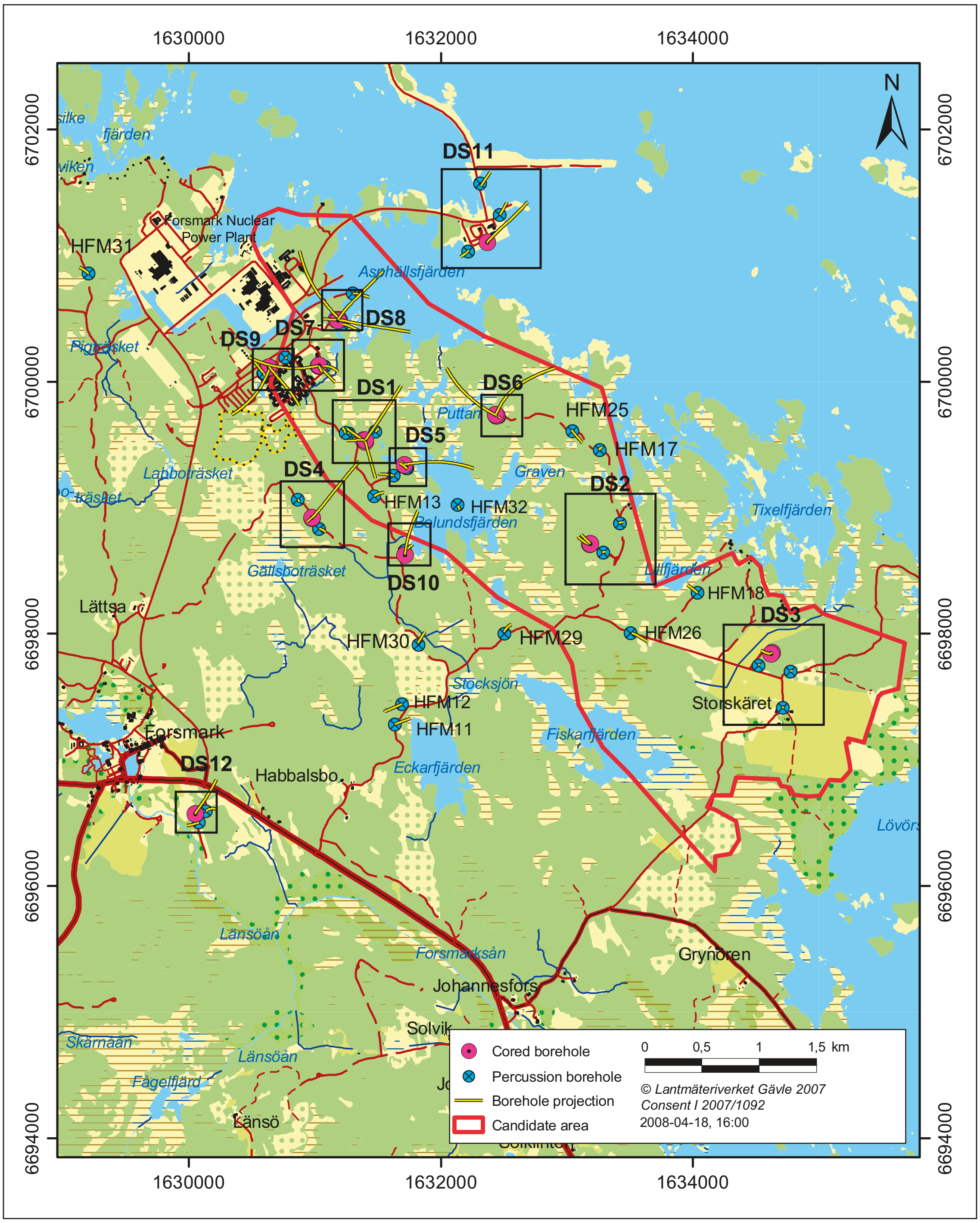


***Figure 1-4.** The locations of the drill sites (DS) used for the Forsmark site investigations and the location of the candidate area. /SKB 2008b, Figure 2-1/.*

Figure 1-5. *General view towards the south of the Forsmark site showing the outline (dotted line) of a portion of the investigated area. The surface infrastructure associated with Forsmark SFR Facility is in the foreground.*

A drilling and investigation programme comprising four deep-cored boreholes and a several additional percussion boreholes was carried out to establish the general characteristics of the tectonic lens 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 Forsmark site was favourable, and complete investigations were commenced.

Complete Site Investigations

The Complete Site Investigations (CSI) commenced in 2005 and was completed in 2007. During this 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 were completed in 2008.

The findings from the CSI are compiled in the site descriptive model and given in /SKB 2008b, SDM-Site/. 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 out in parallel to the site investigation programme. The reporting of the results from those activities and the process used to achieve them are described below.

1.2.2 Design steps

The repository design has been an iterative and stepwise process during the Site Investigation phase. 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 /Brantberger 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 produce 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 areal 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 2009a/ and are summarised in UDP/D2. They build on feedback from the safety assessment described in SR-Can /SKB 2006a/, a preparatory stage to the SR-Site safety assessment, based on the preliminary site descriptions /SKB 2005, SKB 2006bd/ and associated layouts. This feedback was considered in /SKB 2007/. The feedback from /SKB 2006a/ and the results from the site investigations /SKB 2005, SKB 2006d/ were used to develop general guidelines and site-specific constraints for the repository. These guidelines were documented in the Forsmark 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 Forsmark site. The results from those design studies are presented in the following reports:

- Layout studies /Hansson et al. 2008/.
- Rock mechanics and rock support /Eriksson et al. 2008/.
- Ground behaviour and grouting measures /Brantberger and Janson 2008/.

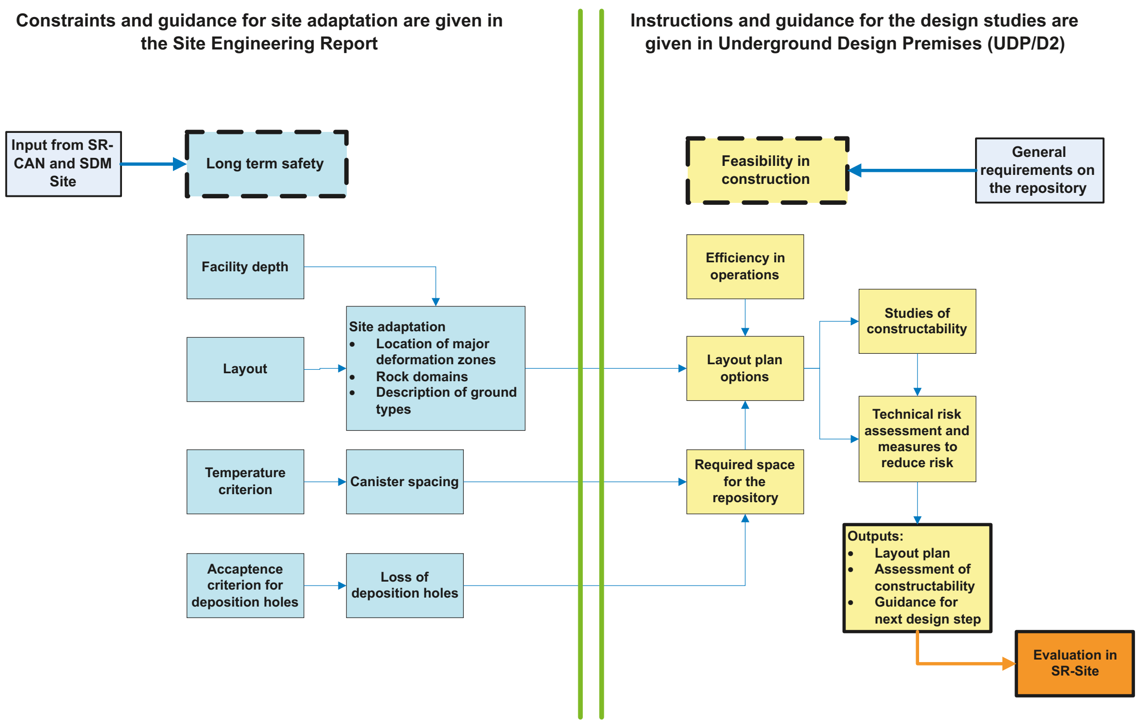


Figure 1-6. *Overview of the constraints and main deliverables from the SER (blue boxes) into design activities in accordance to UDP/D2 (yellow boxes).*

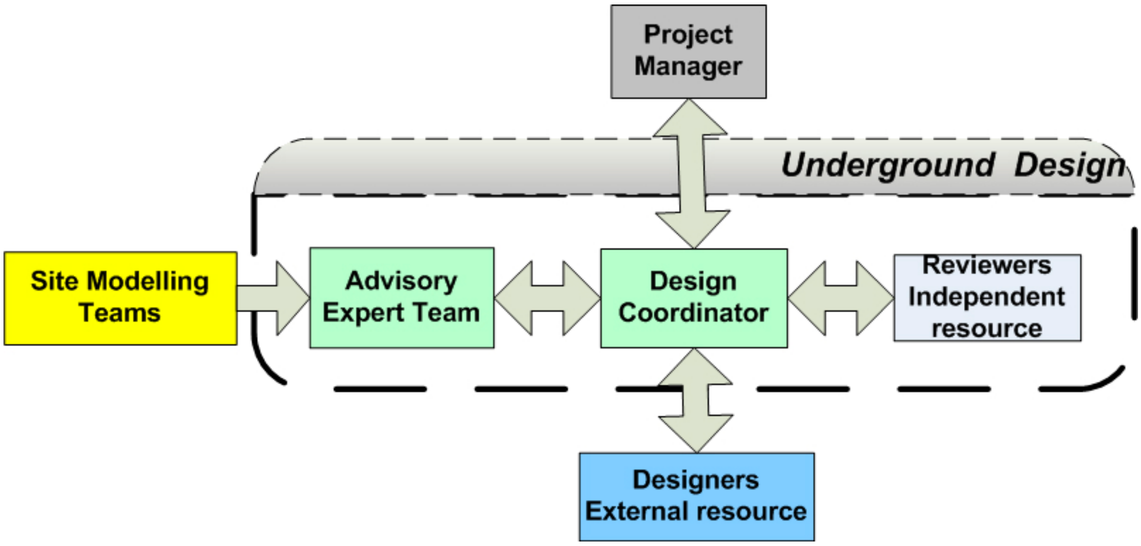


Figure 1-7. Overall Organisation of the Rock Engineering Design and its interfaces with respect to division of responsibilities and information /SKB 2007/. Compare Figure 2-1 by the colour codes for the different deliverables.

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 (SER) 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 /Hansson et al. 2008/.

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. 2008/.

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 2008/.

Chapter 8 assesses uncertainty and risk in Design D2. In this Chapter the key uncertainties identified from the findings of the site descriptive modelling (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 1-7).

Chapter 9 concludes the findings of underground design Forsmark, layout D2.

Appendix A provides typical drawings for the underground openings associated with repository.

Appendix B shows a proposed sequence for a step-wise construction and deposition for a timeline up to 50 years.

Appendix C contains an assessment of the potential loss of deposition-hole positions due to spalling.

2 Guidelines for the design D2 studies

An overview of the documents that were used in the underground 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/.

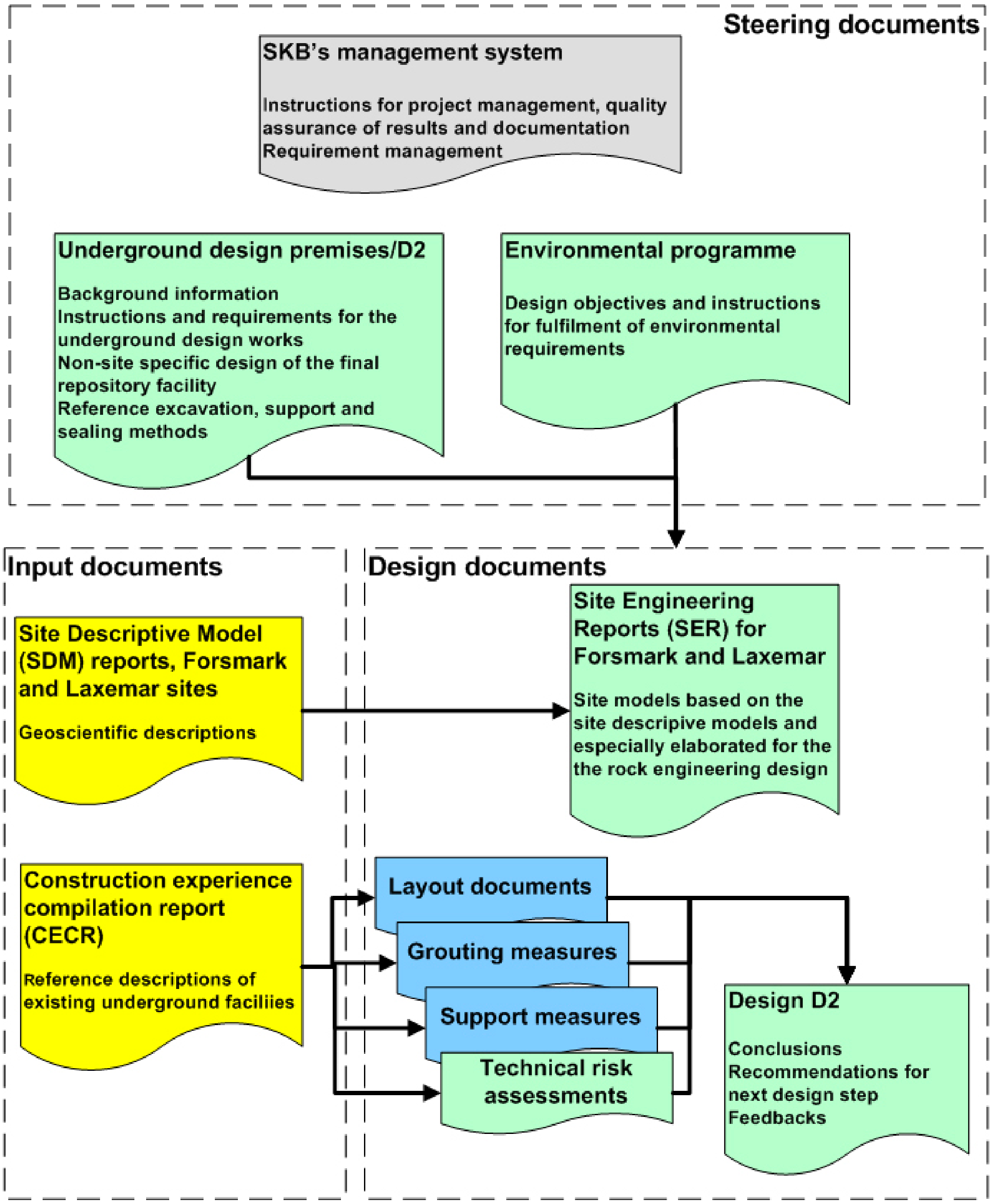


Figure 2-1. Overview of the documents that were used in the underground design in design step D2 /SKB 2007/. Compare colour codes in Figure 1-7 for responsibilities of the different documents in this Figure.

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. 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 2009a/.

The SER provides:

- 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 extracts 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 document 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 surface facility within the industrial area of the Forsmark nuclear power plant as specified in the municipality detail development plan. The accessible area for the surface facility is bounded to the northwest by the cooling water channel and to the northeast by the shoreline.

The repository layout was limited by the extension of the tectonic lens (cf. Section 3.2), and restrictions to the northwest and southeast given by the municipality detail development plan, see Figure 2-2. The area within the local model area, delimited by the rock domain boundaries (blue), and the administrative boundaries (yellow) is termed *the design target area* in this report.

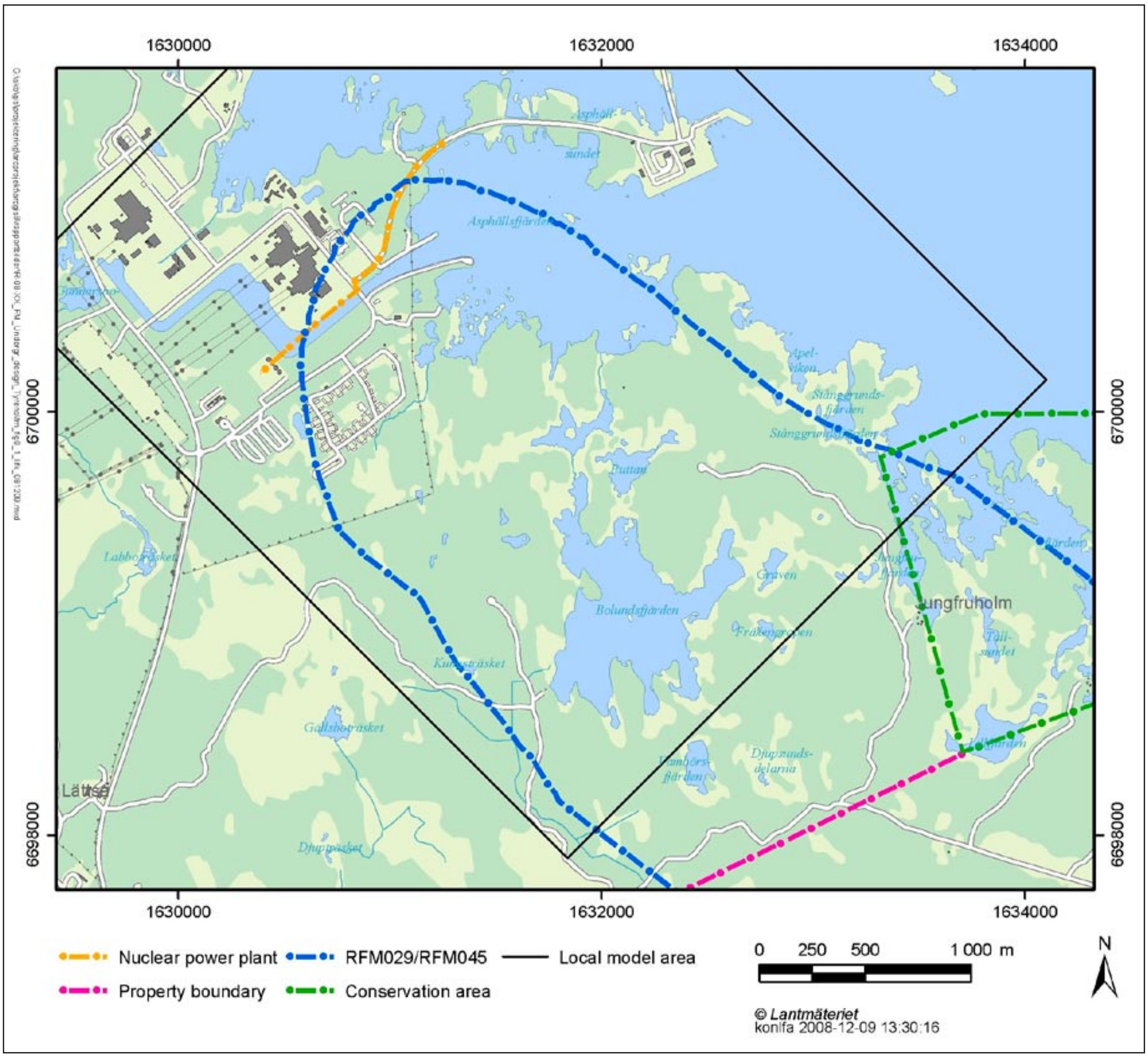


Figure 2-2. *Available underground area and administrative limits, projected on the surface.*

3 Site conditions considered in the design

3.1 Rock domains

The layout D2 shall be located in the dominant rock domains RFM029 and RFM045 within the tectonic lens /SKB 2008b/, see Figure 3-1. These two rock domains show similar rock composition, but they differ in the degree of early stage alteration referred to as albitisation.

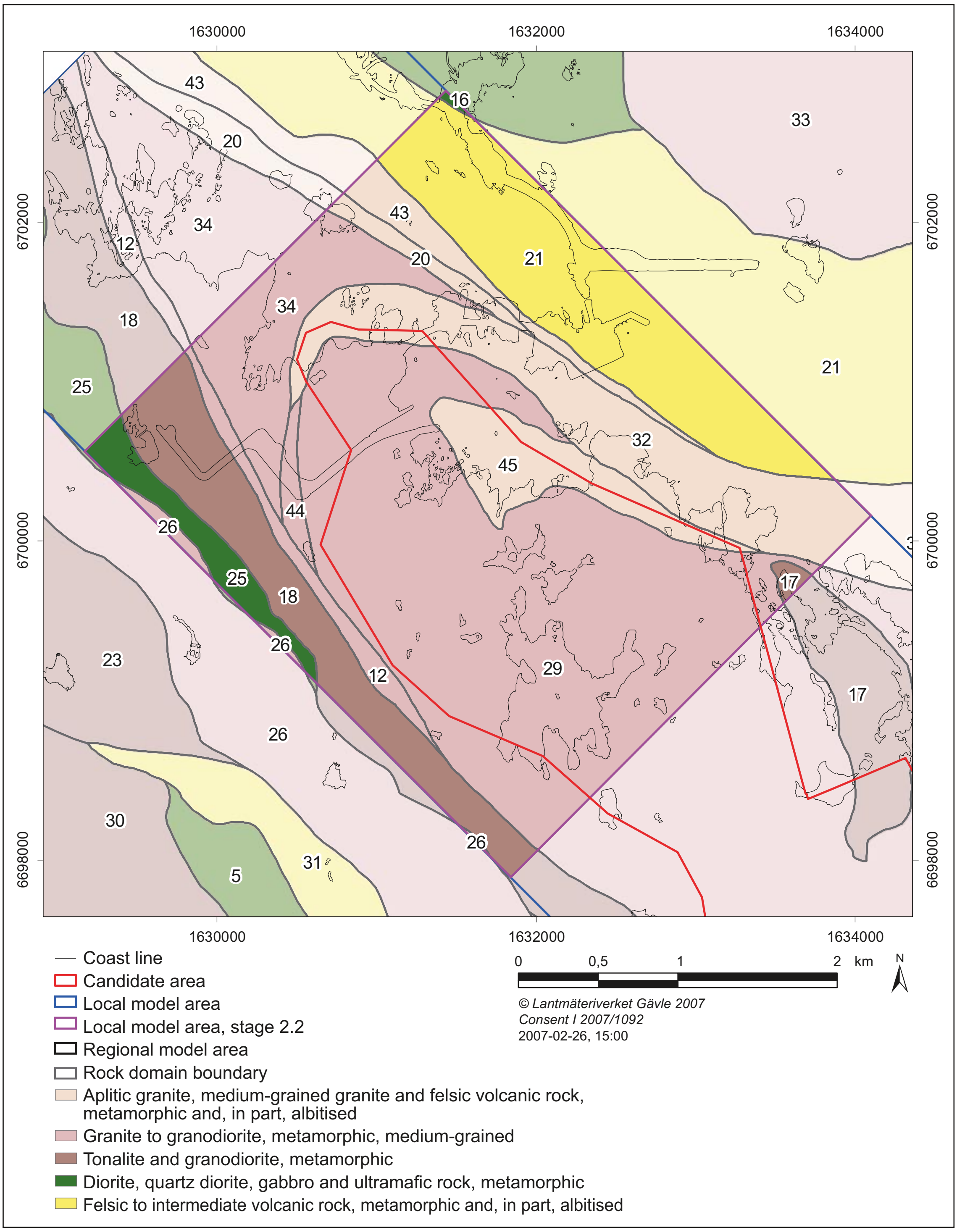


Figure 3-1. Plan view of the rock domains (Figure 5-24 in /SKB 2008b/).

Rock domain RFM029 is volumetrically the most significant domain. The dominant rock type in domain RFM029 is medium-grained metagranite to granodiorite, which comprises c. 74% of that domain. Subordinate rock types are pegmatite and pegmatitic granite (c. 13%), fine- to medium-grained metagranitoid (c. 5%), and amphibolite and other minor mafic to intermediate rocks (c. 5%). The subordinate rocks are forming isolated minor bodies or lenses and dyke-like sheets.

Rock domain RFM045 forms a subordinate part inside the target volume and is located north-east of rock domain RFM029. The domain has a conspicuous occurrence of albitized and metamorphosed granitic rocks, and is a generally finer grain size than rock domain RFM029. The dominant rock types in this domain are aplitic metagranite and medium-grained metagranite, which constitute approximately c. 49% and c. 18%, respectively, of the rock domain volume. Both these rock types are commonly affected by Na-K alteration (albitisation). It is also indicated from modal analyses that the quartz content is markedly increased and the K-feldspar content decreased, compared with unaltered rocks. Subordinate rock types in rock domain RFM045 are essentially the same as in rock domain RFM029 and include pegmatite and pegmatitic granite (14%), medium-grained metagranitoid (9%), amphibolite and other minor mafic to intermediate rocks (7%).

3.2 Fracture domains

Smaller zones and fractures not covered by the deterministic deformation zone model are handled in a statistical way through discrete fracture network (DFN) models. The DFN models are based on fracture observations in the boreholes, mapped fractures at outcrops, size modelling and from interpretation of lineaments. The DFN model captures both open and sealed fractures and hence this approach overestimates the open fracture frequency.

The modelling assumptions are given in /SKB 2008b/. Based on a systematic assessment of the variation in the frequency of fractures with depth along each borehole, the bedrock between deterministically modelled deformation zones has been divided into fracture domains. Thus, fracture domains and deterministically modelled deformation zones are mutually exclusive volumes, whereas rock domains contain both fracture domains and deterministically modelled deformation zones /SKB 2008b/.

There are four fracture domains in the design target volume /SKB 2008b/ (see Figure 3-2):

1. Fracture domain FFM01 is situated within rock domain RFM029 inside the target volume, below the surface stress-released fractured rock referred to as fracture domain FFM02.
   Steeply dipping fractures that strike ENE to NNE and NNW, as well as gently dipping to sub-horizontal fractures are characteristic of this sparsely fractured domain. The experience at the SFR Facility, while outside this domain, suggests sub-horizontal fractures may appear as localised occurrences of limited areal extent.
2. Fracture domain FFM02. High frequency of gently dipping to sub-horizontal fractures and vertical to steeply dipping fractures that strike ENE or NNW are most conspicuous in this domain and occur to approximately 150 m depth. This fracture domain contains the open and hydraulically connected fractures and stress-relief fractures. The vertical extension of FFM02 appears to increase towards SE and has an observed maximum depth of 150 m in the vicinity of the gently dipping deformation zone A2.
3. Fracture domain FFM03. FFM03 is situated within rock domains RFM029 and RFM017, southeast of and outside the target volume. In particular, it is inferred to be present above zone A2 in borehole KFM02A and along the whole length of the boreholes KFM03A and KFM03B to the southeast of the local model volume. The rock domains in this volume are characterised by a high frequency of gently dipping fracture zones containing both open and sealed fractures. High frequency of gently dipping minor fracture zones that is open and shows hydraulic connections over a large area.
4. Fracture domain FFM06. FFM06 is situated within rock domain RFM045, inside the target volume. It resembles fracture domain FFM01 in the sense that it lays beneath both deformation zone A2 and fracture domain FFM02 (Figure 3-2). It is distinguished from domain FFM01 simply on the basis of the widespread occurrence of fine-grained, altered (albitised) granitic rock, with slightly higher contents of quartz compared to unaltered granitic rock, i.e. on the basis of rock type.

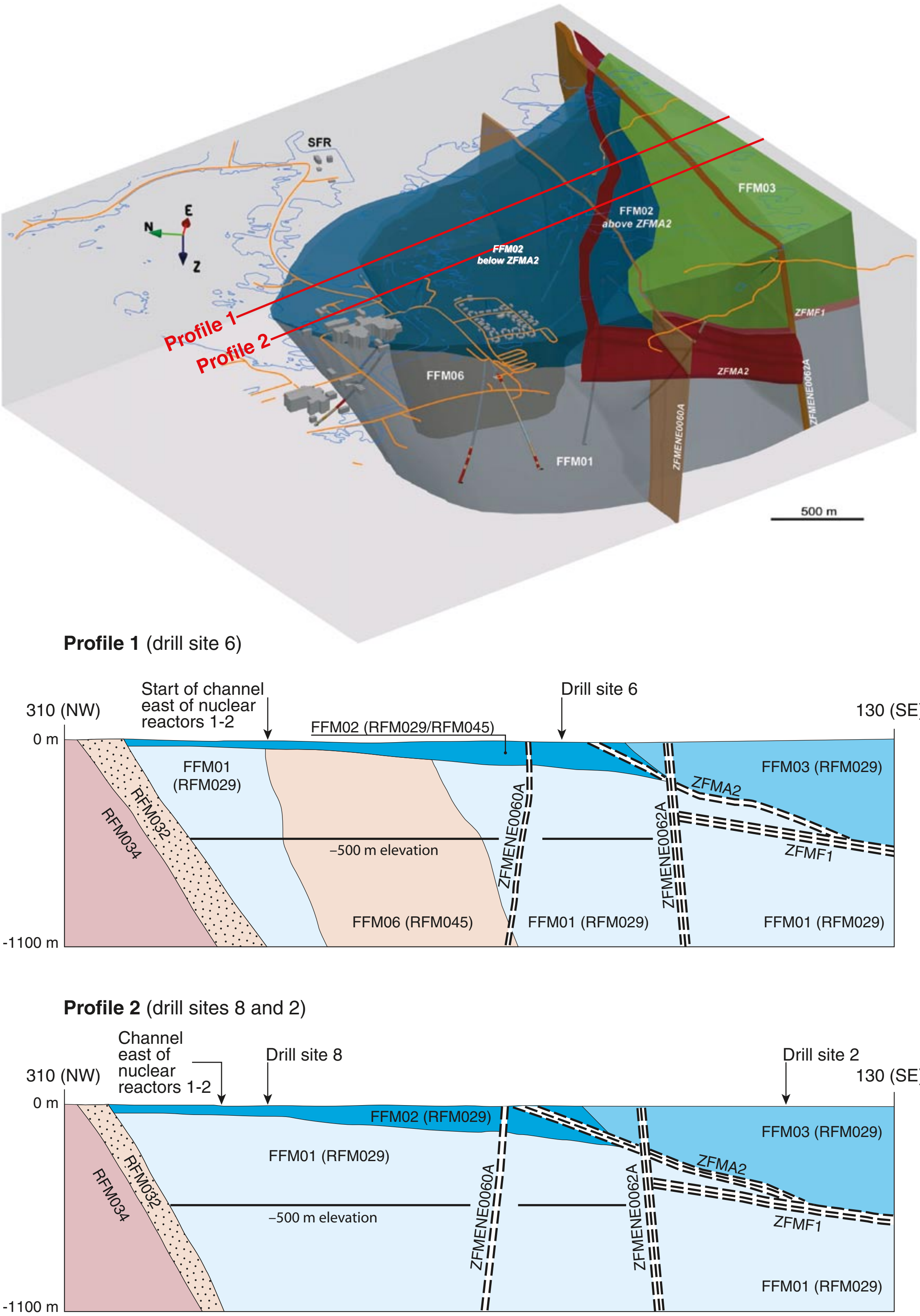


Figure 3-2. *Three-dimensional model for fracture domains FFM01, FFM02, FFM03 and FFM06 and the major deformation zones. The vertical sections also indicate the rock domains, Figure 11-14 in /SKB2008b/. The location of the line marking the –500 m elevation is shown for reference.*

3.3 Deformation zones and respect distances

According to the Design Premises – Long Term Safety /SKB 2009a/ deposition-hole positions are not allowed to be placed closer than 100 m to deformation zones with a trace length longer than 3 km. SDM Site identified three deformation zones that are potentially long enough to require a respect distance: ENE060A, ENE062A and NW0123 and the gently dipping zone A2, see Figure 3-2 and Figure 4-8.The locations of these deformation zones are based on drill hole intersections and surface trace lengths from magnetic surveys.

3.4 Rock mechanics

The laboratory strength and deformation properties of the intact rock types encountered in FFM01 and FFM06 at Forsmark are given in Table 3-1. As indicated in Table 3-1, these uniaxial compressive strength values are classed as either R5 (Very strong – mean UCS 226 MPa) or R6 (Extremely Strong – mean UCS373 MPa) using the ISRM Classification. The crack initiation stress from Table 3-1 was used in the spalling assessment for FFM01 and FFM06.

The rock mass at Forsmark was divided into four Ground Types /SKB 2008a/, Table 3-2. These ground types are a general description of the rock type and the discontinuities and used as input when establishing the ground control measures for the site. The anticipated distributions of these ground types are given as in Table 3-3.

Table 3-1. Laboratory strength and deformation properties for intact rock in fracture domains FFM01 and FFM06 (compiled from Table 7-3 in SDM Site, TR-08-05).

Parameter	FFM01	FFM06	
	101057 Mean/stdev	101057 Mean/stdev	101058 Mean/stdev
Young's modulus (GPa)	76/3	80/1	82/3
Poisson's ratio	0.23/0.04	0.29/0.02	0.27/0.03
Uniaxial Compressive strength (MPa)	226/29	373/20	310/58
Crack initiation stress (MPa)	116/23	196/20	169/29
Indirect tensile strength (MPa)	13.2/2	14.8/1	–

Note:
101057 – Granite to granodiorite, metamorphic, medium grained (albitized in FFM06);
101058 – Granite, metamorphic, aplitic (albitized).

Table 3-2. Summary of the four ground types for design stage D2.

Ground type	Description
GT1a	Massive to sparsely fractured rock mass in RFM029 (FFM01)
GT1b	Massive to sparsely fractured rock mass in RFM045 (FFM06)
GT2	Blocky rock mass. Moderately fractured rock contains fractures and hairline cracks, but the blocks between joints are intimately interlocked. (FFM02)
GT3	Deformation zone containing sealed fracture network, fault breccias and cataclasite
GT4	Regional deformation zone, containing fault breccias, crushed rock, sealed networks and cataclasite

Table 3-3. Ground Type distribution.

Description	GT1	GT2	GT3
Deformation zones			
ENE0060A	20	40	40
ENE0060A (Respect distance)	80	20	
Gently dipping	–	100	–
Steeply dipping (< 3 km)	20	40	40
Fracture domains (Deformation zones excluded)			
FFM02	85	15	
FFM01*	95	5	
FFM06*	95	5	

*This apply also for the boundaries of the Tectonic Lens at the repository level.

The estimated stress models for Forsmark are given in Table 3-4. Because of the elevated stress magnitudes at Forsmark and the uncertainty in these magnitudes at repository level, particularly the maximum horizontal stress, three stress models were evaluated in design D2. These stress models were used as input for the assessing the spalling potential around deposition holes and tunnels and caverns at repository level.

3.5 Hydraulic properties

The hydraulic properties in the design target volume are controlled by the fracture domains, and the steeply-dipping and gently-dipping deterministic deformation zones. Fracture domain FFM02, located near the surface down to a depth of about 100 to 150 m, has a relatively high frequency of transmissive fractures. Within FFM02, most of the flow occurs on sub-horizontal and/or gently dipping fractures /Follin et al. 2007/. The fracture frequency within FFM02, particularly the subhorizontal and gently dipping fractures, rapidly decreases with depth. Fracture domain FFM01 occurs below FFM02 and is characterized by sparsely fractured rock. FFM06 has the similar hydraulic/fracture characteristics as FFM01.

The SDM-Site Forsmark/SKB 2008b/ has shown that in FFM01 over 70% of the gently dipping water bearing fractures occur above Elevation –400 m and that over 90% of these features occur above Elevation –450 m. Between Elevations –200 and –400 the linear frequency of flowing fractures is about 0.05/m and the rock mass has an average hydraulic conductivity of approximately 5.2 x 10^{-10} m/s. Below elevation –400 m the observed frequency of flowing features is 0.005/m (i.e. observed frequency of flowing fractures is on average 1 every 200 m) and the rock mass has an average hydraulic conductivity in the order of 6.3 x 10^{-11} m/s, see Table 3-5. This suggests that the rock mass in FFM01 between the deformation zones at the deposition level approaches the permeability of intact rock although in-frequent occurrence of low transmissive joints cannot be fully excluded.

Table 3-4. Stress magnitudes and stress orientations for the three stress models used for Design Step D2 /SKB 2008a/.

Depth Range (m)	Maximum horizontal stress – σ_H (MPa)	Trend (°)	Minimum horizontal stress – σ_h (MPa)	Trend (°)	Vertical stress – σ_{vert} (MPa)
Most Likely					
0–150	**19+0.008z, ±20%**	145 ±20	11+0.006z, ±25%	055	0.0265z ±0.0005
150–400	9.1+0.074z, ±15%	145 ±15	6.8+0.034z, ±25%	055	0.0265z ±0.0005
400–600	29.5+0.023z, ±15%	145 ±15	9.2+0.028z, ±20%	055	0.0265z ±0.0005
400	38.7 ±5.8	145 ±15	20.4 ±4.0	055	10.6 ±0.2
500	41.0 ±6.2	145 ±15	23.2 ±4.6	055	13.2 ±0.3
Unlikely minimum scenario					
400	19.2 ±0.7	124 ±6	9.3 ±1.1	034	10.4
500	22.7 ±1.1	124 ±6	10.2 ±1.6	034	13.0
Unlikely maximum scenario					
450–475	56 ± 6	145 ±15	35 ±15	055	0.0265z ±0.0005

Table 3-5. Summary of flowing fracture transmissivity statistics for the different fracture domains detected by the so-called Posiva Flow Log (PFL). $P10_{,PFL}$ denotes the linear fracture frequency [m^{-1}], T denotes transmissivity [m^2/s] of individual fractures. (Compiled from Tables 10-17 to 10-24 of Follin et al. 2007).

Fracture Domain	Σ BH Length (m)	No. of flowing fractures PFL-f	Flowing fracture frequency ($P_{10,PFL}$ 1/m)	ΣT/L (m/s)	Max T of an individual fracture (m^2/s)	Min T of an individual fracture (m^2/s)	Mean log(T) of individual fractures	Std of log(T) of individual fractures
FFM01								
100–200	474.4	52	0.152	1.4E-07	4.68E-05	2.48E-10	–7.84	1.28
200–400	1,387.5	39	0.042	5.2E-10	1.83E-07	2.67E-10	–8.51	0.88
<-400	3,279.7	12*	0.005	6.3E-11	8.89E-08	6.16E-10	–8.19	0.66
FFM02	366.4	81	0.326	4.3E-08	7.31E-06	2.45E-10	–8.02	1.00
FFM03	1,334	49	0.072	1.6E-09	6.77E-07	1.09E-09		
FFM04	154.9	15	0.152	7.4E-09	2.80E-07	4.59E-09		
FFM05	122.0	2	0.027	3.2E-09	2.00E-07	2.00E-07		
FFM06	210.4	0	0.000	–	–	.		

While there is a significant decrease in the frequency and transmissivity of the gently dipping fractures with depth in FFM01, the same trends are less pronounced for the steeply dipping transmissive fractures and/or deformation zones. SDM Site suggests that steeply dipping deterministic deformation zones at the depth of the repository will only contain a few flowing fractures even though the zone may be several 10's of metres thick. At the depth of the repository the maximum transmissivity of theses steeply dipping deformation zones did not exceed 10^{-6} m^2/s and many of these zones did not have detectable flowing features.

The SER /SKB 2008a/ describes the strategy and methodology with regard to grouting. In brief, the need of grouting varies depending on the fracture domain concerned. At the repository depth (in fracture domain FFM01 and FFM06, respectively) it is anticipated that systematic grouting will not be needed as indicated in the hydrogeological modelling results given in Table 3-6. However considerable variation in hydrogeological conditions is expected in fracture domain FFM02 and the need for grouting may be substantial in some places in this domain.

3.6 Site adaptation

3.6.1 Repository depth

The general rock mass quality improves significantly in the depth interval 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. These factors are assessed and balanced in the Site Engineering Report /SKB 2008a/. 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 two factors are prominent at the Forsmark site because of the massive relatively low permeability rock in fracture domain FFM01 and the potential for deposition-hole spalling in this fracture domain.

Table 3-6. Relative percentages of the distribution of transmissivity values for 20-m-long horizontal sections at the repository depth. These transmissivity values were determined from hydrogeological semi-correlated discrete fracture network modelling described in SER /SKB 2008a/.

T (m^2/s)	$<4\cdot10^{-9}$	$4\cdot10^{-9}$–$3\cdot10^{-8}$	$3\cdot10^{-8}$–$2\cdot10^{-7}$	$2\cdot10^{-7}$–$5\cdot10^{-7}$	$5\cdot10^{-7}$–$1\cdot10^{-6}$	$>1\cdot10^{-6}$
%	97.4	2.1	0.42	0.04	0.02	0.02

In summary, a repository depth between 450 m and 500 m meets the requirements outlined in the SER /SKB 2008a/ and reduces the risk for encountering water bearing fractures without significantly increasing the risk of spalling.

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 Forsmark, the orientation of the maximum horizontal stress is 145±15 degree /SER, SKB 2008a/. Hence Design D2 optimised the layout with respect to 145±15 degree.

3.6.3 Deposition hole spacing

For design stage D2, the strategy for thermal dimensioning was based on the proposal by /Hökmark et al. 2008/. 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 in the bentonite 100°C.

The minimum centre-to-centre spacing for the deposition tunnels was set to 40 m and the minimum centre-to-centre spacing for the deposition holes was set to 6 m in RFM029 and 6.8 m in RFM045 /SKB 2008a/. This spacing is selected to ensure that the highest permissible temperature in the buffer does not exceed the 100°C criterion.

3.6.4 Loss of deposition-hole positions

There are two primary factors that contribute to the potential loss of deposition-hole positions /SER, SKB 2008a/:

1) Loss due to the intersection with minor deformation zones (length <3 km) or large fractures (radius >75 m). These structures have the potential for secondary shear movement more than 5 cm that may jeopardise the canister. According to the design premises long-term safety /SKB 2009a/, this means that the deposition holes meeting Extended Full Perimeter Intersection (EFPC) criterion /Munier 2006/, must not be used.
2) Loss to due to unacceptable water inflows.

At Forsmark because of the low open fracture frequency at depth it is likely that the 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 that considered up to 30% loss of positions, (Table 3-7).

Table 3-7. Gross number of positions required for various potential loss of deposition hole positions.

Loss (%)	Required gross number of deposition-hole positions	Net number of available deposition-hole positions
0	6,000	6,000
13	6,897	6,000
23	7,792	6,000
27	8,219	6,000
30	8,571	6,000

4 Repository facility and layout

As previously noted the Final Repository will consist of several functional areas: Surface facilities, Repository Access (ramps and shafts), Central Area and the Deposition Area /SKB 2009b/. This chapter provides an overview of each function area and the recommended layout for the Deposition Area.

4.1 Surface Facility

The Surface Facility (the industrial area) 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 in its turn is divided into an outer and inner operation area. The nuclear industrial activities are operated in the inner operation area; defined as an area with the more extensive physical protection, while other activities related to operation are carried out in the outer operation area.

The Surface Facility must be located:

- Within the industrial area as given by community development plans.
- Minimise any impact on Forsmarks Kraftgrupp AB (FKA), the company operating the neighbouring nuclear power plant.
- Meet all functional and environmental requirements.

Three alternative locations for the surface facilities were evaluated; see Figure 4-2. Because the Surface Facility is connected to the Central Area through vertical shafts and a ramp choosing a surface location must also consider the impact on the associated underground excavations.

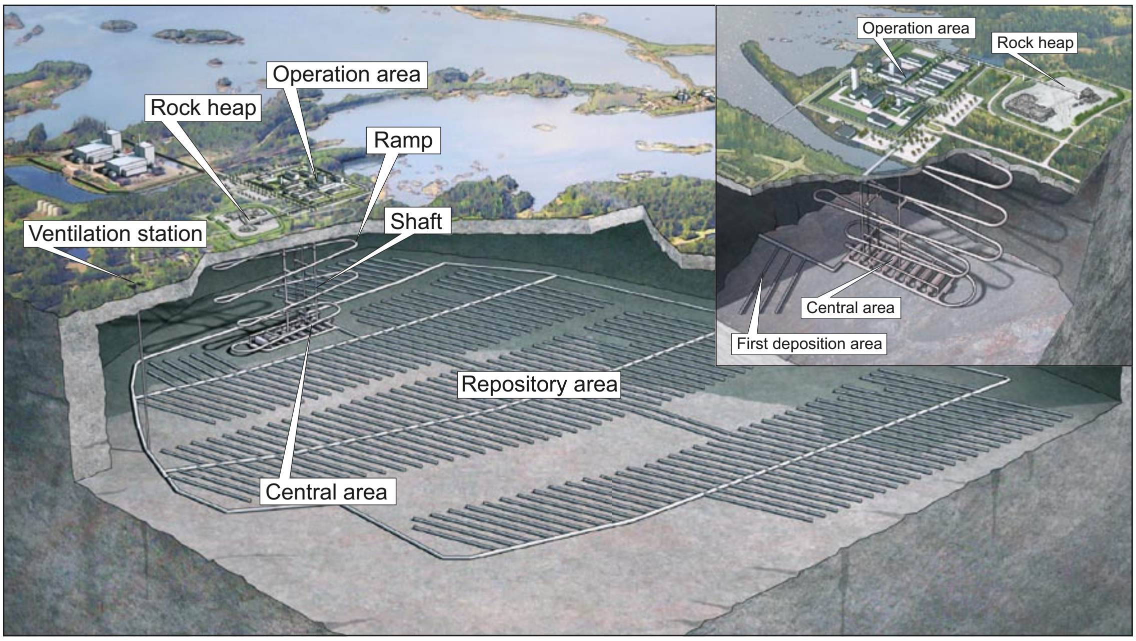
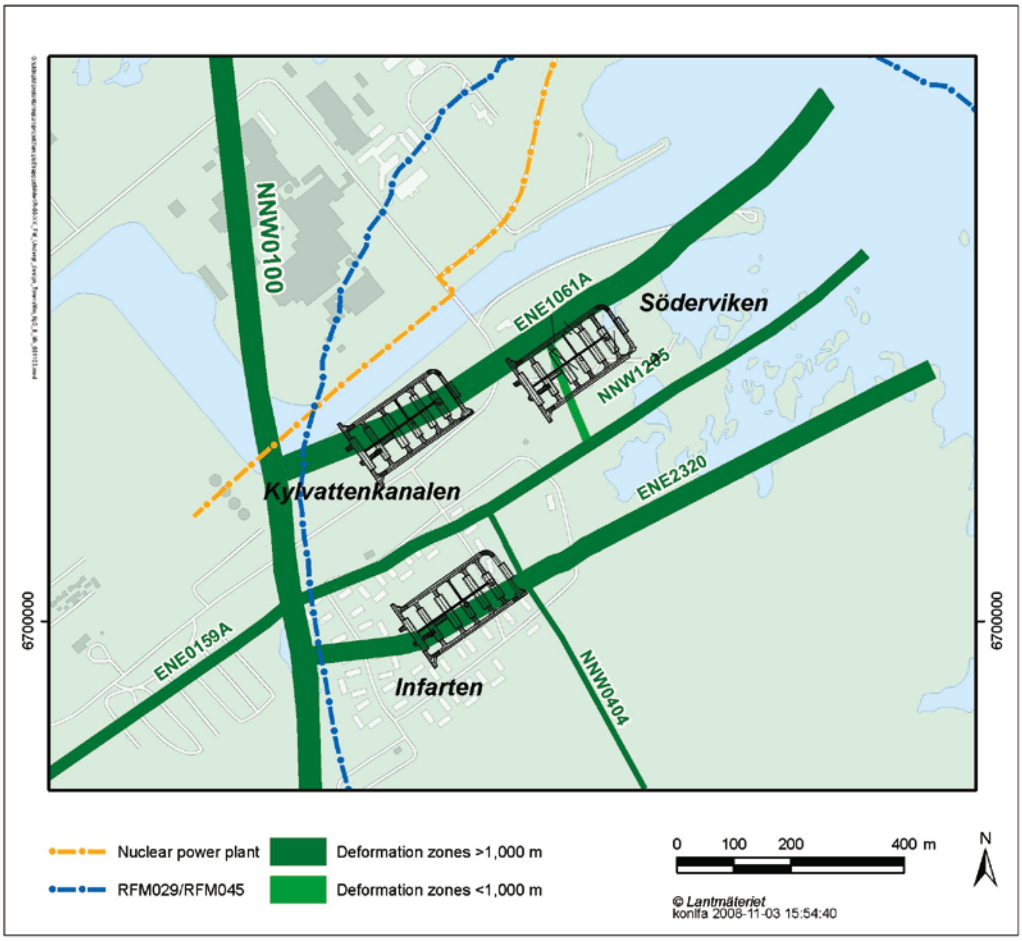
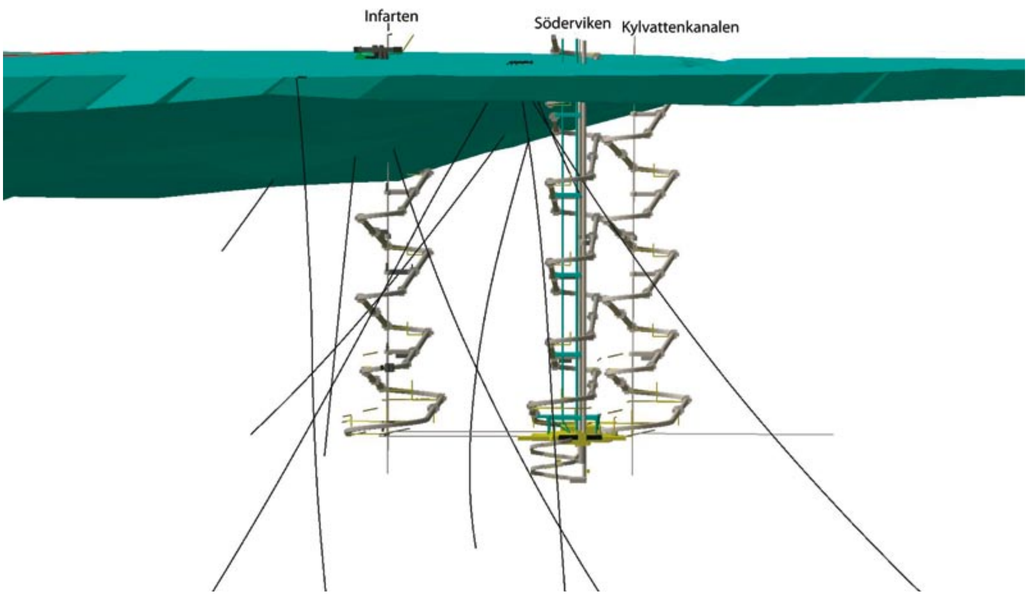


Figure 4-1. *General layout showing the location of the underground functional areas (Access, Central & Deposition Area) and the surface facilities.*



a) Plan view of the three locations showing the deformation zones within the vicinity.



b) Cross section viewed looking towards the southwest. The green-blue area represents fracture domain FFM02 while black lines represent cored boreholes. The ramps and shafts are also shown.

Figure 4-2. *Three optional locations examined for the surface facility. Note the thickness of fracture domain FFM02 near the Infarten Site.*

As noted in Chapter 3, fracture domain FFM02 is expected to have the greatest frequency of water bearing fractures. Hence a primary objective in choosing the surface facility was to limit the length of underground access through this fracture domain. Table 4-1 provides the findings from the study carried out by /Hansson et al. 2008/ who concluded that the *Söderviken* option met all the surface requirements while minimising the risks related to the underground excavations. Hence the *Söderviken* location is the recommended location for the Surface Facility.

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 (Figure 4-3). 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° 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 –470 m level. Minimum curve radius is set to 25 m. The total length of the ramp is approximately 4.7 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. The skip shaft shall accommodate transport and handling equipment for transport of rock, buffer and backfill material. The shaft shall also have room for electric cables for feeding to the central and Deposition Areas, 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.

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).

Table 4-1 Length of ramp excavation through fracture domain FFMO2 for the different surface facility options.

Option	Ramp length in FFM02	Estimated number of zone passages for the ramp (Zone)				
		ENE-2320	NNW-0404	1203	NNW-0100	ENE-1061A
Infarten	1,430	10	5	2	0	0
Kylvattenkanalen	550	0	0	0	Bordering	7
Söderviken	560	0	0	0	0	Bordering

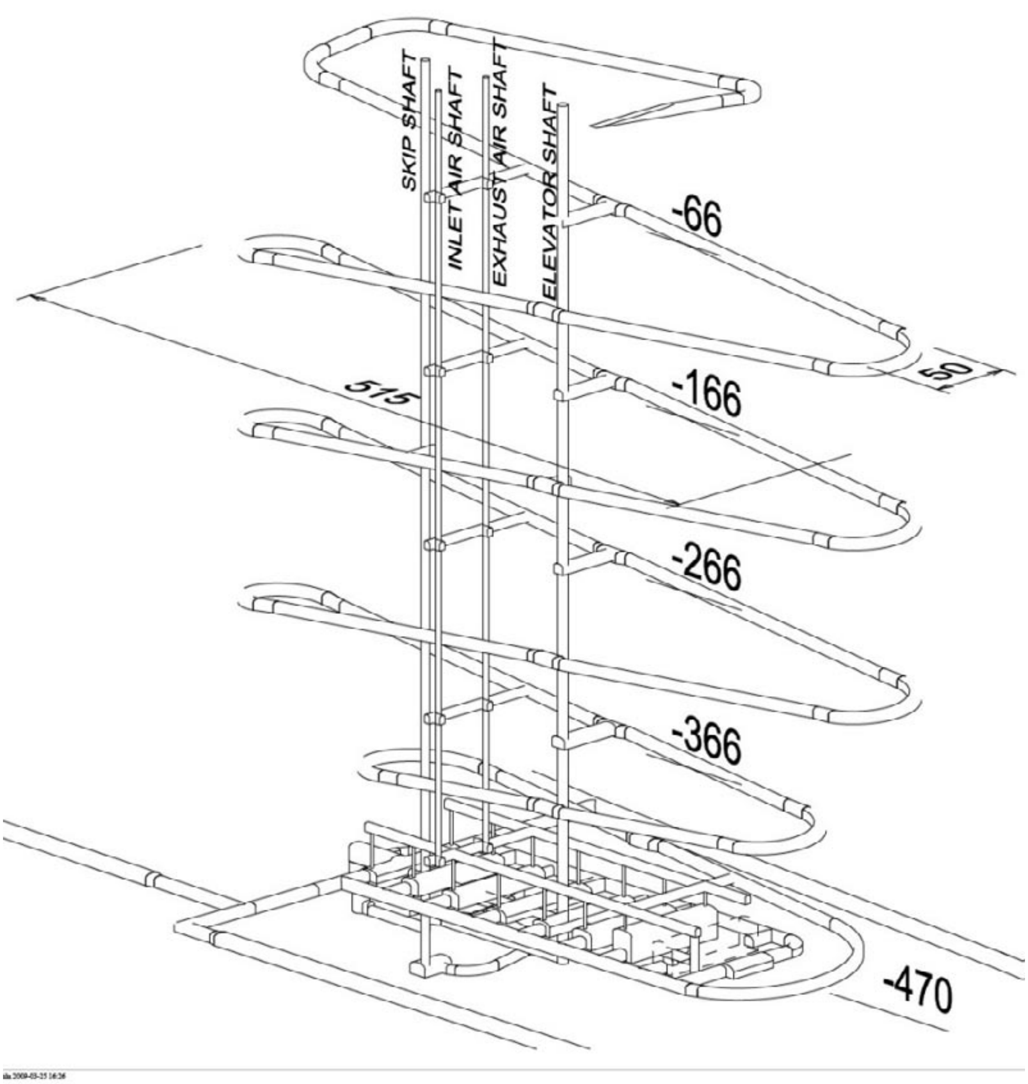


Figure 4-3. General view of ramp and shaft access from the ground surface to the Central Area located at a depth 470 m.

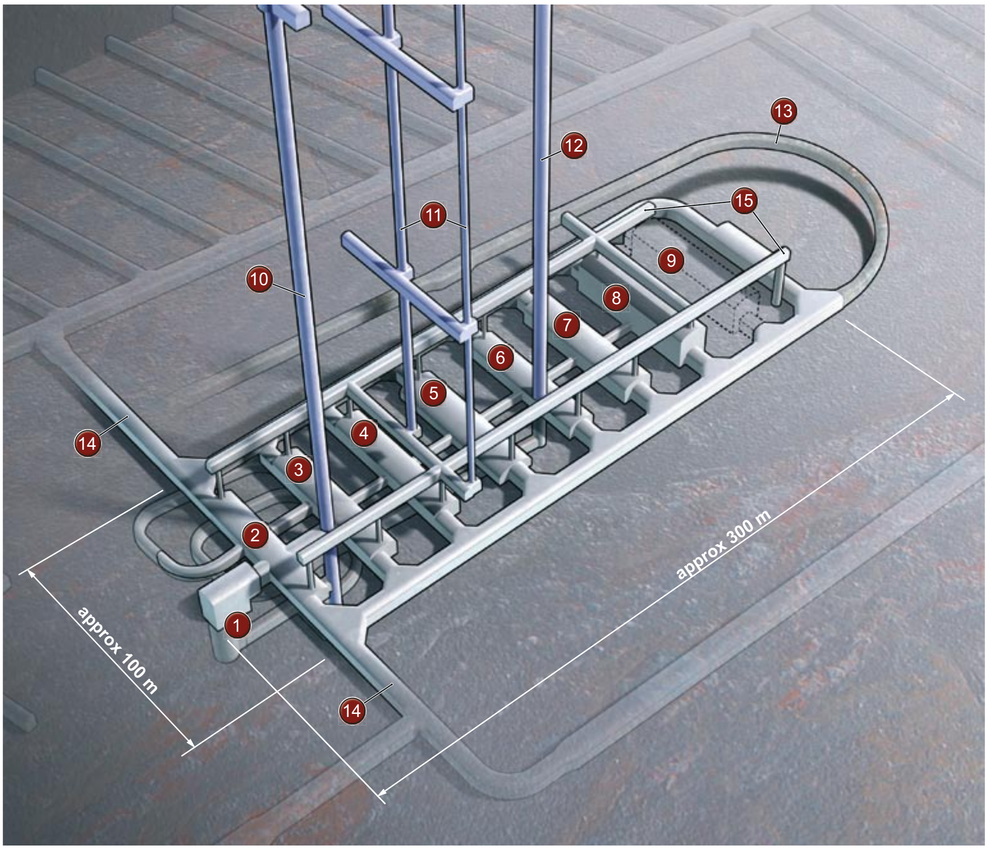
4.2.4 Ventilation shafts

There are one supply airshaft and one exhaust airshaft 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 rock hall and skip hall are placed nearest the Deposition Areas 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*.

As shown in *Figure 4-4* the Central Facility has several large caverns. /Carlsson and Christiansson 2007a/ using the experience from the SFR Facility noted that large caverns can be constructed in this rock mass at depths of approximately 100 to 130 m with minimal support. Therefore the stability of these caverns is not expected to be an issue and that traditional rock support will be adequate.



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=Lengt h.

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

As noted previously the location of Central Area was governed by the requirement to minimise length of the ramp excavations in fracture domain FFM02. As a consequence one of the access tunnels in the Central Area will be partially located within deformation zone ENE1061A (Figure 4-5). In addition one of the smaller deformation zones, NNW1205, also cross cuts the Central Area and tunnel accesses (Figure 4-6). These deformation zones are projections from borehole intersections and surface trace lengths and therefore their exact location at the depth of the Central Area are uncertain. During the Detailed Design their spatial location will need to be established more precisely and their potential impact on cavern stability evaluated using the Observational Method. At present all the geotechnical information for these deformation zones, SER /SKB 2008a/ suggests that impact of these deformation zones on the Central Area caverns will be very minor.

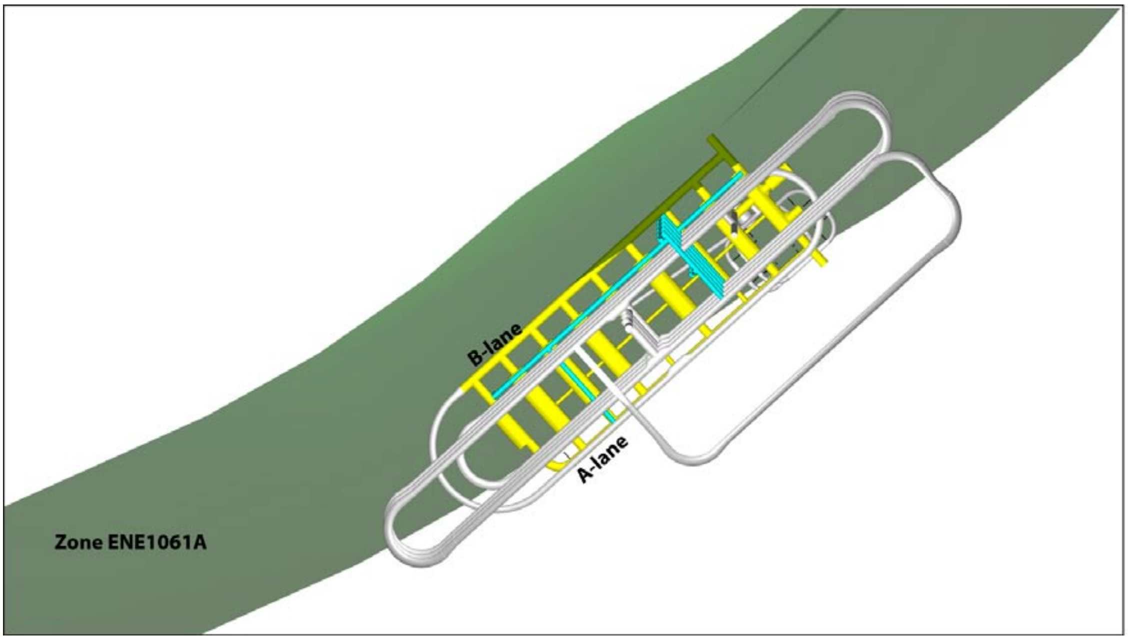


Figure 4-5. *Location of deformation zone ENE1061A close to the Central Area (plan view – dark green colour indicates the parts of the access main tunnel part that lie in the deformation zone).*

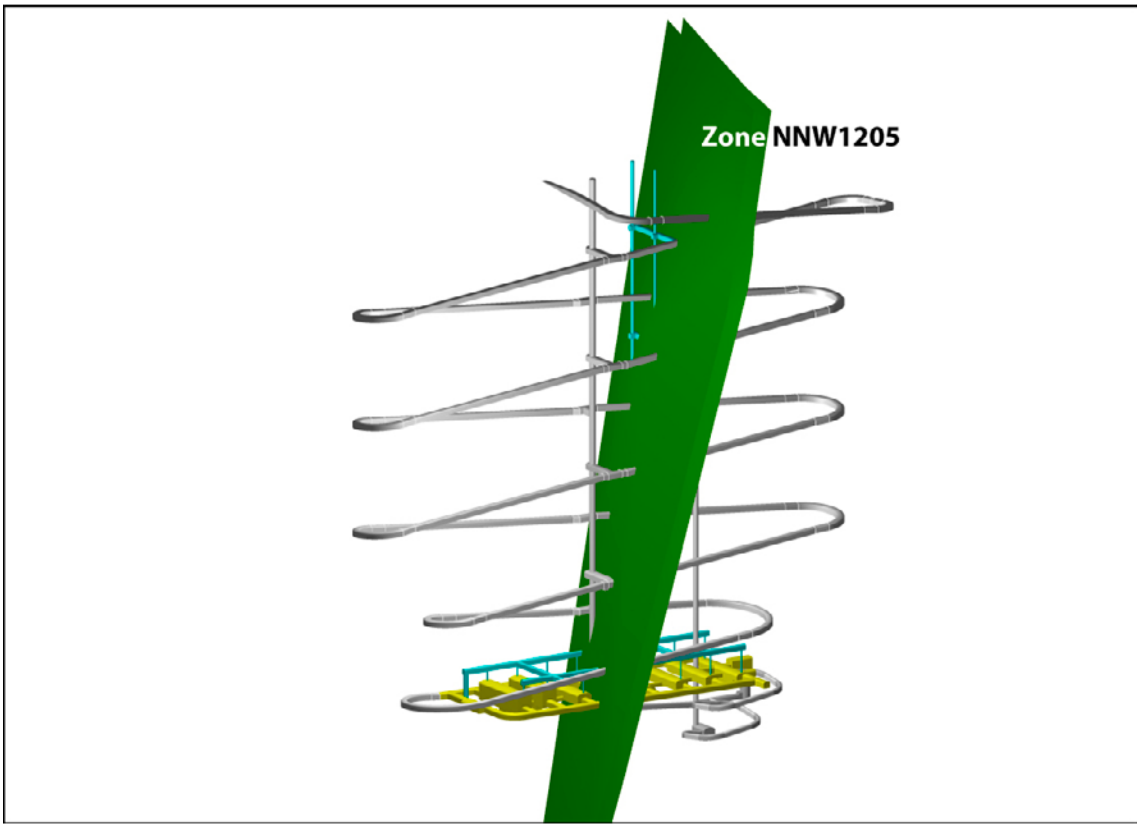


Figure 4-6. *The location of deformation zone NNW1205 relative to the Central Facility.View from the south.*

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 /SKB 2007/ and /SKB 2008a/. The facilities and operation are also adapted, as previously mentioned, to avoid unfavourable environmental consequences.

4.4.1 Layout constraints

There were a number of guidelines provided by SER /SKB2008a/ that constrained the layout of the Deposition Area. These are summarised below:

1. **Deposition holes and tunnels:** The Deposition Area was to be located within rock domains RFM029 and RFM045 /SKB 2008a/. The deposition hole centre-to-centre spacing was 6-m (RFM029) and 6.8 m in RFM045. 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-hole position lies at least 20.6 m from the entrance to the deposition tunnel and the last deposition-hole position will be located 10 m from the end of the deposition tunnel. Deposition-hole positions were not placed in deterministic deformation zones regardless of trace length. Figure 4-7 shows the location of the rock domains RFM029 and RFM045 and all the deterministic deformation zones.
2. **Tectonic lens boundaries**: /SKB 2008a/ established that the boundary for the tectonic lens is gradual increase in rock ductile strain (increased foliation intensity), which is of minor importance to tunnel construction or stability. Hence transport tunnels could be located outside the tectonic lens.
3. **Deformation zones**: Deformation zones A2,ENE060A, ENE062A and NW0123 require a Respect Distance, see Figure 4-8.
4. **In situ stress**: The deposition tunnels should be aligned parallel or sub-parallel (±30°) to the maximum horizontal stress (Azimuth 145 degrees) to minimise the stress magnitude concentration on the deposition tunnels and deposition holes.
5. **Repository depth**: The roof level of the highest located deposition tunnel shall be kept below the 450 m-level specified in SER /SKB 2008a/.
6. SER /SKB2008a/ noted that the rock mass quality of the deformation zones at Forsmark was suitable for the location of main and transport tunnels.

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

4.4.2 Transport to/from Central Area

The Central Area is located at the north-western edge of the repository and the repository will be developed from this area towards the southeast. As a consequence, the distance from the Central Area to the Deposition Area will become unnecessarily long in the later stages of operations. To overcome this, some deposition tunnels were established as temporary transport tunnels, i.e. short cuts (See Figure 4-9). This is further discussed in Sections 5.3 and 5.4.1. Two deposition tunnels would be used as one-way central transport tunnels during operation.

4.4.3 Ventilation

The Deposition Area must have a sufficient number of ventilation shafts to ensure a safe working environment and to enable ventilation at the deposition level. Because only limited regions of the Deposition Area will require a fresh-air flow, i.e. areas are backfilled once deposition is completed, two 3-m-diameter ventilation shafts will be adequate to meet the environmental and operational requirements. The proposed positions for these ventilation shafts have been selected in close co-operation with SKB experts on ventilation system and on environmental issues (see Figure 4-10).

Exhaust shaft SA02 will be constructed first and in operation at commencement of deposition works. This shaft will need to be located in vicinity of the Central Area, and preferably at one end of a main tunnel to provide ventilation and for smoke evacuation in case of fire. This shaft intersects 178 m of fracture domain FFM02 and may require pre-grouting. Suitable alternative locations will also encounter FFM02.

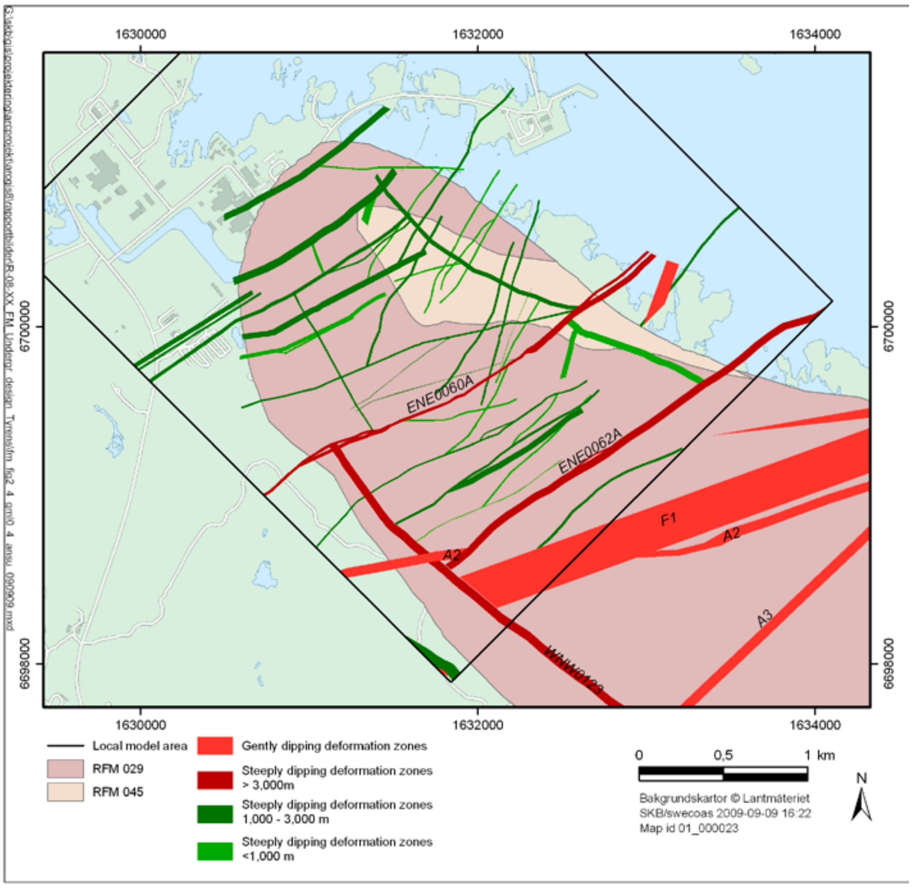


Figure 4-7. Map with all identified deformation zones at the depth of 470 m.

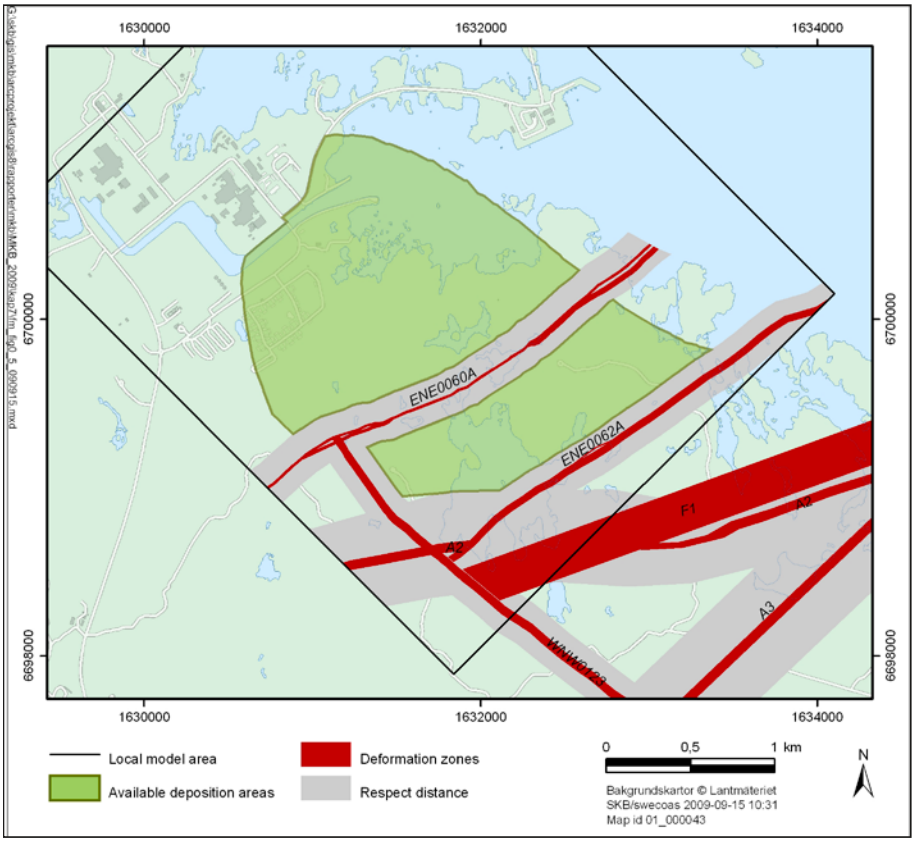


Figure 4-8. Available Deposition Area at 470 m depth, after complying with respect distance requirements for deformation zones >3,000 m.

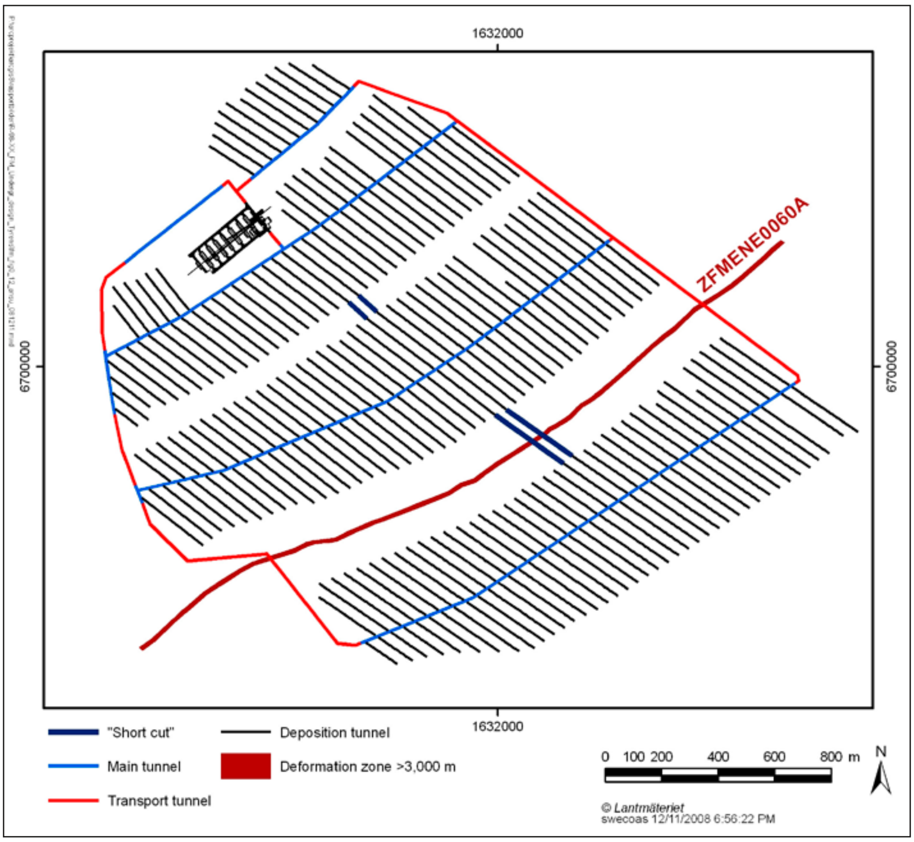


Figure 4-9. *Position of temporary transport tunnels ("Short cut") at 470 m depth.*

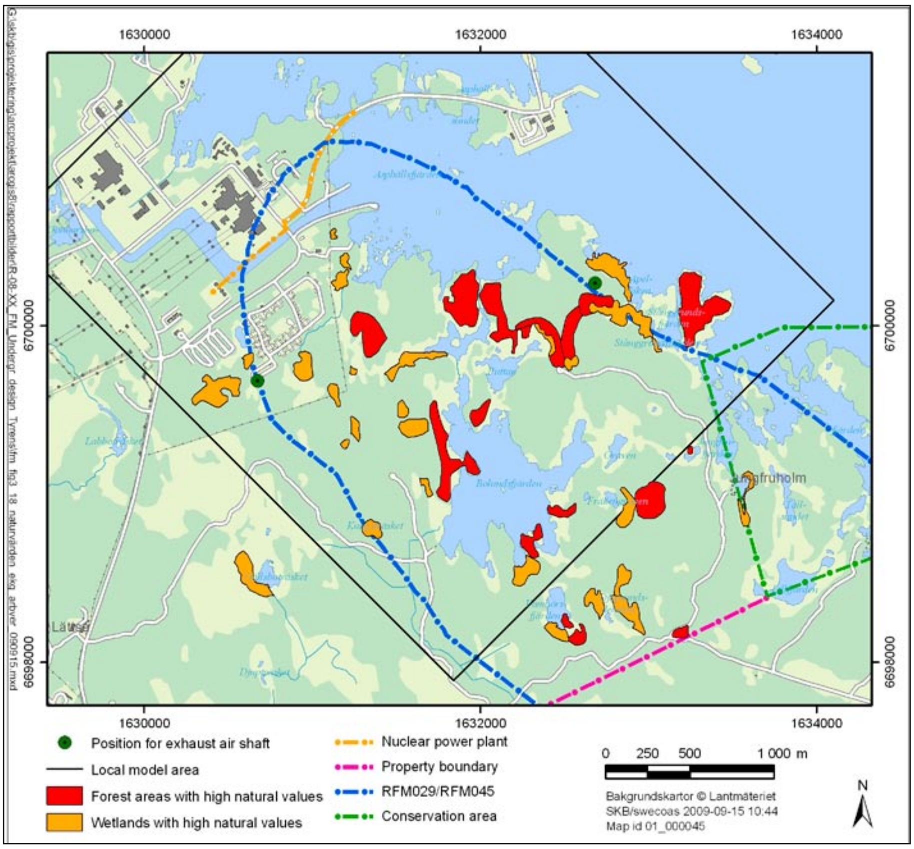


Figure 4-10. *Proposed locations for ventilation shafts.*

The second exhaust shaft (SA01), will be constructed after approximately 20 years of repository operation. This shaft will intersect fracture domain FFM02 for approximately 109 m and also pass deformation zone B7 at a depth of 310 m, see Figure 4-11. As with the other gently dipping zones at Forsmark, B7 may be locally severely water-conducting and difficult to seal, see / Brantberger and Janson 2008/. It is anticipated that both shafts will be constructed using raise-boring technology and that the construction of both ventilation shafts will require pre-grouting. The pre-grouting would be carried out from the surface and from the underground excavations.

As noted previously only limited portions of the repository will require fresh air at any given time as the repository will be backfilled once deposition is complete. Ongoing studies are evaluating the possibility of optimising the air-flow such that one or both of the exhaust shafts may be deleted. The detailed design studies will report the findings from those studies.

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 4-12). Local pumping pits are approximately 1 km from each other, providing a maximum head difference of approximately 5 m. The drainage water will either be pumped from one location to the next Deposition Area, or led by gravity led to a pumping pit or the Central Area. At the Central Area, primary filtration/settling basins will be used to remove sediments and oil residues. From these basins the water will be pumped up to the surface water treatment plant. The water handling system will be designed to withstand a power outage of at least 24 hours for the Central Area electrical system. In case of an emergency, e.g. a major fire, explosion, etc, where a power outage occurs for longer periods, an additional storage capacity for the drainage water is handled by an automatic overflow system that leads the surplus drainage water to the bottom of the skip shaft. Other options may be available but have not been explored for this design step.

Drainage water from ramp and shafts in the Central Area will be collected at every 100 m level, where it will be pumped up to the surface water treatment plant. Since most of the leakage into the ramp and shafts is expected in the upper parts located in fracture domain FFM02, the major portion of the total drainage will only have to be pumped up 100 m.

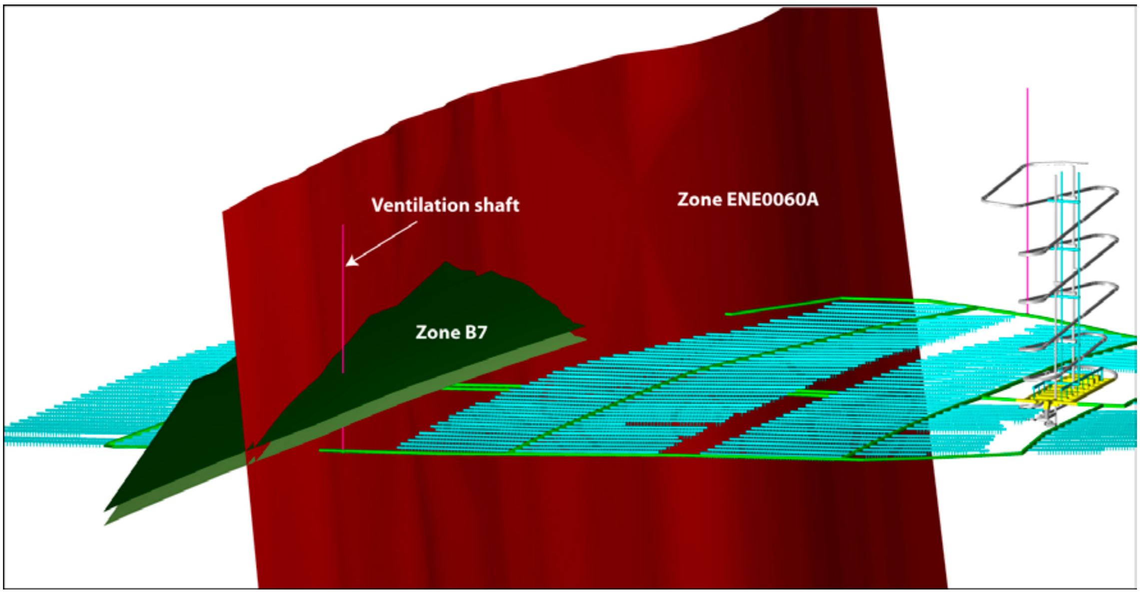


Figure 4-11. *Illustration showing the intersection between ventilation exhaust shaft (SA01) and zone B7 at a depth of 310 m. Underground facilities at –470 m. View from northeast.*

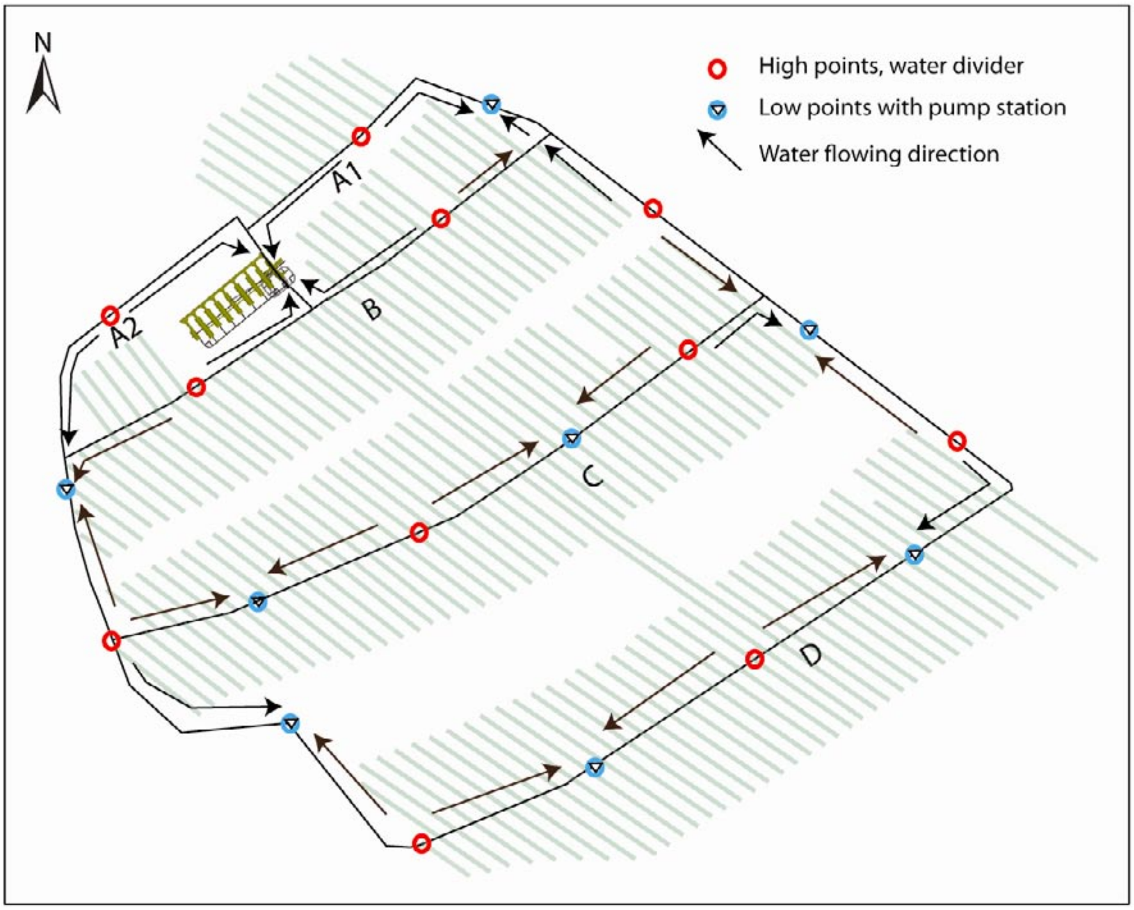


Figure 4-12. *Drainage system, plan at 470 m depth. The water will run from the higher points (water divider) to lower points where it will be pumped on to next section.*

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-hole 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-hole positions

The proposed layout that meet all the requirements given in SER and UDP/D2 is shown in Figure 4-13. The gross capacity of repository is 7,818 canisters, allowing for approximately 23% loss of deposition-hole positions. The alignment of the deposition tunnels is approximately 22 degrees to the maximum horizontal stress.

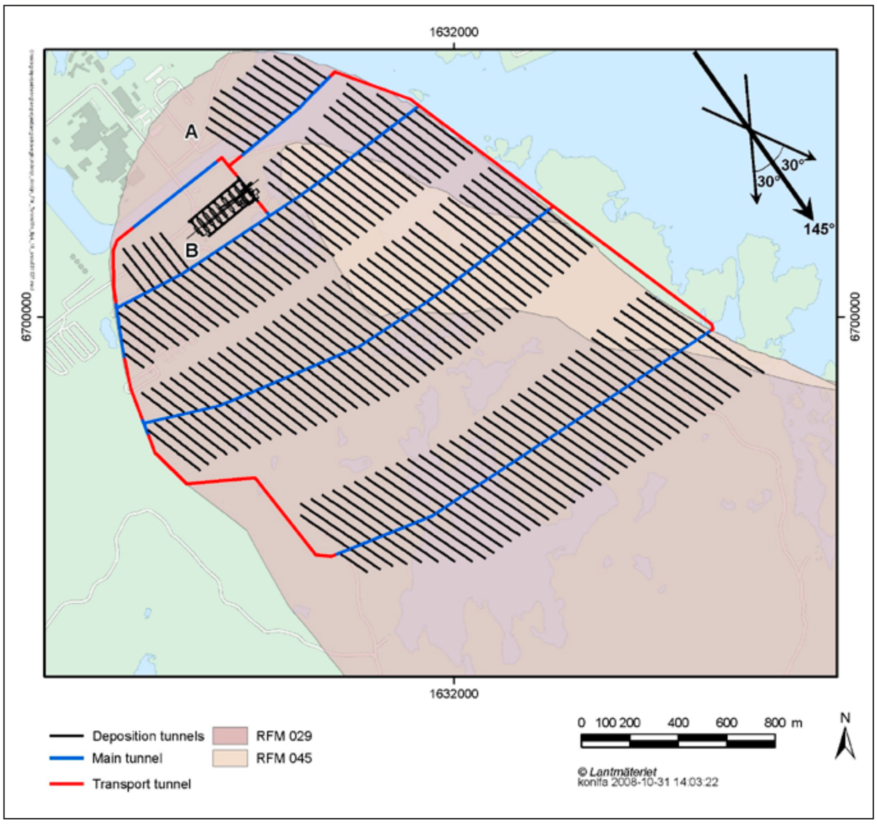


Figure 4-13. *Deposition tunnels relative to the orientation of the major horizontal stress, section at 470 m depth. The Figure represents the area for a gross capacity of 7,818 canisters (i.e. allowing for a possible loss of 23% due to discarded deposition positions).*

An alternative layout was investigated to assess the space required to achieve a utilisation factor of 70%, i.e. 30% loss of deposition-hole positions. Figure 4-14 shows the potential space that would be required to meet this 30% target. While the space is available it would encroach on an area underneath the nuclear power plant (FKA) on the northwestern edge of the tectonic lens and it would also require utilising an area to the east of deformation zone ENE0062A. This 30% loss would provide for a gross capacity of 8,571 deposition-hole positions. Other options may be available but have not been explored for this design step.

Transport tunnels

As noted previously the boundary of the tectonic lens is geologically defined based on a gradual increase in ductile strain and therefore is not an abrupt boundary. Experiences from previous tunnel construction works in the Forsmark area suggests that in most of the rock mass good to fair tunnelling conditions can be expected /Carlsson and Christiansson 2007a/. It is therefore proposed that the transport tunnels shall follow the borders of the tectonic lens in order to maximally utilise the domains, characterised as having the eligible properties for deposition, Figure 4-15. At the western side of the tectonic lens the geometric boundary of the lens is considered more defined resulting in a more definite change of rock properties. Consequently the western transport tunnel has been located just inside but at border line of the tectonic lens. The exact location of the borders of the tectonic lens can only be resolved by investigations during construction.

To reduce the transport required for the separated construction and deposition activities is it proposed to use two parallel deposition tunnels as transport routes. This temporary transport route is preferably located central in the Deposition Area (see Figure 4-9). These transport tunnels will at end of the operation period for the repository be re-established and used for deposition of canisters.

The alternative with temporary transport tunnels will result in a negligible increase in environmental impact due to a larger amount of excavated rock, but a quite large reduction in environment impact due to decreased quantity of fuel, and consequently less emission.

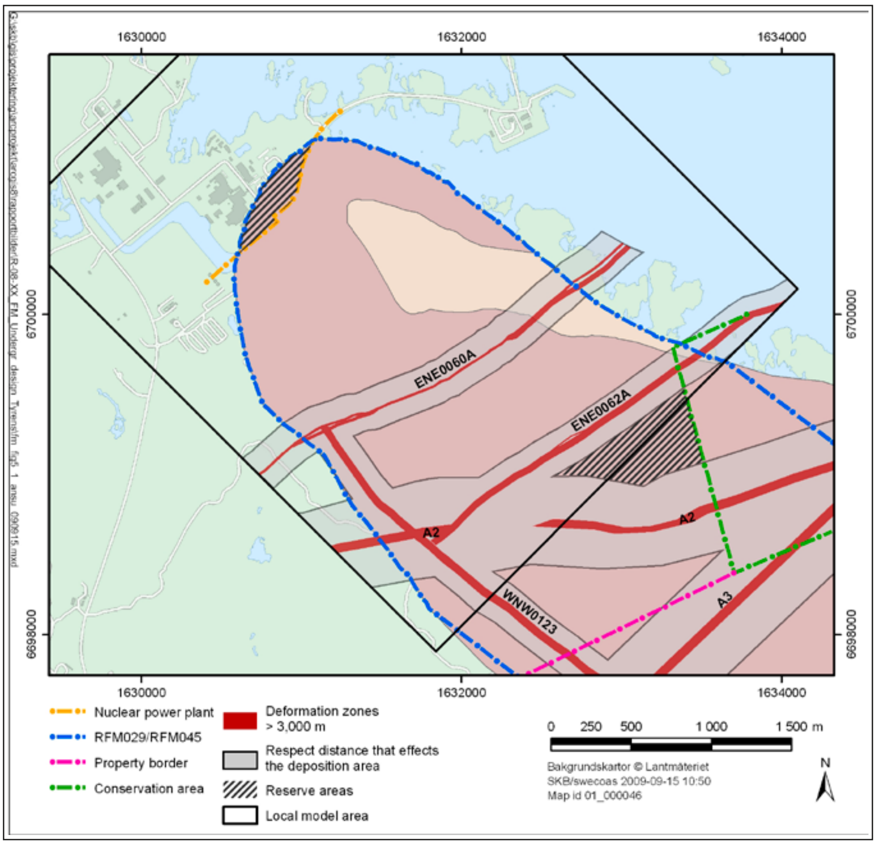


Figure 4-14. *Identified reserve areas (in case of a loss up to 30%) at 470 m depth. One area south of deformation zone ENE0062A and one area underneath the nuclear power plant, FKA.*

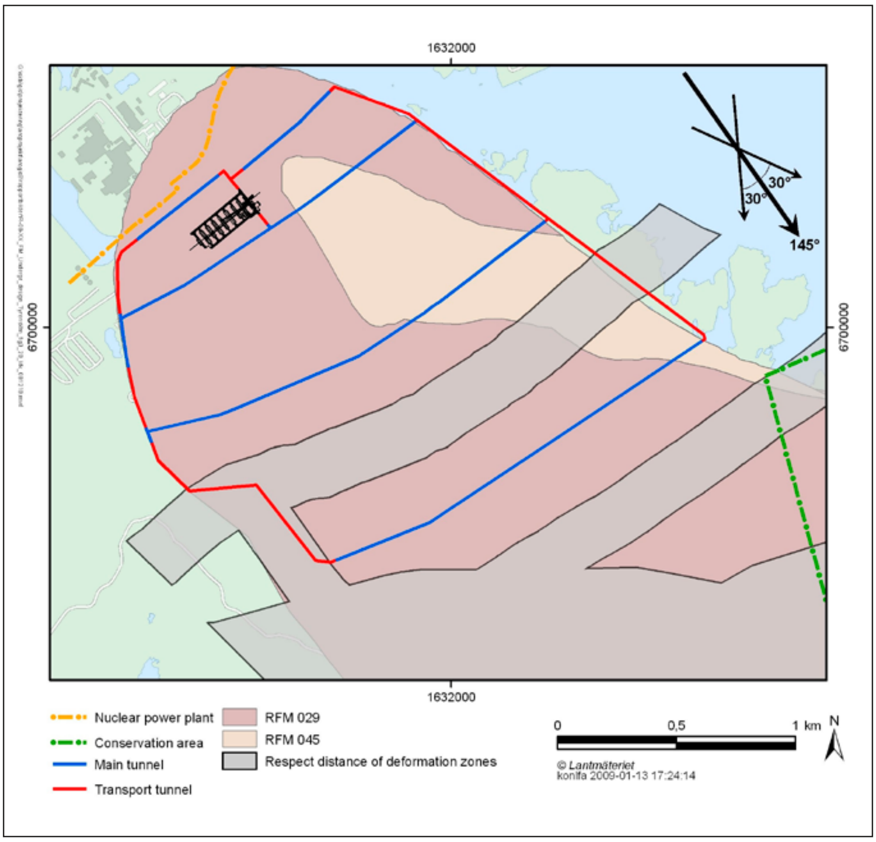


Figure 4-15. *Location of proposed main Tunnels and Transport tunnel at depth 470 m.*

Main tunnels

Four northeast-southwest aligned main tunnels are required for full utilisation of the available area at 450–470 m depth. As noted previously portions of these tunnels are located within close proximity of some minor deformation zone (Figure 4-16). If this location is not used, the minor deformation zones will intersect the Deposition Areas and consequently decrease the number of deposition-hole positions available. By locating main tunnels in minor deformation zones, there is a risk for increasing the amount of grouting and rock support. Given the construction experience from Forsmark and the geological description of these deformation zones in SDM Site, these risks are very low. However, the flexibility for optimising the layout and maximising the utilisation of the space available is greatly increased. The exact location of the minor deposition zones can only be resolved by investigations during construction.

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 4-2.

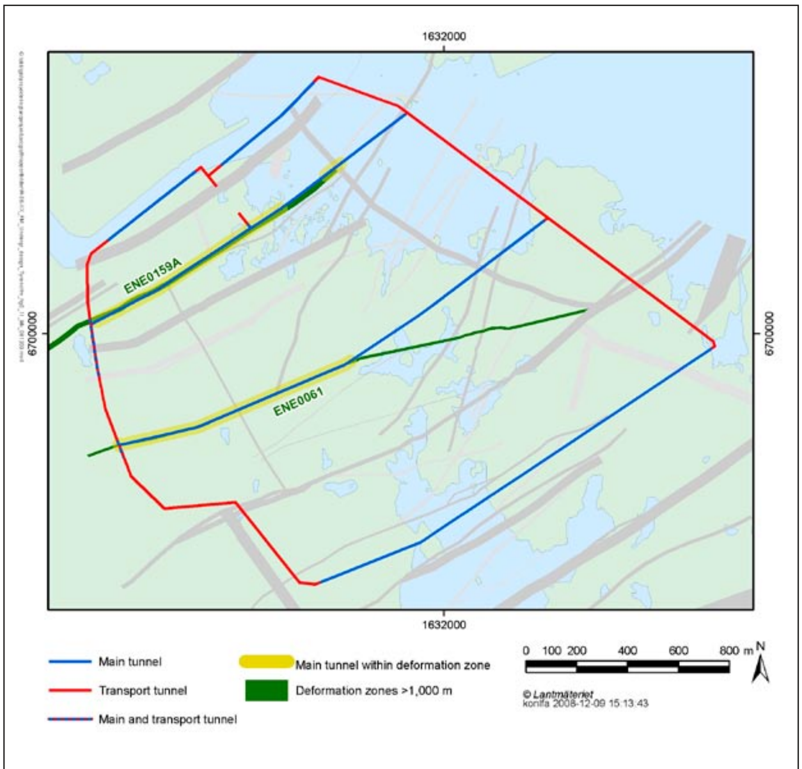


Figure 4-16. Location of main and transport tunnels proposed to be located within deformation zones at 470 m depth due to the good rock quality and low transmissivity of these NE trending deformation zones.

Table 4-2. Tunnel length and excavated volumes for tunnels at repository level –470.

	Theoretical volume ($m^3 \times 10^3$)	Length (km)
Repository Access		
Ramps & shaft	205	
Rock loading station	7	
Central area		
Caverns	112	
Deposition Area		
Transport tunnels	182	4.6
Main tunnels	384	6.4
Deposition tunnels	1,171	61.1
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 must always be separated by a physical protection allowing for no contact between the activities. These development steps may range a few years up to about 10 years.

The operation and construction plan shall provide basis for the required 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, all work tasks were analysed, and a time allocated for each activity (see details in /Hansson et al. 2008/). 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 (max length approx. 300 m) 105 ± 10 weeks *(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 *(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* is described in previous SKB reports, e.g. see /SKB 2006c/ and an example of possible construction steps for the Forsmark D1 layout is given in Figure 5-1. This method simply requires the excavation and deposition activities to alternate sides as a deposition panel is excavated and filled. This requires that these two activities be tightly controlled to maintain production efficiency.

The main disadvantage with the side-change strategy is the reduction in construction efficiency as the rock excavation changes from one side to the other, see Figure 5-2. 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 20 weeks to complete. If the deposition of canisters immediately continues at the other side, this would require the excavation work to stop for a minimum of 20 weeks at both sides, since deposition work would be ongoing on both sides. Another disadvantage occurs at end of the repository's lifetime when only one main tunnel remains and no side-change is possible (see Figure 5-1, construction step 13–15). To fulfil the separation requirement this will necessitate that all excavation activities in this main tunnel be constructed before deposition starts, i.e. deposition will have to be delayed for up to 10 years. While it may be possible to reduce these time lags, it is obvious that the side-change strategy has significant scheduling and efficiency penalties.

An alternative to side-change construction strategy, *Separation by linear development method* is described in the next section.

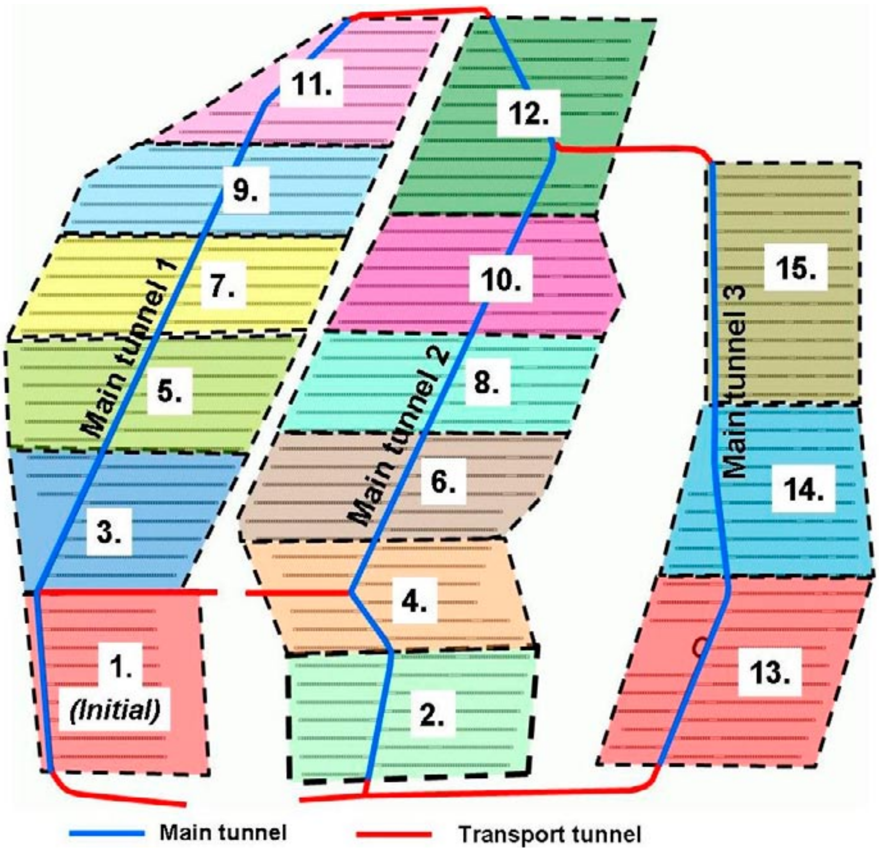


Figure 5-1. Example of the construction steps planned for the D1 Layout (for example in three-year steps). The numbers indicate in which order the steps are constructed applying Separation by side-change /Hansson et al. 2008/.

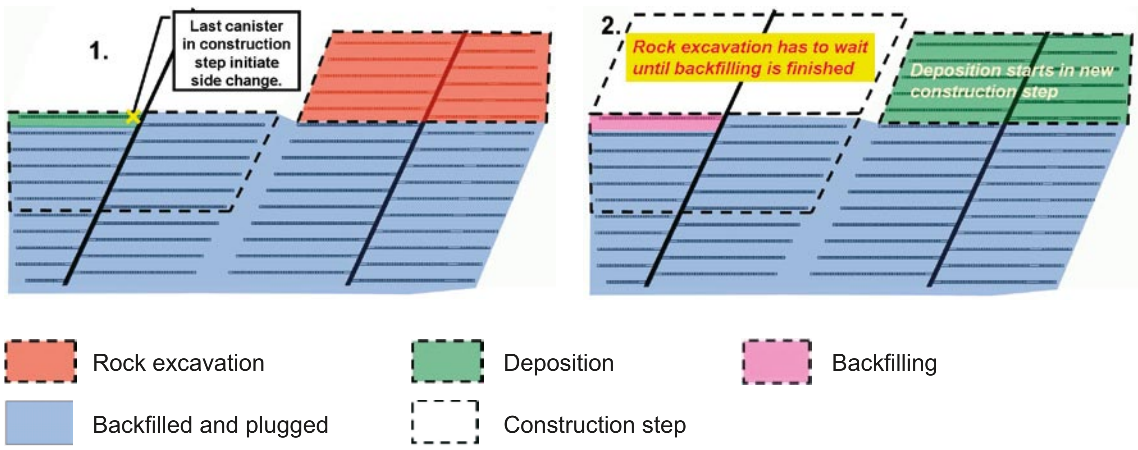


Figure 5-2. Simplified example of reduction in operational efficiency related to the side-change method (Hansson et al. 2008/.

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 5-3, and be separated by a barrier (wall/door).

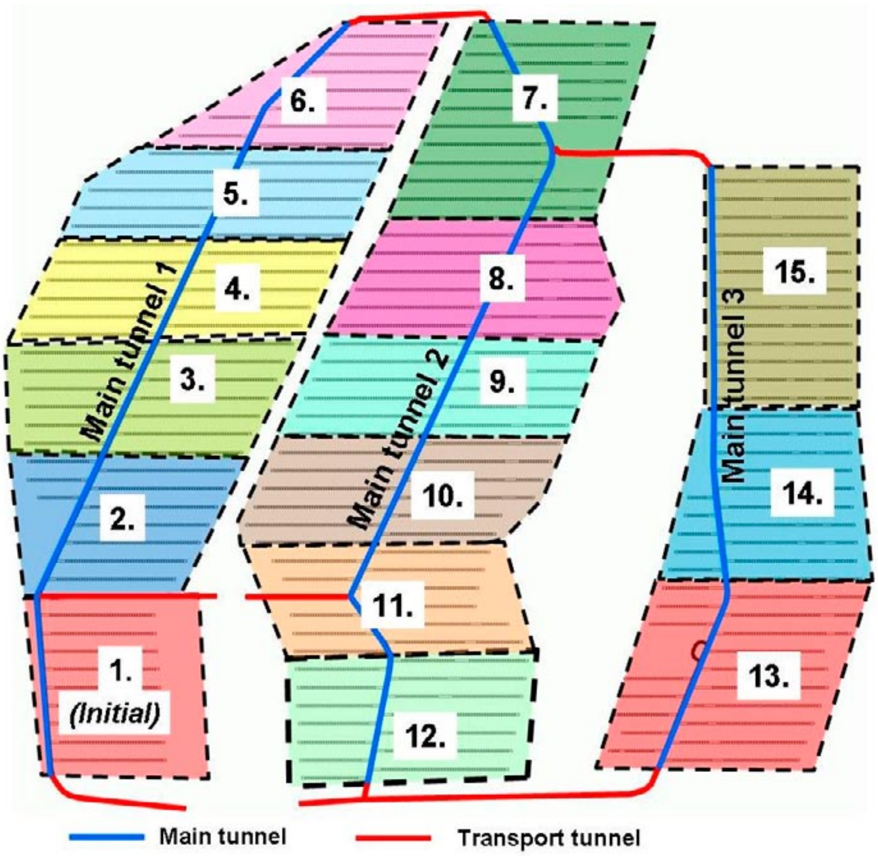


***Figure 5-3.** Example of construction steps for the D1 layout (assuming approximately 3 years step intervals) when applying the Separation by linear development method /Hansson et al. 2008/.*

Figure 5-4 illustrates a detailed-sequence that could be used in 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 5-4) 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 5.4.3.

5.2 Strategy for step-wise excavation/operation

A tentative plan for the construction of Forsmark underground facility has been developed assuming time-steps of approximately 4 years /Hansson et al. 2008/. 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 5-5). The principle for step-wise development is given in Figure 5-6. A proposed development sequence for a timeline of up to 50 years is given in Appendix B.

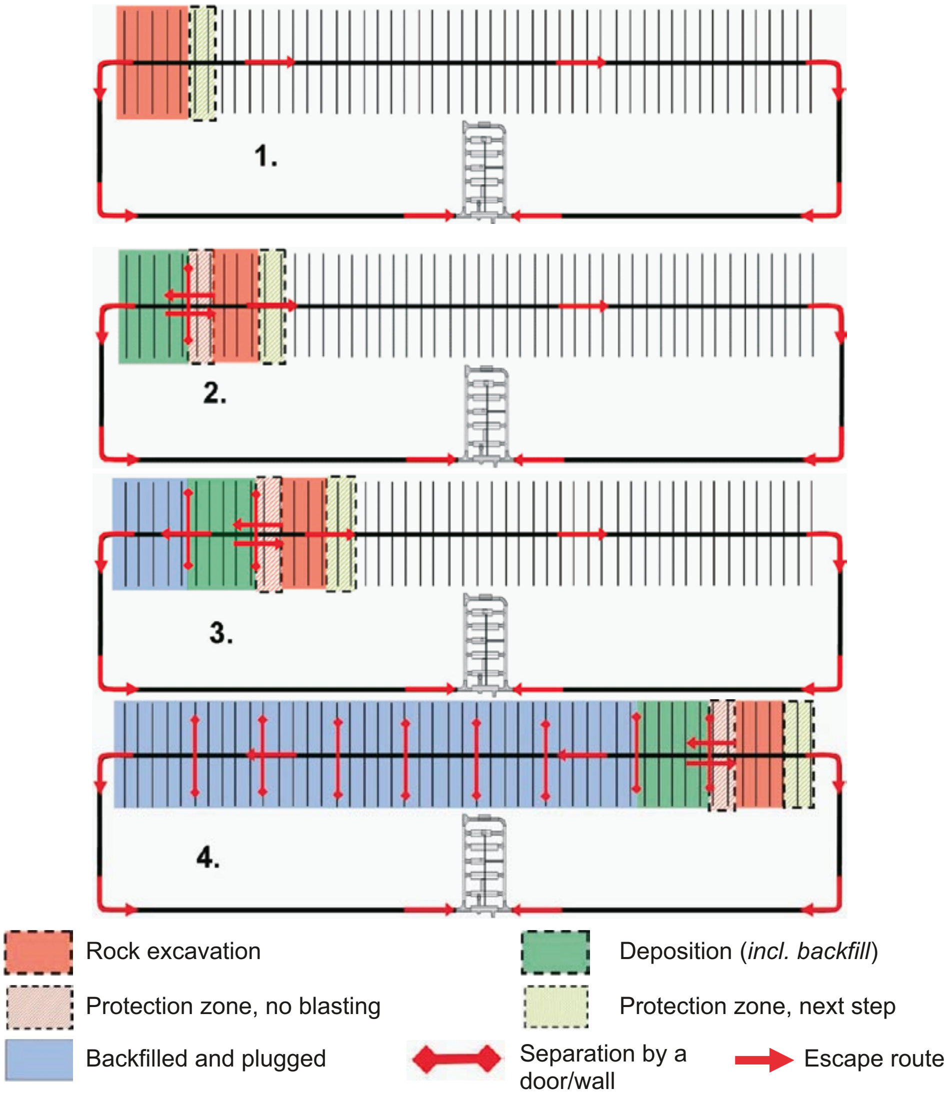


Figure 5-4. *Schematic view of construction of the repository using the "Separation by linear development method". This method ensures that two evacuation paths are always available /Hansson et al. 2008/.*

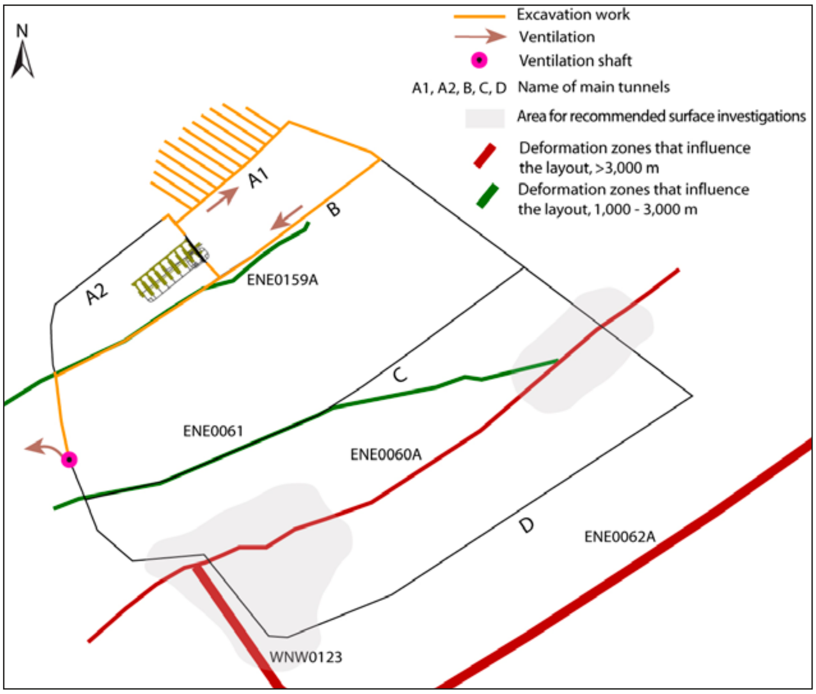


Figure 5-5. Location for the proposed excavation, year 0.

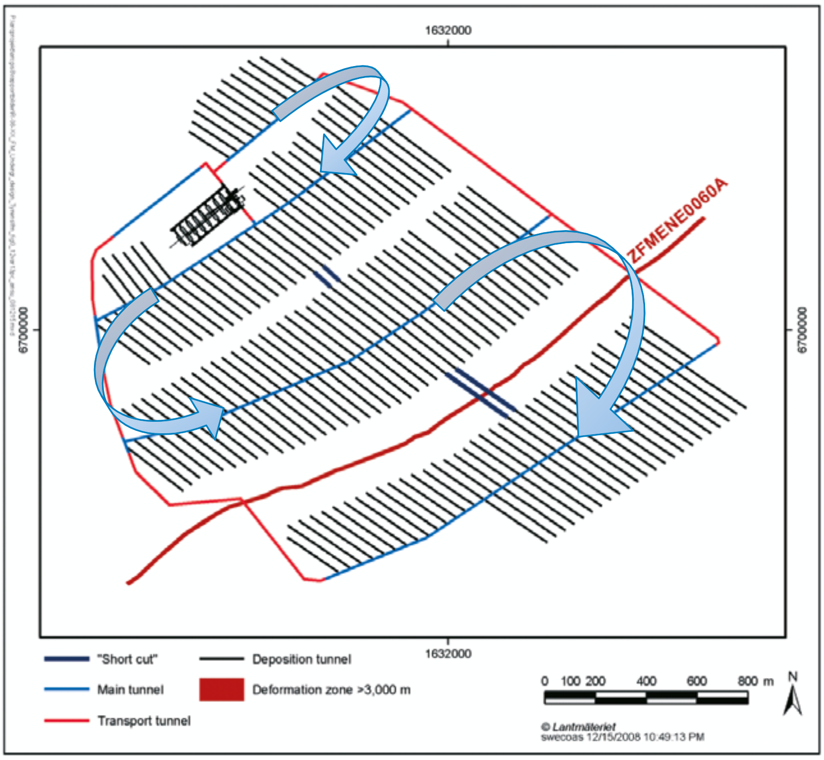


Figure 5-6. The proposed sequencing development of the repository.

The proposed development strategy (Figure 5-6) allows for incorporation of site investigation plans that will ensure the tectonic lens is developed for 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 boundaries of the tectonic lens. 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 B comprise the following volumes (excl. of deposition holes), given in theoretical in situ volume ($m^3 \times 10^3$), see Table 5-1 below.

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 5-7. 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 5-7. 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.

Table 5-1. Volumes of tunnel excavation for each construction step (excluding deposition holes).

	Year													
Type of tunnel	**0**	**5**	**9**	**12**	**15**	**18**	**22**	**26**	**30**	**34**	**38**	**42**	**46**	**Total**
Transport (10^3 m^3)	26	81	7	0	0	19	26	0	23	0	0	0	0	182
Main (10^3 m^3)	131	30	0	0	114	0	0	0	46	63	0	0	0	384
Deposition (10^3 m^3)	57	120	61	54	70	134	102	104	73	99	72	123	102	1,171
Total (10^3 m^3)	214	231	68	54	184	153	128	104	141	162	72	123	102	1,736

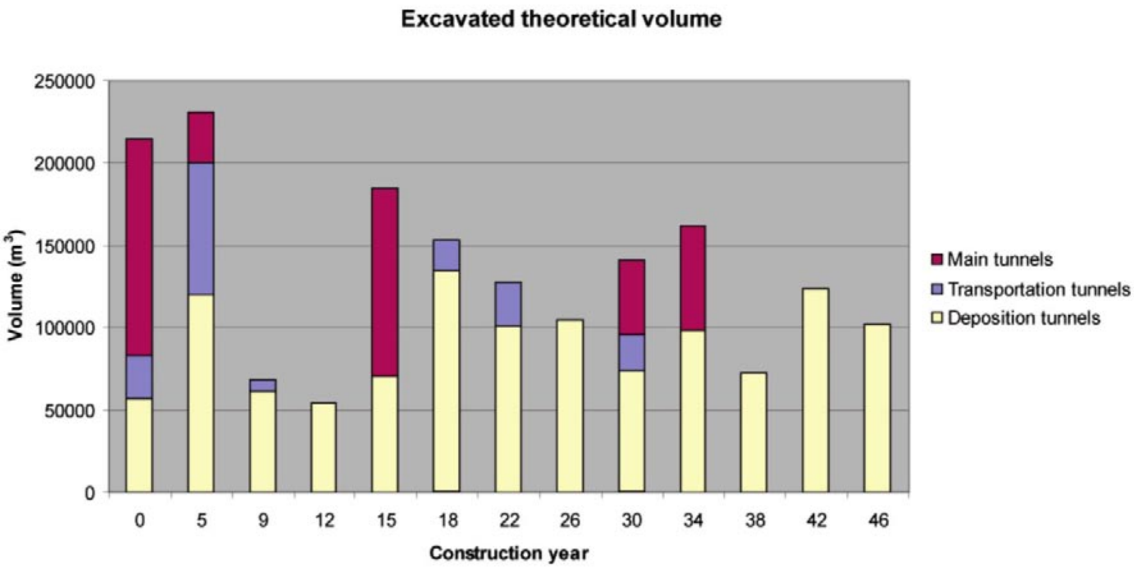


Figure 5-7. *Estimated excavated volumes for each construction step.*

Table 5-2. Total length of tunnel types.

Type of tunnel	Km tunnel length
Transport tunnels	4,6
Main tunnels	6,4
Deposition tunnels	61,1

5.3 Transport issues during operation

The linear-development method will generate an increased amount of transport work compared to the alternative strategy with Separation with side-change. The reason for the increased transport work is that there will be a need to get access at both ends of a main tunnel during work, see Figure 5-8, as the only alternative would be to transport the excavated rock mass through the Deposition Area which is unacceptable. The impact on project economy due to increased transport work is considered of minor importance, but with the ambition to fulfil environmental objectives it is essential to find solutions that can reduce the transport work.

The increase in transport work has for the recommended layout using the Separation with linear development method been estimated at some additional 20%, see Table 5-3. Since no feasible layout, from a functional point of view, has been prepared for the side-change alternative, the estimated transport work for this alternative is calculated for the shortest possible transport route without considering any restraint from the other works. The obtained estimate consequently is the theoretical minimum of transport work for the side-change alternative, i.e. a most conservative value.

The transport work according to Table 5-3 below can also be expressed in a diagram, see Figure 5-9.

With the objective to reduce the transport work "shortcuts" have been introduced by connecting deposition tunnels from different main tunnels and to temporary use these deposition tunnels for transport of rock and/or backfill. Since the section of a deposition tunnel is too narrow to allow for traffic to meet, two tunnels with one-way traffic have been arranged, see Figure 4-9. The introduced shortcuts result in a reduction of transport work of some 20–30% depending on which material that is transported.

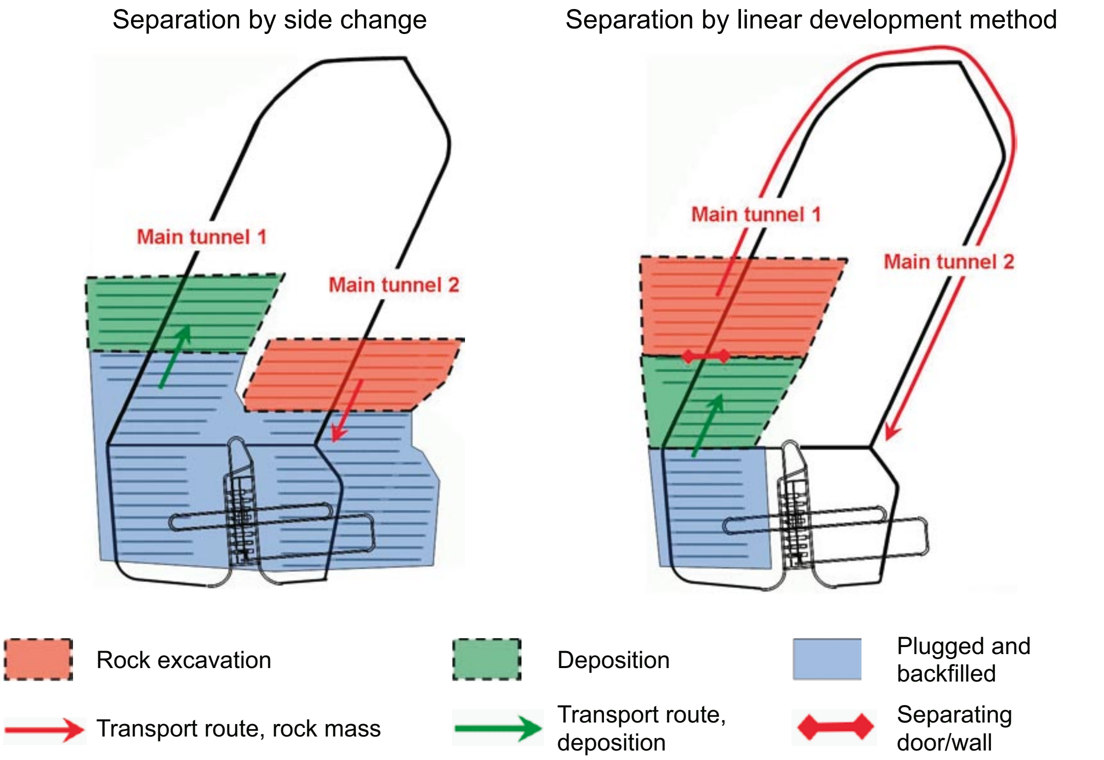


Figure 5-8. Example of increased transport work when the separation by linear development method is used.

Table 5-3. Estimated amount of transport work for the two excavation alternatives.

Alternative	Transport work (ton*km*10⁶)			
	without "shortcuts"		with "shortcuts"	
Transport of	*Rock*	*Backfill*	*Rock*	*Backfill*
Separation by linear development method(proposed alternative)	11,3	5,2	8,2	4,4
Separation by side-change	9,8	4,6	6,9	3,2

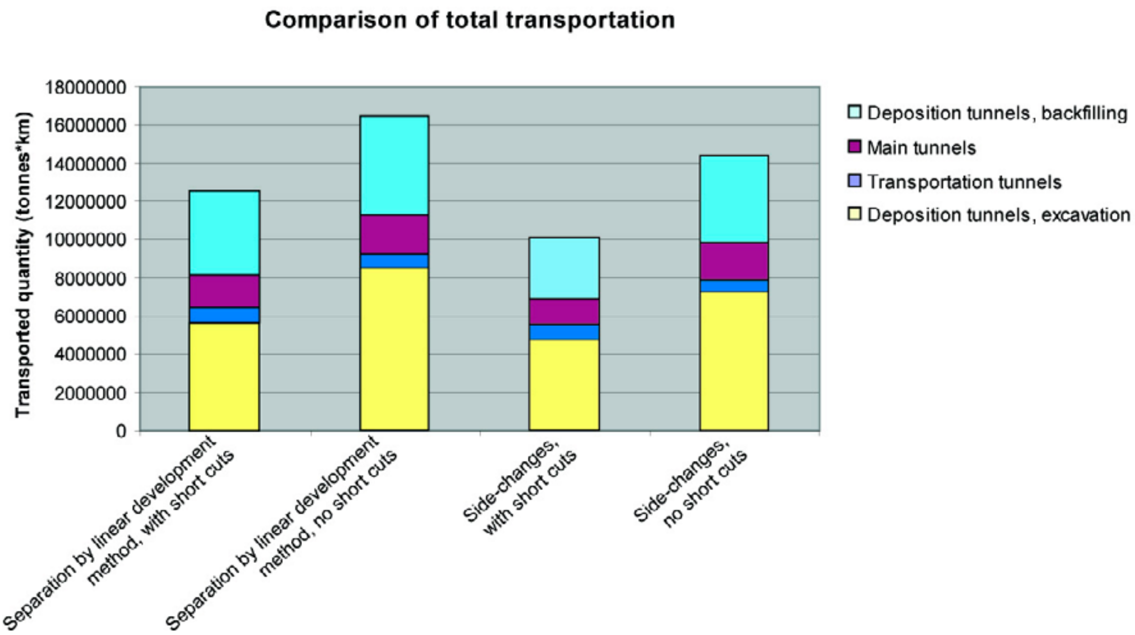


Figure 5-9. *Total transport work quantities for each of the various construction strategies. The bar furthest to the left represents the construction strategy proposed in this study.*

5.4 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 /Hanson et al. 2008/.

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).

5.4.1 Escape routes

As a general rule two separate escape routes shall be required for all working areas as soon as practical. This requirement is met by the Separation by linear-development method, which ensures that two separate exits are always accessible. The separation of the deposition activities from construction strictly stipulates that transports are not allowed or will be possible across the separating door/wall. However, in case of an emergency, this separation rule does not apply and emergency arrangements will be made for passing these separating doors/walls. The outline of this escape route strategy is illustrated by Figure 5-4, as well as in Appendix B.

5.4.2 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 Areas.

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.

5.4.3 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.

5.5 Summary

The two development strategies, separation by side-change and separation by linear development method are compared in Table 5-4. The advantages of the Linear-development method are self evident and are recommended as the preferred option for the reference design.

Table 5-4. Comparison of the side-change strategy with separation by linear development method.

Description	Side change	Linear development
• Early allocation of rock volume for the repository development (= flexibility in layout development).	–	+
• Long-term effects of decisions or incidents at an early stage of development (difficult to predict).	–	+
• Operative planning for extension of repository (degrees of freedom for future development).	–	+
• Dependency between rock construction, deposition and backfilling.	–	+
• Utilisation of the total rockwork capacity.	–	+
• Required number of TBM-drilling equipment.	–	+
• Transportation of equipment and personnel.	–	+
• Sensitivity to disturbances (operation greatly dependent on the layout).	–	+
• Deposition holes kept open for a long period of time before being used.	–	+
• Workers safety and escape routes.	± 0	± 0
• Transport work (distance) for excavated rock and backfill.	+	-

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 /Palmström and Stille 2007/ and summarised in the SER /SKB 2008a/ as given i Table 6-1.

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. 2008/ 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.

Table 6-1. General categories for ground behaviour (GB) /SKB 2008a/.

GB1.	Gravity driven, mostly discontinuity controlled failures (block falls), where pre-existing fragments or blocks in the roof and sidewalls become free to move once the excavation is made.	
GB2.	Stress induced, gravity assisted failures caused by overstressing, i.e. the stresses developed in the ground reaching the local strength of the material. These failures may occur in two main forms, namely:	
	GB2A	as spalling, buckling or rock burst in materials with brittle properties, i.e. massive brittle rocks;
	GB2B	as plastic deformation, creep, or squeezing in materials having ductile or deformable properties, i.e. massive, soft/ductile rocks or particulate materials (soils and heavy jointed rocks).
GB3.	Water pressure; an important load to consider in design especially in heterogeneous rock conditions.	
	GB3A	Groundwater initiated failures may cause flowing ground in particulate materials exposed to large quantities of water, and trigger unstable conditions (e.g. swelling, slaking, etc.) in some rocks containing special minerals. Water may also dissolve minerals like calcite in limestone.
	GB3B	Water may also influence block falls, as it may lower the shear strength of unfavourable joint surfaces, especially those with a soft filling or coating.

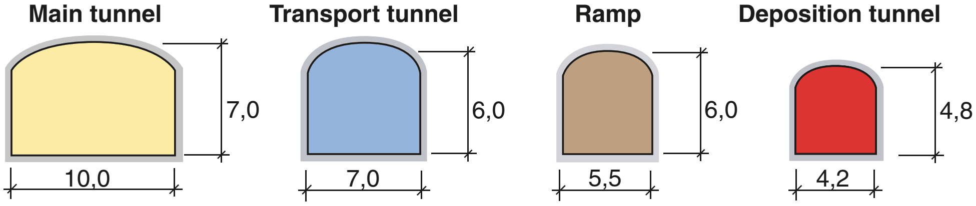


__Figure 6-1.__ Profiles and dimensions of tunnels used for repository access and deposition.

6.1 Analysis of the system behaviour

The system behaviour was analysed using different methods for various parts of the repository. In the upper parts of the repository, down to a depth of approximately 150 m (i.e. fracture domain FFM02), the system behaviour was assessed using the construction experience and performance of the SFR facility /Carlsson and Christiansson 2007a/. Below 150 m depth the system behaviour was assessed using:

- 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.

/Eriksson et al. 2008/ established the distribution of Ground Types and the associated Ground Behaviour for each of the fracture domains (FFM01, FFM02 and FFM06) and the minor deformation zones (MDZ) (Table 6-2). The Ground Behaviour was assessed using both the "most likely" and "unlikely maximum" stress models. The distribution of Ground Behaviour in each of the functional areas expressed in linear metres (tunnel length) is given in Table 6-3.

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%. At present the long legs of the ramp are oriented NE-SW, which means that long portions of the ramp will be approximately perpendicular 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" stress model.

The distribution of expected Ground Behaviour for the Repository Access is summarised in Table 6-3. For the most "likely stress" model the Ground Behaviour is expected to be dominated by gravity-induced structurally-controlled block-falls (wedges).

Table 6-2. Distribution of Ground Behaviour for each Ground Type in each of the fracture domains (FFM01, FFM02, FFM06) and the minor deformation zones (MDZ).

	Description	FFM01 & FFM06 (%)	FFM02 (%)	MDZ >70° (%)	MDZ <70° (%)
Expected case					
GT1–GB1	Sparsely fractured, isotropic rock with gravity driven, mostly discontinuity controlled failures (block falls).	95	85	20	–
GT2–GB1	Blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls). Water-bearing fractures occur, especially in MDZ <70°.	5	5	40	100
GT2–GB3B	Blocky rock mass with possible water assisted block falls, especially in fractures with soft mineral filling.	–	10	–	–
GT3–GB1	Sealed fracture network. If reactivated it may result in blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls).	–	–	40	–
		100	100	100	100
Additional combinations in the most unfavourable case					
GT1–GB2A	Sparsely fractured, isotropic rock with possible spalling.	All underground openings with a longitudinal direction deviating by more than 30° from the direction of σ_H.			
GT2–GB2A	Blocky rock mass with possible spalling.				
GT3–GB2A	Sealed fracture network. If reactivated it may result in blocky rock mass with possible spalling.				

Table 6-3. Expected distribution of Ground Behaviour expressed as underground opening length (m) in each of the functional areas. The distribution for the "unlikely maximum stress" model is also given as the "Most Unfavourable distribution".

Functional Area	Most Likely stress model		Unlikely Maximum stress model		
	GB1 [m]	GB3B [m]	GB1 [m]	GB2A [m]	GB3B [m]
Repository Access					
Ramp	5,012	46	5,072	651	69
Niches (ramp – shafts)	187	–	187	–	–
Fresh air Ventilation shafts	1,167	4	1,162	–	13
Tunnel (pressure chamber above the Central area)	1,008	–	494	514	–
Central Area					
Caverns (sub-parallel to major horizontal stress)	604	–	561	43	–
Tunnels (primarily sub-perpendicular to major horizontal stress)	1,623	–	542	1,081	–
Silo (ø 9.5 m)	22	–	22	–	–
Deposition Area					
Exhaust Ventilation shafts SA01 and SA02	908	27	895	–	40
Main tunnel (10.0 m wide)	6,473	–	220	6,253	–
Transport tunnel (7.0 m wide)	4,621	–	3,505	1,116	–
Deposition tunnel (4.2 m wide)	61,189	–	61,189	–	–

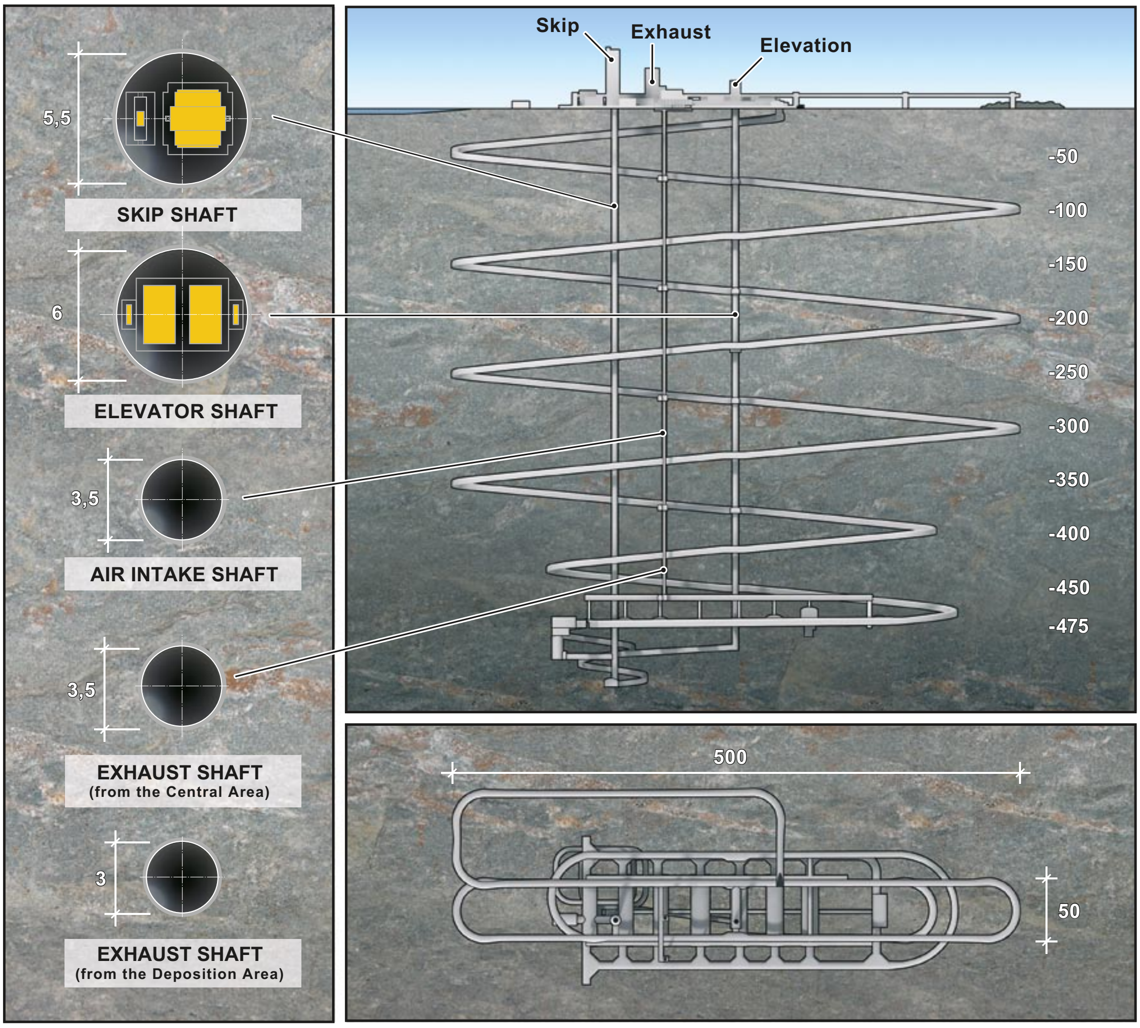


Figure 6-2. Vertical cross-section through the ramp and shaft excavations used for the repository access. The ramp will be excavated at an approximate grade of 10% (1:10).

6.1.2 Central Area

The distribution of the expected Ground Behaviour (GB) for the Central Area is given in Table 6-3 /Eriksson et al. 2008/, with the same assumption as for the Repository Accesses. The analyses indicated that the left side spring line of cavern B in Figure 6-3 was always subjected to the highest elastic stress concentrations. For the cases with the caverns aligned 0 degree to 30 degrees relative to the major horizontal stress, the stress concentrations were 47 MPa and 65 MPa respectively. The results indicate that these stress concentrations are well below the estimated spalling strength of 114 MPa (RFM029) and 196 MPa (RFM045).

Three-dimensional analyses were not carried out for this reference design, but underground construction experience suggests that the cavern intersections may require additional local reinforcement. This additional reinforcement has been factored into the support estimates. Three-dimensional analyses should be used during the Detailed Design to check these support estimates when the detailed layout of the Central Area has been established.

6.1.3 Deposition area

The low frequency of open fractures at the depth of the repository (0.005/m) suggests that the most likely Ground Behaviour that will be encountered during construction of the tunnels (Main, Transport, Deposition) in the Deposition Area will be spalling, 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 to assess the magnitude of the tangential stress concentrations. The geometry of the tunnels used in the various three dimensional analyses are given in Figure 6-4 and Figure 6-5. For each geometry, the orientation of the deposition tunnel relative to the orientation of the maximum horizontal stress was varied from 0 to 90 degrees in increments of 30 degrees. The calculated maximum elastic tangential stress concentrations ranged in magnitude from 75 to 102 MPa. These stress concentrations are below the estimated spalling strength of 114 MPa for RFM029 and 196 MPa for RFM 045 (Table 2-4 in SKB 2008a). 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. 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 lifting the level for the spring line 0.2 and 0.4 m. They concluded that a flatter roof profile increases the tangential stress stresses on the spring line area while reducing the tangential stresses in the roof. Such a profile change could be applied if spalling was encountered in the roof during construction.

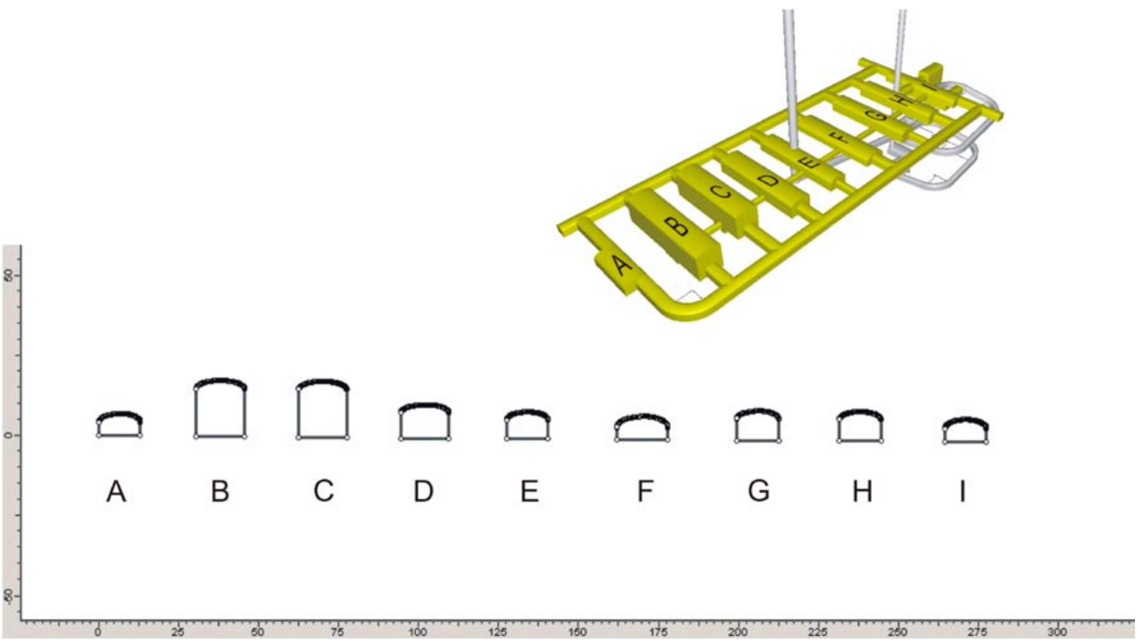


Figure 6-3. Layout of the Central Area and the two-dimensional sections (scale in metres) of the caverns used in the calculations.

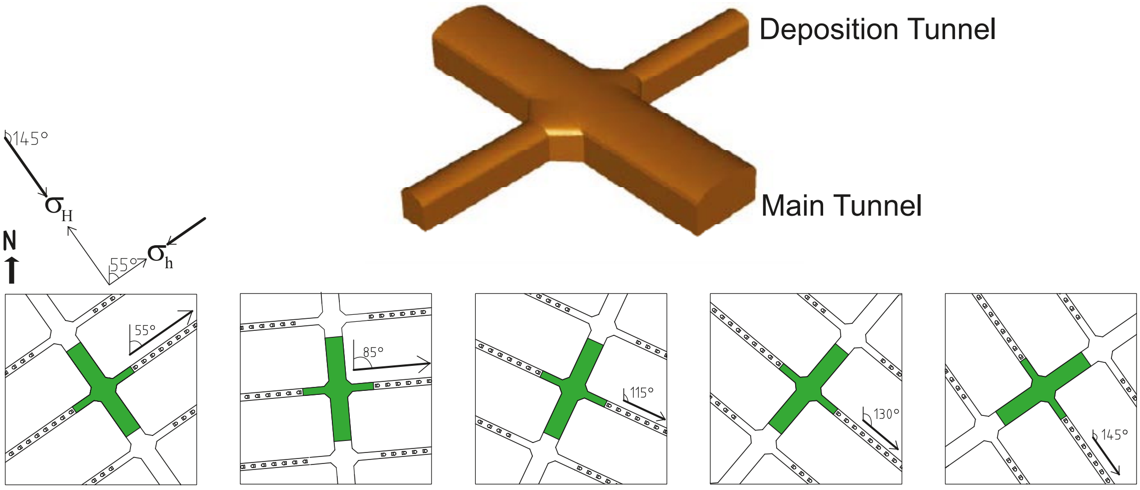


Figure 6-4. Illustration of different 3D cases with orthogonal main tunnel – deposition tunnel crossing.

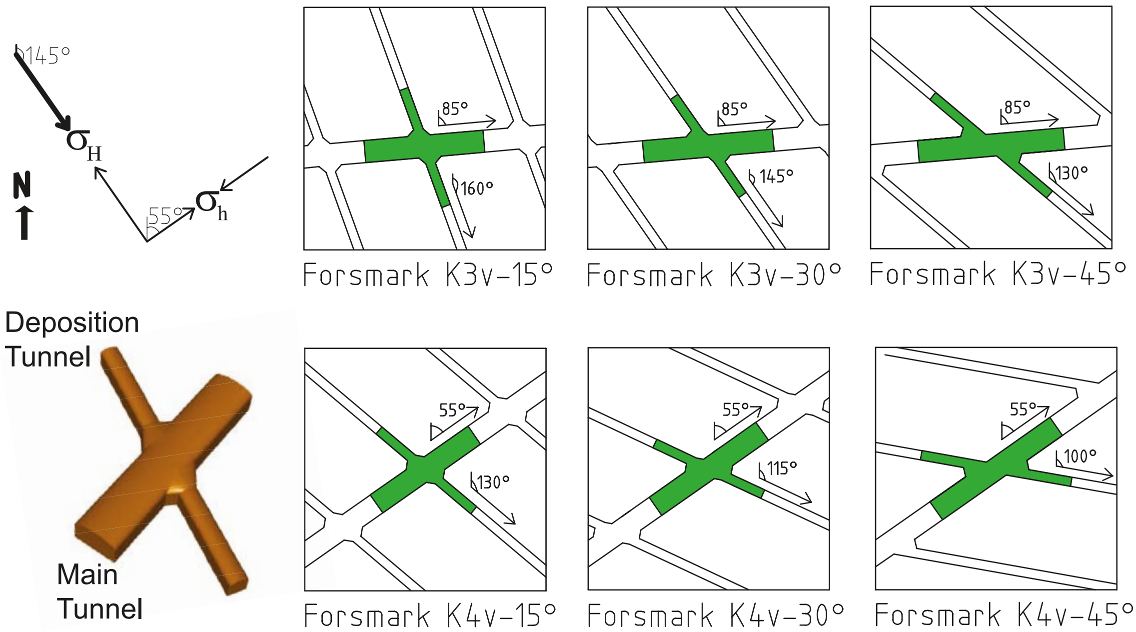


Figure 6-5. Illustration of different 3D cases with skewed main tunnel – deposition tunnel crossing.

/Eriksson et al. 2008/ used the results from the three dimensional stress analyses, to estimate the distribution of the Ground Behaviour for the tunnels and exhaust ventilation shafts in the Deposition Area (Table 6-3).

The stress concentrations around deposition holes were also evaluated using three-dimensional elastic analyses (Figure 6-6). The impact of the tunnel orientation relative to the orientation of the maximum horizontal stress is shown in Figure 6-7. 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 6-7). The effect of stress induced spalling on the deposition tunnels due to the thermally-induced stresses will be analysed in SR-Site.

Based on the stress analyses presented in this section, the orientation of the deposition tunnels relative to the orientation of the maximum horizontal stress shall range between 0 and 30 degrees. This orientation also reduces the stress concentrations on the deposition holes and fulfils the recommendations given in the SER /SKB 2008a/. This implies that the orientation of the maximum horizontal stress must be known prior to developing the Deposition Area.

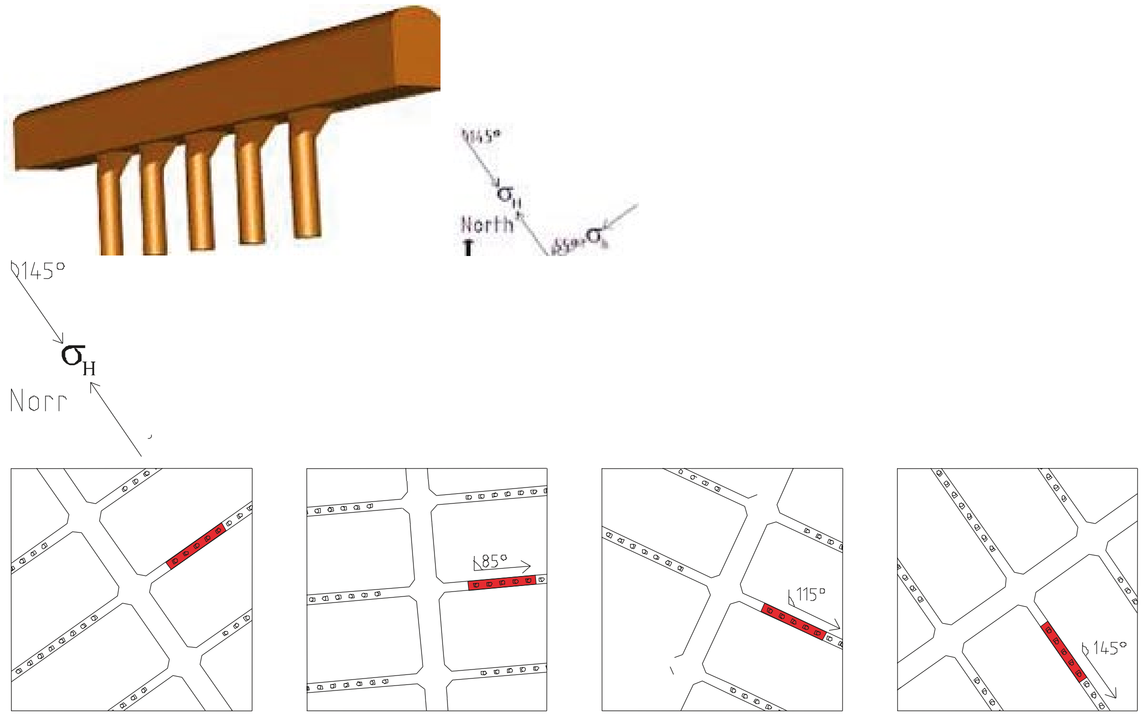


Figure 6-6. Illustration of different cases for study of stress concentration around a deposition hole.

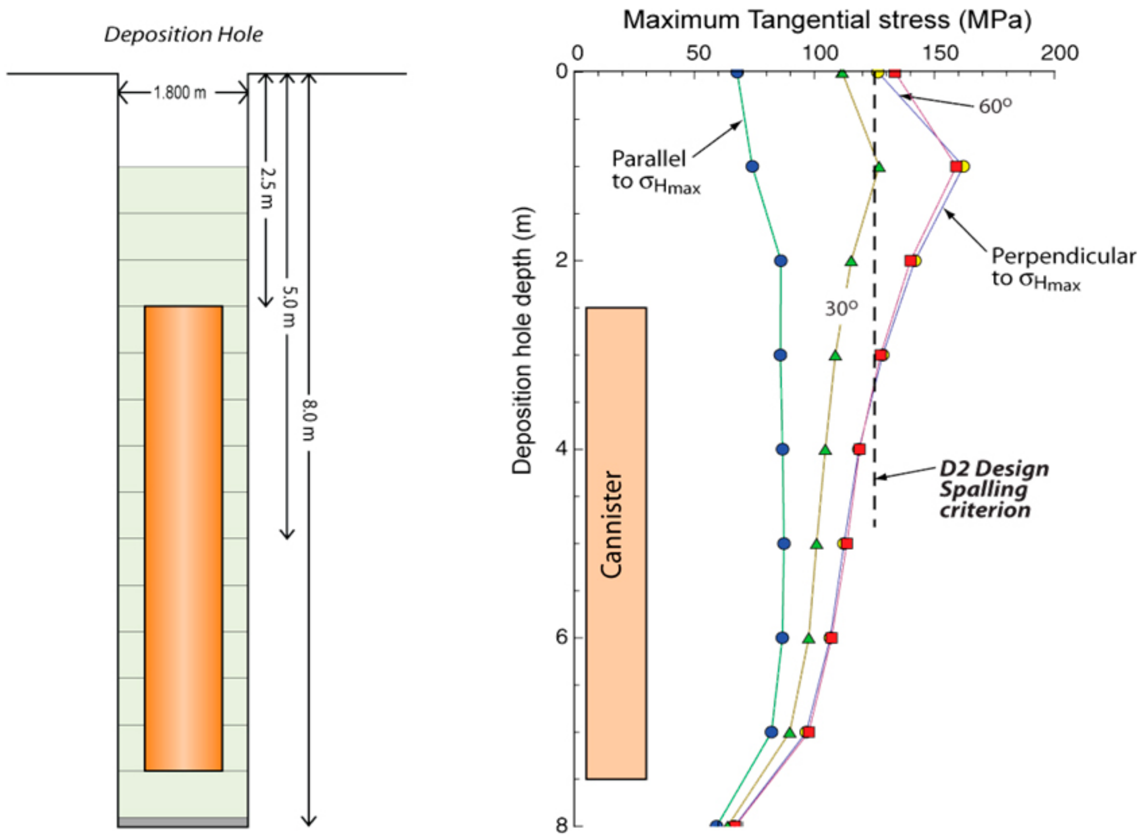


Figure 6-7. Maximum tangential stress from three-dimensional elastic analyses versus deposition hole depth for the four stress scenarios shown in Figure 6-4. The results suggest that when the deposition tunnels are oriented between 0 deg and 30 dg relative to the orientation of the maximum horizontal stress, the stress concentrations are below the spalling strength used for this design.

6.2 Support measures

The system behaviour was assessed by /Eriksson et al. 2008/ using analytical methods, rock characterisation methods (the Q system), numerical methods, and comparative studies, based on experience from previous underground projects in the Forsmark area. An overview of the support types for each Ground Type and Ground Behaviour proposed by/Eriksson et al. 2008/ is summarised in Table 6-4.

The expectations of GT1, with a Q-value above 100, are that the rock has very few fractures and that reinforcement of blocks will therefore only be needed locally. However, there may well be local occurrences of weaker, blockier rocks and for those situations systematic bolting may be needed in order to maintain a load-bearing arch. It is likely that this will occur in FFM02 in locations were high frequency of gently dipping water bearing fractures may occur. Experience suggests that it should be possible to take care of smaller blocks in the roof and spring line with shotcrete, while reinforcement of larger blocks may be supplemented with selective bolting.

/Eriksson et al. 2008/ concluded that the parts of the installation classified as GT1, under the expected stress conditions may be treated with ST1. For other combinations of rock classes and ground behaviour, for the most likely stress conditions (i.e. GT2–GB1, GT2–GB3B, GT3–GB1), it is suggested that ST2 will be a suitable support type. Under unfavourable rock conditions in combination with large spans, it should be possible to use ST3 for parts of the excavation classified as GT2 and GT3.

The results from the various analyses used by /Eriksson et al. 2008/, all indicate that conventional underground support measures would be sufficient to ensure that the performance of the underground openings was acceptable. The estimated amount of support is given in Table 6-5. The estimated amount of support is on average very low because of the very good quality rock mass anticipated.

The analyses of the ground behaviour indicate that stable openings can be readily achieved using traditional tunnel support systems. It is anticipated that the openings for the Repository Access in FFM02 to a depth of 150 m may require systematic support while the openings below 150 m depth will be supported with minimal support. 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 6-6 contains the estimated quantities of the material used for ground support in each function area for this design step.

Table 6-4. Proposed support types (ST) for the expected different Ground Behaviour in the different Ground Types.

Support type	Description	Ground types	Ground behaviour
ST1	Fibre-reinforced shotcrete 30 mm in roof + uppermost 1 m of walls. Spot bolting: 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. Spot bolting: 1 bolt/50 m^2 in walls (ø25 mm, length 3 m). Systematic bolting: c/c 2 m in roof (ø25 mm, length 3 m).	GT2, GT3	GB1, GB2A, GB3B
ST3	Fibre-reinforced shotcrete 75 mm in roof + uppermost 1 m of walls. Spot bolting: 1 bolt/50 m^2 in walls (ø 25 mm, length 3 m). Systematic bolting: c/c 1 m in roof (ø 25 mm, length 3 m).	GT4	GB1, GB2A, GB3B
ST4	Concrete or shotcrete lining	GT4	GB2B, GB3A
ST Deposition	Wire mesh in roof + uppermost 1 m of walls. 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. Spot bolting: 1 bolt / 50 m^2 in walls (ø 25 mm, length 3 m). Systematic bolting: c/c 2 m in roof (ø 25 mm, length 3 m).	GT1, GT2, GT3	GB1, GB2A

Table 6-5. Compilation of reinforcement amounts for different facility parts of the repository assuming the most-likely stress model.

Facility part	No of bolts	Average no of bolts/m	Quantity of shotcrete [m³]	Quantity of wire mesh [m²]
Repository Access: Ramp				
Tunnel (5.5 m wide)	1,762	0.43	1,144	–
Tunnel (6.0 m wide)	46	0.39	34	–
Tunnel (7.0 m wide)	315	0.45	224	–
Passing places (8.0 m wide)	71	0.53	48	–
Niche (5.5 m wide)	7	0.35	6	–
Niche (7.0 m wide)	53	0.41	42	–
Niche (10.0 m wide with 5x5x16 m below)	23	0.61	16	–
Repository Access: Ventilation				
Shaft (ø 1.5 m)	24	0.09	38	–
Shaft (ø 2.5 m)	70	0.16	118	–
Shaft (ø 3.5 m)	98	0.22	165	–
Shaft (ø 4.5 m)	7	0.28	11	–
Tunnel (4.0 m wide)	372	0.37	231	–
Central area				
Skip shaft (ø 5.0 m)	165	0.31	257	–
Elevator shaft (ø 6.0 m)	186	0.38	311	–
Silo (ø 9.5 m)	13	0.59	20	–
Tunnel (3.0 m wide)	37	0.28	25	–
Tunnel (4.0 m wide)	8	0.28	6	–
Service tunnel (4.0 m wide)	146	0.29	109	–
Tunnel (5.1 m wide)	15	0.31	13	–
Tunnel (7.0 m wide)[2]	634	0.70	317	–
Caverns (13.0–16.0 m wide)	2,347	4.34	487	–
Sump (12.0 m wide)	74	3.70	16	–
Electricity hall (7.0 m wide)	48	2.29	11	–
Crushing hall (10.3 m wide)	71	3.23	15	–
Deposition area				
Ventilation shafts SA01 and SA02 (ø 3.0 m)	176	0.19	0	–
Main tunnel (10.0 m wide)	7,660	1.18	3,108	–
Transport tunnel (7.0 m wide)	2,936	0.63	1,601	–
Deposition tunnel (4.2 m wide)	15,175	0.25	0	57,629
Total	**32,539**	**–**	**8,373**	**57,629**

Table 6-6. 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 Bolts							
Rock bolts (l=3 m, d=25 mm)	4	27		52		182	
Wire mesh (1,7 kg/m²)						96	
Fixing bolts (29,329 pcs)						28.2	
Rock Bolt Grout							
Cement	340	15	7	28	13	98	47
Silica	226.7	10	5	19	9	65	31
Water	266.6	12	12	22	22	77	77
Glennium 51	4	0.2	0.2	0.3	0.3	1	1
Quarts filler	1,324	57	29	109	54	381	191
Shotcrete							
Water	158	239	239	340	340	744	744
Ordinary Portland cement CEM I 42.5	210	318	151	452	215	989	471
Silica fume	140	212	101	301	143	659	314
Coarse aggregate (5–11)	552	836	492	1,187	698	2,600	1,529
Natural sand (0–5)	1,025	1,552	913	2,205	1,297	4,227	2,839
Quarts filler (0–0,25) or Limestone filler (0–0,5)	250	379	189	538	269	1,177	589
Superplasticiser “Glennium 51” from Degussa	3	4.5	4.5	6.5	6.5	14	14
Air entraining agent “Sika AER S”	2.5	3.8	3.8	5.4	5.4	12	12
Accelerator “Sigunit” from Sika or AF 2000 from Rescon	7%[1]	0.1	0.1	0.2	0.2	0.3	0.3

1) Tests performed have given values between 4 and 10%. An average value of 7% was however chosen for these calculations.

6.3 Summary

While there is little uncertainty in the ground behaviour at shallow depths i.e. <150 m, there is some uncertainty in the ground behaviour in the more competent rock below 150 m depth. This uncertainty is related to the uncertainty in stress magnitudes and how they increase with depth.

The deposition tunnels shall be aligned in a small angle to the maximum horizontal stress to minimise the risk for stress induced spalling in deposition holes.

7 Groundwater control and grouting

The SDM Site states that the groundwater system in the bedrock at Forsmark can be divided into two hydraulic flow domains: (1) flow controlled by the characteristics of the deformation zones and (2) flow controlled by the connected fracture network through the rock mass. The excavations required for the Final Repository will start at the ground surface and penetrate both of these hydraulic domains. 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.

Inflow quantities and drawdown estimates were carried out using the hydraulic characteristics of the rock mass and deformation zones provided in the SER /SKB 2008a/. An assessment of the grouting measures required to control the inflows to the specified levels was carried out using the grouting technology specified in UDP/D2 and was restricted to the application of a low-PH grout recipe using proven grouting technology.

7.1 Inflow estimates

The near-surface bedrock down to about 50 to 150 m is heterogeneously intersected by horizontal sheet joints and gently dipping fractures referred to fracture domain FFM02. The fractures in this domain are highly transmissive over long distances, ranging from 10^{-6} to 10^{-3} m^2/s, in the uppermost parts of the bedrock.

The rock mass below 150 m (fracture domain FFM01 and FFM06) has a low frequency of flowing fractures. Between levels 200–400 m the linear frequency of flowing fractures is about 0.05 m^{-1} and the rock mass has an "average conductivity" in the order of $5.2 \cdot 10^{-10}$ m/s. Below approximately 400 m depth, the observed frequency of flowing features is 0.005 m^{-1} and the rock mass "average conductivity" in the order of $6.3 \cdot 10^{-11}$ m/s. This suggests that the rock mass between the deformation zones at the repository level approaches the permeability of intact rock although the possibility in-frequent occurrence of low transmissive joints cannot be excluded /SKB 2008a/.

According to /Bergman and Nord 1982/ the water inflow into a circular tunnel at a given depth 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)

ξ = skin factor inside seal (dimensionless)

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

If a deformation zone is evaluated, hydraulic conductivity can be replace by transmissivity (T, m^2/s) T where T= $K \cdot$ 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. Because of the heterogeneous hydraulic nature of the Forsmark bedrock, the inflow predicted using Equation 7-1 could range significantly. Table 7-1 presents the estimated inflow based on the "average conductivity" provided in the SER /SKB 2008a/. A range was assessed; i.e. minimum, typical and maximum inflow for FFM02 and deformation zones where the fracture transmissivity are more heterogeneous.

Table 7-1. Estimated water inflow to various excavations before grouting using Equation 7-1 /Brantberger and Janson 2008/.

Ramp (depth 0–470 m)	**Inflow per 100 m, (litre/min)**		
FFM02 (0–50 m)	Min.: 4	Typical: 200	Max.: 3,900
FFM02 (50–100 m)	Min.: 10	Typical: 480	Max.: 9,600
FFM01 (100–200 m)	120		
FFM01 (200–400 m)	0.8		
FFM01 (400–470 m)	0.1		
	Inflow per zone, (litre/min)		
Steep zone (200–400 m)	Min.: ~0	Typical: 0,2	Max.: 15
Steep zone (400–470 m)	Min.: ~0	Typical: 0,2	Max.: 21
Shaft (depth 0–470 m)	**Inflow per 100 m, (litre/min)**		
FFM02 (0–50 m)	Min.: 2	Typical: 100	Max.: 2,100
FFM02 (50–100 m)	Min.: 6	Typical: 310	Max.: 6,200
FFM01/FFM06 (100–200 m)	62		
FFM01/FFM06 (200–400 m)	0.6		
FFM01/FFM06 (400–470 m)	0.1		
	Inflow per zone, (litre/min)		
Steep zone (200–400 m)	Min.: ~0	Typical: 0,1	Max.: 12
Rock cavern (depth 470 m)	**Inflow per 100 m, (litre/min)**		
FFM01	0.2		
	Inflow per zone, (litre/min)		
Steeply dipping zone	Min.: ~0	Typical: 0,3	Max.: 26
Deposition tunnel (depth 470 m)	**Inflow per 100 m, (litre/min)**		
FFM01/FFM06	0.1		
	Inflow per zone, (litre/min)		
Steep zone	Min.: ~0	Typical: 0,2	Max: 22
Transport & main tunnel (depth 470 m)	**Inflow per 100 m, (litre/min)**		
FFM01/FFM06	0.1		
	Inflow per zone, (litre/min)		
Transport tunnel: Steep zone	Min.: ~0	Typical: 0.2	Max: 23
Transport tunnel: Steep zone (ENE0060A)	0.7		
Main tunnel: Steep zone	Min.: ~0	Typical: 1,2	Max: 120
Exhaust shaft SA01 (0–470 m)	**Inflow per 100 m, (litre/min)**		
FFM02 (depth 0–50 m)	Min.: 2	Typical: 100	Max.: 2,000
FFM02 (depth 50–100 m)	Min.: 6	Typical: 300	Max:. 6,000
FFM01 (depth 100–200 m)	60		
FFM01 (depth 200–300 m)	0.5		
FFM01 (depth 330–400 m)	0.7		
FFM01 (depth 400–470 m)	0.1		
	Inflow per zone, (litre/min)		
Gently dipping zone B7 (depth 300 –330 m)	6,3		
Exhaust shaft SA02 (0–470 m)	**Inflow per 100 m, (litre/min)**		
FFM02 (depth 0–50 m)	Min.: 2	Typical: 100	max.: 2,000
FFM02 (depth 50–100 m)	Min.: 6	Typical: 300	max:. 6,000
FFM02 (depth 100–170 m)	22		
FFM01 (depth 170–200 m)	74		
FFM01 (depth 200–400 m)	0,6		
FFM01 (depth 400–470 m)	0.1		

The acceptable inflow criteria into the repository is currently 5 l/min for deposition tunnel length, and for all other openings 10 l/min per 100 m tunnel length /SKB 2007/. As shown in Table 7-1 the estimated inflows to some areas of the facility will exceed this inflow criterion. Consequently grouting measures, particularly in FFM02, may be required to reduce the leakage into some functional areas to acceptable levels. However, for tunnels at repository level located in fracture domain FFM01 minor grouting is expected as very few flowing fractures will be encountered.

Estimates of the inflow to the different parts of the facility after grouting have been computed utilising Equation 7-1, and the numerical simulation models MIKE SHE /Gustafsson et al. 2009/ and Darcy Tools /Svensson and Follin 2009/. Both numerical models provide inflows into the repository for three different development stages (Scenario A after ~15 years, B after ~30 years and C after ~45 years) using three different conductivities for the grouted zone (10^{-7}, 10^{-8} and 10^{-9} m/s). The total inflow for the simulation models slowly increases in time due to the development of the repository. A summary of the estimated total inflows to the various functional area predicted using all three calculation methods is given in Table 7-2.

Estimating inflows to tunnels is always challenging and the predicted range in the total inflows in Table 7-2 is large. The models used for estimating the inflows in design step D2 are well known and accepted, but required considerable simplification. Nonetheless, the estimates are considered adequate for this design step and fall within the ranges measured at existing underground facilities. For example the total inflow to the Forsmark SFR facility was approximately 720 l/min in 1988, decreasing to 320 l/min in 2005 /Carlsson and Christiansson 2007a/ with the lower ungrouted construction tunnel with a length of 900 m contributing 60 l/min (6.6 l/min per 100 m length). At Äspö HRL, the inflows in 1995 were 2,479 l/min decreasing to 1,100 l/min by 2005 /Carlsson and Christiansson 2007b/. While none of theses measurements are directly comparable to the repository facility, they are comparable to portion of the ramp access and it appears that the estimated inflows are in keeping with construction experience. More importantly, according to Table 7-1 the predicted inflows before grouting demonstrate that at the Forsmark site the expected inflows in the 300-m-long deposition tunnels will be lower than the allowable inflow criterion of 5 l/deposition tunnel by a factor of approximately 16.

Table 7-2. Calculated inflow to various parts of the repository facility after grouting (scenario after 45 years).

	Darcy Tools model, calculated inflow [litre/min]			MIKE SHE model, calculated inflow [litre/min]			Analytical, calculated inflow [litre/min] min/type/max
	Grouted zone $K_g = 10^{-7}$	Grouted zone $K_g = 10^{-8}$	Grouted zone $Kg = 10^{-9}$	Grouted zone $K_g = 10^{-7}$	Grouted zone $K_g = 10^{-8}$	Grouted zone $K_g = 10^{-9}$	Grouted zone $K_g = 10^{-8}$ to 10^{-9}
Ramp	1,000	350	100	The model is partitioned in layers, and below only the total inflow to the repository is reported			10/190/8,900
Deposition tunnels	550	500	250				60/210/6,700
Main tunnels	1,000	650	200				10/80/9,000
Other areas	500	200	~0				–/280/–
Total	3,050	1,700	550	2,150	1,300	600	–/760/–

7.2 Grouting strategy

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 2008/, however, these general guidelines may need to be revised during the detailed design:

- Prior to starting construction of the ramp and shafts to the Central Area, a pilot drilling and grouting programme may be needed to establish the groutability of the gently-dipping fractures in FFM02. Such grouting trials are required to develop the detailed grouting procedures needed to meet the inflow criterion.
- A cut-off grout curtain, may be required, from surface level around all ramps and shafts to a depth of between depth 50 and 100 m. The aim of the grout curtain is to cut-off the superficial large shallow fractures in order to enable a more effective and safe rock excavation. Results from the test drilling and grouting trials will be utilised to develop a detailed grout plan.
- Niches in the ramp are to be used for grouting in stretches about 100 m long around the drilled shafts (lift and ventilation shafts in the Central Area).
- The skip shaft will be grouted from the shaft bottom as part of the excavation cycle. Some of the curtain grouting holes may need to extend below 100 m depth, to create dryer conditions for the shaft sinking. Less time will then be needed for grouting from the bottom of the shaft.
- If grouting is required at the repository level, it is anticipated that cement based grouting will be adequate to achieve the required sealing efficiency. In some situations the transmissivity of the fracture could be so low that reaching the required sealing efficiency may not be practical with cement based grouts. To achieve the sealing new technologies, such as those that recently tested at the Äspö HRL /Funehag 2008/ may be required.
- Grouting of different underground openings under the depth 200 m can be carried out as a selective pre-grouting, with probe hole investigations, when passing deformation zones and where discrete water-bearing fractures are encountered.
- Individual fractures should if possible be identified and pre-grouted in deposition tunnels in order to avoid post-grouting of point leakage. Due to the difficulty in identifying individual water bearing fractures, potential remaining point leakages >0.1 l/min in deposition tunnels will mainly be sealed by post-grouting.

7.2.1 Accesses, Central Area and Deposition Areas

Ramp

Grouting of the ramp will be required in fracture domain FFM02. At this stage of the design the boundary of the domain varies from Elevation –50 to Elevation –150. At the proposed location of the ramp, the depth of FFM02 is expected to be less than 100 m. However, because of the uncertainty in the lower boundary of FFM02, for this design step, it is assumed that continuous grouting of the ramp will be required down to Elevation –200 m. Excavation and grouting of the ramp in an efficient manner requires well-planned and implemented probe drilling. The number of probing holes, length and direction must be adapted to facilitate early identification of the superficial, water-bearing, sub-horizontal zones.

The grouting fans, including the tunnel-front holes, shall cross the gently dipping and water-bearing zones as far as possible from the tunnel front (Figure 7-1). It is anticipated that hole spacing, drilling angle and fan length may vary between individual grouting fans, as the site conditions require.

Shafts

As with grouting in the ramp, the main grouting in the shafts will be made down to a depth of 150 m. The shafts will be excavated using two different methods, shaft sinking (skip shaft) and raise-drilling technique (lift and ventilation shafts).

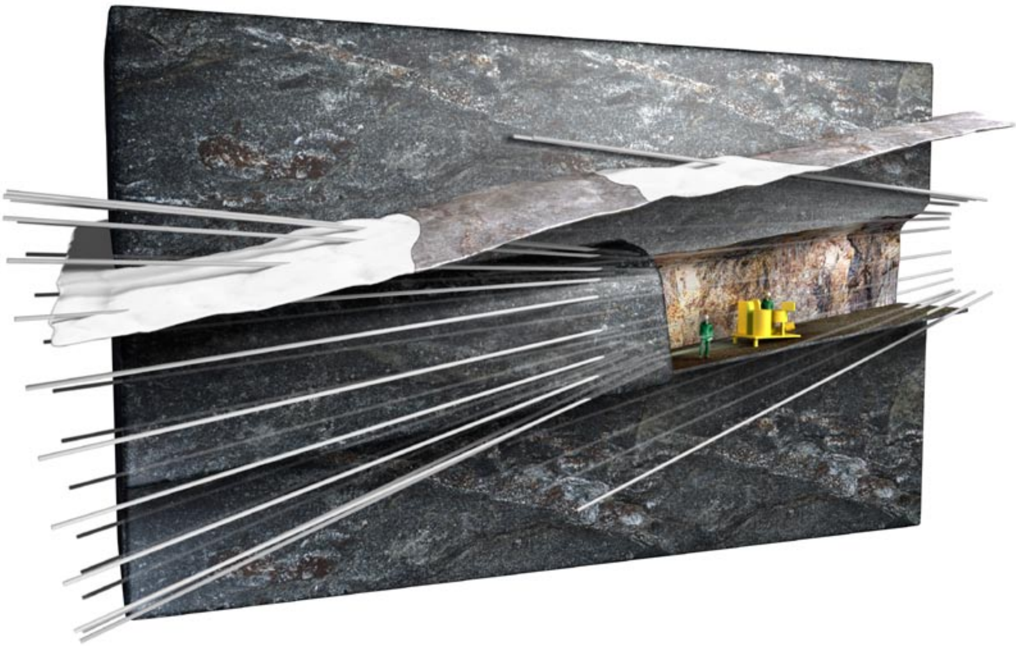


Figure 7-1. An illustration of a grouting fan in the ramp, where the holes are angled as much as possible to cross the gently dipping water-bearing fracture zones.

With regard to the uncertainty concerning fulfilment of requirements on inflow in the drilled shafts in the Deposition Area, 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. The principles of tunnel grouting apply, although the grouting fans are drilled vertically instead of horizontally; see Figure 7-2. This type of grouting is sometimes denoted as "cover grouting". Furthermore, it is suggested that some of the holes in the cut-off curtain grouting are extended in the sink shaft down to 150 m to reduce the risk of large inflows during the shaft sinking. The water-bearing zones are dominantly gently dipping, which is advantageous for the grouting efficiency.

Lift and ventilation shafts through the Central Area

The grouting of these shafts will be carried out before starting the raise drilling. The grouting is carried out in long vertical boreholes, which are drilled in a ring outside the contour of the shafts. Figure 7-3 presents the principle for grouting the lift and ventilation shafts in the Central Area.

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.

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.

Central area

The unique geometries in the Central Area compared to other functional areas are the large rock caverns. The size of the rock caverns, ranges from about 104 to 255 m^2 in cross-sectional area. If grouting is required, the rock caverns will be grouted using either selective grouting or systematic grouting following the general grouting guidelines outlined above.

Figure 7-2. *Illustration of a possible grout-hole configuration for the 6-m-diameter skip shaft intersecting the gently dipping water bearing zones in FFM02.*

Figure 7-3. *Illustration of a possible pre-grouting borehole configuration for the 6-m-diameter liftshaft and 3-m-diameter ventilation shafts.*

Deposition area

Deposition tunnels

Depending on the prevailing ground conditions, different types of grouting may be required. Grouting is anticipated when passing through deformation zones (Figure 7-4). Between zones inflows lower than the allowable criterion of 5 l/deposition tunnel are expected. The cumulative density function of transmissivities values in 20-m-long sections indicates that on average, less than 2% of the 20 m sections between deformation zones will require grouting /SKB 2008a/. For those grouting fans the bottom holes will be drilled inside the tunnel contour to prevent grouting holes from intersecting potential deposition-hole positions. Grouting fans for deformation zones are not constrained since deposition are not allowed in any deformation zone. Decision to grout will be based on probing inside the tunnel perimeter. In some situations the transmissivity of the fracture could be so low that reaching the required sealing efficiency may not be practical with cement based grouts. To achieve the sealing new technologies, such as those that recently tested at the Äspö HRL /Funehag 2008/ may be required.

Exhaust shafts in the deposition area

The exhaust shafts in the Deposition Area will be carried out using a raise-boring technique, similar to the technique used for lift and ventilation shafts in the Central Area. Grouting in these shafts, if required, will be carried out before raise drilling begins. This grouting would require long boreholes with minimum deviation. Boreholes could be drilled from the surface and from underground. The detailed requirements and working plan will need to be developed during the detailed design.

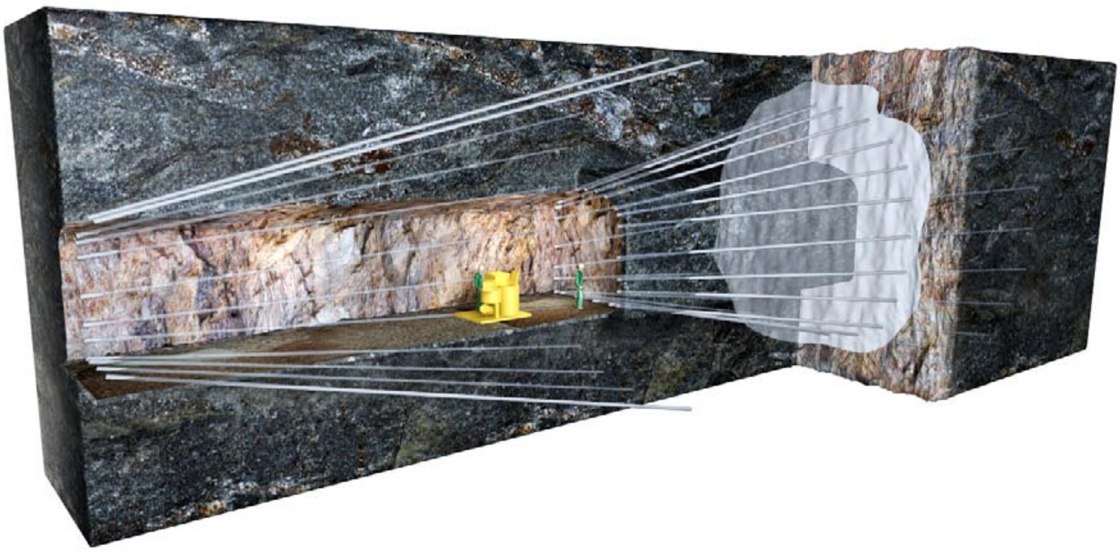


Figure 7-4. *Illustration of possible grout fan used to seal steeply-dipping deformation zones with water bearing fractures encountered in the Deposition Area.*

7.2.2 Intersection with deformation zones

One of the characteristics of the Forsmark site is the very low frequency of flowing fractures within the sub-vertical deformation zones below a depth of 150 m /SKB 2008a/. Hence the grouting requirements for passing these deformation zones is likely to require selective and localised pre-grouting rather than systematic pre-grouting.

Passing deformation zones on the repository level it is anticipated that cement based grouting will be adequate to achieve the required sealing efficiency. In some situations with high demands on the required sealing efficiency it may not be practical with cement based grouts. To achieve the sealing new technologies, such as those that recently tested at the Äspö HRL /Funehag 2008/ may be required. The exact procedure to achieve the low hydraulic conductivity values will depend on the characteristics of the flowing fractures, e.g. channelized flow, planar flow, etc. Application of the Observational Method will be used to develop the grouting methodology required to meet the inflow criterion.

7.3 Estimated amounts of grouting material

Based on calculations made, Table 7-3 summarises the assessed amount of grouting material for different functional areas. The quantities presented refer to the total amount of grout including tunnel-front grouting, curtain grouting and post-grouting. Hole filling is not included in the calculation. 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 for grouting of large fractures, which is anticipated mainly at depths between 0–100 m in FFM02. For less permeable rock the amount of grout is judged to be smaller.
- *Stop grout* is anticipated for grouting, e.g. a first grouting round in rock mass of high hydraulic conductivity.
- *Injection grout* is the grout that is used primarily, in rock mass of low hydraulic conductivity.
- *Silica sol* represents the assessed need for new grouting technologies, such as those that recently tested at the Äspö HRL /Funehag 2008/. This is expected in rock conditions, where the sealing efficiency of cement may not be practical, or to be sufficient to satisfy tightness requirements. Assessed quantities for post-grouting were also included here.

Based on the assessed proportion of the different grouts, the amount of sub-material included can be calculated based on recipes (for details on recipes, ref. to /Brantberger and Janson 2008/) of the individual grouts. In *Table 7-4*, the quantities remaining in the rock after excavation have been converted to weight, depending on the recipe. Based on general experiences, the calculated amounts are considered reasonable, see /Brantberger and Janson 2008/.

Table 7-3 gives the amount of grout in m^3 which in *Table 7-4* (for the three defined functional areas) is converted to kg cement or bonding agent, depending on the recipe. Based on experiences from projects, the calculated amounts are considered reasonable, see /Brantberger and Janson 2008/.

In *Table 7-4* it can be seen that large amounts of grout can be anticipated in the ramp and the shafts in the upper 200 metres. The difference between estimated maximum and minimum amounts is however considerable, which reflects the uncertainty about details like porosity and models for calculating the grout take. This implies that test grouting should be carried out in a preliminary phase and an updating of grouting measures and estimations of amounts should be done as the tunnel excavation and grouting progresses. As mentioned earlier, the anticipated grouting at depth 0–100 m will be very extensive and therefore time-consuming.

Table 7-3. Summary of total amounts of pre-grout injected before blasting for different functional areas. The dash (–) means that grouting is not possible except for individual fractures and that the amount of grout is small /Brantberger and Janson 2008/.

Functional areas/ underground openings	Drilling, number/drilled metre (no./m)	Volume of grout Min./type/ max. (m^3)	Proportion plug grout/ stop grout/injection grout/ silica sol (%)
Accesses (0 to –200 m)			
Ramp	7500/150,000	400–1,590 (K_{min})	20/10/50/20
	Curtain grouting: 240/12,000	650–2,570 (K_{typ})	20/30/40/10
		980–3,910 (K_{max})	30/50/10/10
Shaft (4 shafts)	160/32,000	130–520 (K_{min})	20/10/50/20
	Curtain grouting: 60/6,000	220–860 (K_{typ})	20/30/40/10
		330–1,310 (K_{max})	30/50/10/10
Central area (–470 m)			
Rock caverns (grouting in deformation zones)	300/6,000	– (K_{min})	–
		20–60 (K_{typ})	10/10/50/30
		30–140 (K_{max})	10/20/50/20
Deposition area (–470 m)			
Deposition tunnels (grouting in deformation zones, with silica sol concept)	11800/235,800	– (K_{min})	–
		270–1,090 (K_{typ})	10/20/20/50
		600–2,380 (K_{max})	10/20/20/50
Transport tunnels (grouting in deformation zones, including ZFMENE0060A)	960/19,200	– (K_{min})	–
		35–140 (K_{typ})	10/20/50/20
		80–280 (K_{max})	10/20/50/20
Main tunnels (grouting in deformation zones)	4400/88,000	– (K_{min})	–
		170–660 (K_{typ})	10/20/50/20
		360–1,440 (K_{max})	10/20/50/20
Exhaust shaft SA01 (including ZFMB7)	Curtain grouting: 10/3,000	30–130 (K_{min})	–/10/70/20
		50–210 (K_{typ})	10/40/30/10
		80–330 (K_{max})	10/50/10/10
Exhaust shaft SA02	Curtain grouting: 10/2,000	30–110 (K_{min})	–/10/60/30
		50–190 (K_{typ})	10/30/40/20
		70–290 (K_{max})	10/50/20/20

K_{min}, K_{typ} and K_{max} represent intervals in hydraulic characteristics according to SER /SKB 2008a/. Type values refer to the value that has been judged as most probable in the interval between maximum and minimum.

Table 7-4. Estimated quantities of grout materials and drilling that remain in the rock mass after excavation of the different underground openings.

Element	Material	Ramp/Shafts [ton] 1) 2) min	max	Central Area [ton] 1) min	max	Deposition Area [ton] 1) min	max
Cement grouting	Water	350	1,360	3	10	110	440
	Portland 3)	330	1,310	3	8	100	400
	Silica Fume 4)	460	1,790	4	11	140	550
	Super Plasticiser 5)	23	90	0.2	0.5	7	30
Chemical grouting	Silica	105	410	3	9	160	640
	NaCl solution	21	85	0.6	2	30	130
Volume of grout [m^3]		910	3,580	10	30	405	1,620
Drilling	Number of holes	7,980 pcs		300 pcs		17,160 pcs	
	Drilling meter	205,000 m		6,000 m		343,000 m	

1) Based on "type" hydraulic conditions (K_{typ}).

2) Incl. the Exhaust shafts SA01 and SA02.

3) Sulphate resistant Ordinary Portland cement with d_{95} on 16 µm, type Ultrafin 16 or equivalent.

4) Dispersed silica fume, microsilica with d_{90}=1 µm type Grout Aid 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.

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

7.4 Groundwater 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 those 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 areal extent and depth of these potential drawdowns. At this stage of the design there isn't sufficient geotechnical information to predict the nature of these site-specific drawdowns and this will have to be addressed in the final design. Experience from the constructions at the Forsmark SFR Facility and the Äspö HRL suggest that drawdowns in these types of rocks are not expected to be significant /Carlsson and Christiansson 2007ab/. The possible measures that may be used to reduce any potential drawdown to acceptable environmental limits are discussed in the following section.

7.5 Measures to reduce environmental impact of drawdown

7.5.1 Grouting

The grouting strategy outlined in Section 7.2 will be used to minimise the groundwater inflow and environmental impact.

7.5.2 Infiltration

The largest drawdown will occur close to the Central Area due to the ramp and shafts crossing the conductive fracture domain FFM02, and along some transmissive deformation zones. To compensate for the drawdown, if grouting would not be sufficient, infiltration would be the most feasible option.

In particular drainage of a few identified lime rich ponds located in the area are from environmental point of view considered not acceptable, and mitigation measures by arrangement of local infiltration to protect these habitats for the endangered frog specie "Gölgroda" *(Rana lessonae)* must be performed, (see Figure 7-5).

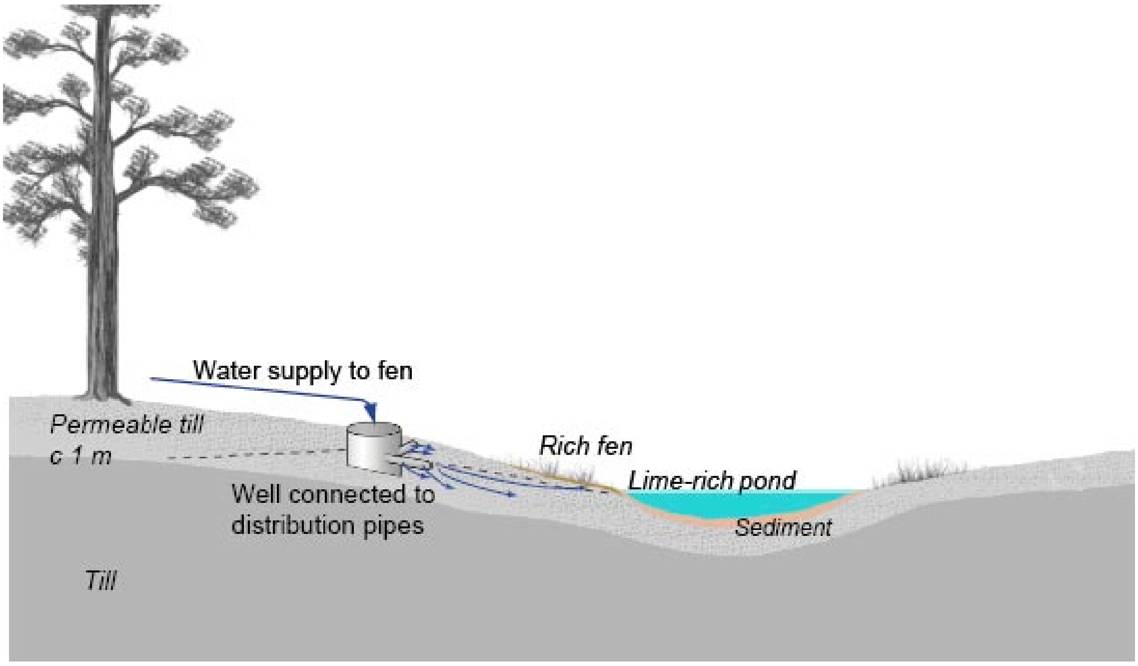


Figure 7-5. *Standard arrangement to maintain a local groundwater level at lime-rich ponds.*

Requirements and methodology for this type of infiltration is presently under investigation in co-operation with environmental experts, but the technology is considered well known and fairly robust, although maintenance must be considered over the lifetime of the repository. Consequently, the methodology for infiltration could be left at this stage to be solved in the next design step.

7.5.3 Lining

It is anticipated that the first 5–600 meters/50–60 m depth) length of the ramp, located in fracture domain FFM02, may encounter inflows that are difficult to control with grouting from inside the tunnel. A consequence of these inflows is an unacceptable drawdown of the groundwater level around this section of the ramp. A waterproof lining for this portion of the ramp was considered as an option to reduce this potential drawdown. However infiltration may be a more feasible technical solution to offset the environmental effects associated with this drawdown and should be evaluated as the primary option in the final design.

7.6 Summary

The expected inflows and the grouting requirements to minimise those inflows at Forsmark are correlated to hydraulic domains. Within the rock mass hydraulic domain near the ground surface (0–150 m) the inflows to the Repository Access excavations may be large, requiring systematic pre-grouting using cement-based grouts. However, during the construction of the Forsmark SFR Facility to depth of 140 m, the inflows were low (<6 l/100 m) even without pre-grouting. Below 150 m depth, the rock mass is sparsely fractured and the hydraulic domain contains few flowing fractures (<0.005/m). As a result the expected inflows are very low and consequently systematic grouting is not anticipated. The predicted grout takes for the various underground openings in each functional areas are given in *Table 7-5*. The grouting of the flowing fractures anticipated in the rock mass and deformation zones in the Deposition Area will be localised, possibly requiring the application of new grouting technology.

Table 7-5. Estimated distribution of grout (m^3/m) for different functional areas.

Functional areas/Underground openings	Length (m)	Grout-take (m^3/m)
Repository Access		
Ramp, 0–100 m	1,000	0.5–2.0
Ramp, 100–200 m	1,000	0.15–0.55
Ramp, > 200 m, no grouting	2,700	–
Shafts, 0–100 m	100	0.45–1.7
Shafts, 100–200 m	100	0.1–0.4
Shafts, > 200 m, no grouting	270	–
Central Area		
Caverns	50	0.05–0.02
Caverns, no grouting	490	
Deposition area		
Deposition tunnels	250	0.05–0.2
Deposition tunnels, no grouting	55,800	–
Transport tunnels	400	0.08–0.3
Transport tunnels, no grouting	4,500	–
Main tunnels	2,200	0.08–0.3
Main tunnels, no grouting	4,500	–

8 Uncertainty and risk in Design D2

8.1 Strategy

Geotechnical engineering for underground design is fundamentally about managing risk. /Stille et al. 2003/ summarized 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 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 in SR-Site.

The confidence in the underground design and of the repository layout described in D2 relics primarily on the confidence in the site descriptive model for Forsmark given in SDM Site. This site descriptive model was used as the basis 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 Forsmark and these values were specified in the SER /SKB 2008a/. 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 nor 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

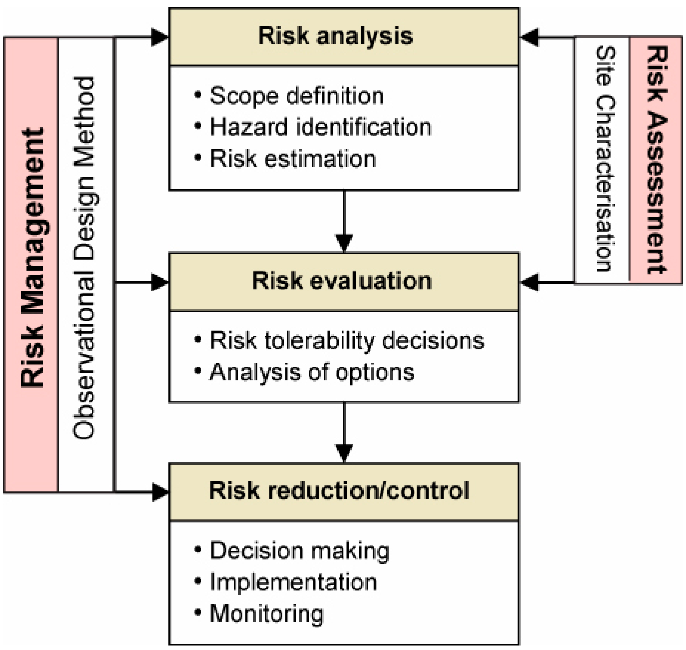


Figure 8-1. *Illustration of the risk management process and its linkage to the Observation Design Method and Site Characterisation, modified from /IEC 1995/.*

this value, may significantly impact the design. For example, the scenario for the potential loss of deposition-hole 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.

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.

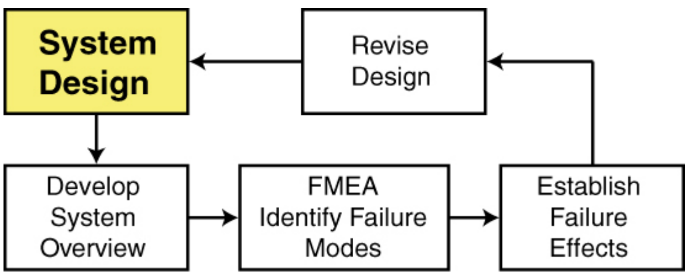


Figure 8-2. *Illustration of the role of Failure Modes and Effects Analyses in the design process, modified from /Rausand and Høyland 2004/.*

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-hole positions.
4. Assessment of inflow potential.
5. Assessment of spalling potential.

The uncertainties and confidence 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-hole positions due to long fractures

In order to mitigate the impact of potential future earthquakes deposition-hole positions are selected such that they do not intersect too long fractures. Theoretically fractures with radii larger than 150 m should be avoided, but since fracture sizes may be very hard to measure more robust criteria are needed. Currently, deposition-hole positions must satisfy the *Extended Full Perimeter Intersection* (EFPC) criterion /Munier 2006/. The resulting potential loss of deposition-hole positions was assessed using the Discrete Fracture Network (DFN) developed for the site. DFN's is 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 occurrence of 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 threshold of 150 m. The current design applied the EFPC criterion using a variety of DFN models to estimate the possible range in the loss of deposition-hole positions. However, there is a lack of confidence in this DFN approach because of our inability to validate the methodology at the repository level and because where it has been applied it has been shown that the EFPC criterion is unnecessarily conservative. There is little doubt that some deposition-hole positions may be rejected due to long discrete fractures and therefore an allowance for this loss must be assessed. The allowance used in the design covers all probable outcomes, including those using the DFN modelling and hence there is confidence that the number of deposition-hole positions that may be lost due to long fractures will be within the values used in the reference design. There is confidence based on the SDM Site, that number of number of deposition-hole positions that may be lost due to long fractures will be relatively minor. 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 inflows for the reference design. These methods are calibrated to hydrogeology measurements for the site and hence there is confidence in the estimated inflows to the underground excavations.

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 2009a/ 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 4×10^{-9} m^2/s /Smith et al. 2007, Appendix C2/. Possible Observation Design approaches for meeting this criterion are:

1. Reject potential deposition-hole position with inflows exceeding the inflow criterion of 0.1 l/s.
2. 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.
3. 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 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 are not strictly controlled. In a repository environment 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 channelised flow. In the current design traditional methods 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 results were not available at the time of this writing. While there is less confidence in the grouting methods and the level of effort required controlling these very low inflows, there is confidence that the number of inflows requiring grouting at Forsmark will be relatively few.

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 and therefore there is confidence in the approach. However, 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 four primary design constraints that significantly influence the repository layout: (1) deformation zones requiring a respect distance, (2) avoidance of minor deformation zones, (3) thermal rock mass properties, (4) the approach used in the thermal dimensioning, and the orientation of the maximum horizontal stress. The layout includes provision for all the deterministic deformation zones identified in the site descriptive model. In the current layout there are no deposition-hole positions in any of these deformation zones. In addition there is a respect distance of 100 m on either side of the borders of the deformation zones longer than 3 km and there are no deposition-hole positions placed in this respect distance. It should be noted that the deformation zones at Forsmark in the target volume 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.

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 6 m in RFM029 and 6.8 m in RFM045. 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. The thermal conductivity of the rock has been assessed using geostatistical techniques. The result is a distribution for the conductivity that results in relatively low values for the tails of the distribution compared to the mean value. These low thermal conductivity values are associated with the darker mafic rocks at Forsmark that typically occur as dykes. Because of the discrete nature of these dykes they are readily identifiable in the field and can be excluded as potential deposition-hole positions. By excluding these low conductivity rocks the design value for the thermal conductivity could be increased which would reduce the repository footprint.

Figure 8-3 shows the relationship between deposition tunnel and deposition hole spacing for various thermal conductivities that meet the maximum temperature criterion for the buffer. The reference D2 design uses a thermal conductivity value of approximately 2.9 W/(m·K) while the mean value for RFM029 is 3.57 W/(m·K). As shown in Figure 8-3 is 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. Optimising the tunnel and the canister spacing for the thermal properties used in this D2 would reduce the footprint area by approximately 20%. 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 easily result in a significantly reduced repository footprint.

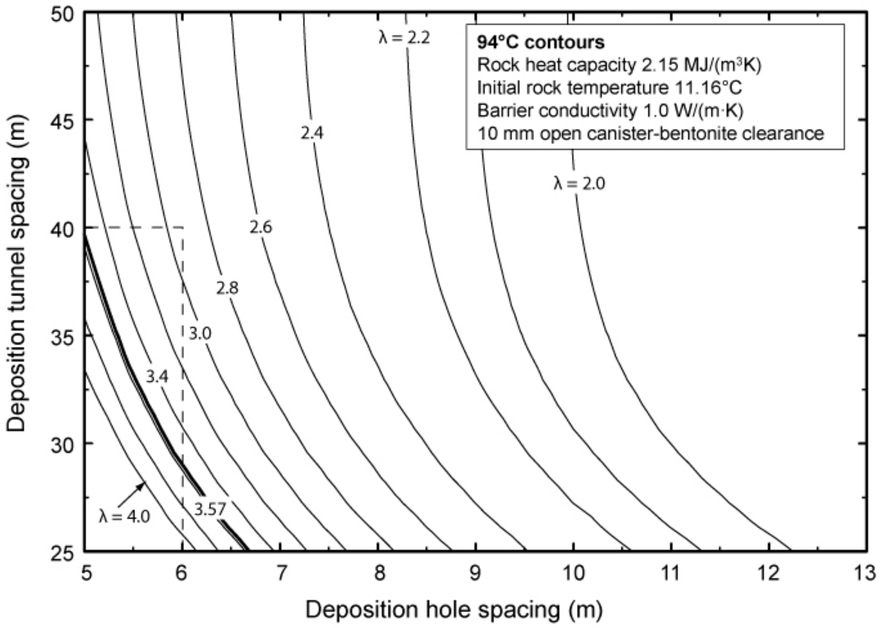


Figure 8-3. *Approximate deposition hole (canister) spacing versus deposition tunnel spacing for different thermal conductivity. The bold line indicate the mean thermal conductivity in domain 29 3.57 W/(m·K) /based on Hökmark et al. 2009/.*

The long axis of the deposition tunnels has been oriented in the general direction of maximum horizontal stress. This orientation minimises the stresses acting on the tunnel boundary and hence reduces the potential for overstressing the rock.

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 programme 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 1.2.2 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 2000c/ 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 2008c/. A brief summary of the remaining uncertainties discussed in /SKB 2008c/ is provided below in Table 8-1.

It should be noted that the uncertainty assessment of the SDM-Site in /SKB 2008c/ 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.

Table 8-1. Summary of the uncertainties in Site Descriptive Model at the end of the site investigations /SKB 2008c, SKB R-08-82/.

Geological Uncertainties (Section 3.2 of /SKB 2008c/)

1. The size of the gently dipping zones are not fully known, and are thus extended to the nearest steeply dipping zone.
2. There may be deformation zones longer than 1 km but not as long as 3 km, not included in the deterministic model. However, these zones are considered in the statistical description.
3. There is considerable variation and uncertainty in the size and intensity of gently dipping and sub-horizontal fractures in the upper part of the bedrock inside the target volume.
4. The size distribution and size-intensity (DFN) models for fractures at repository depth are uncertain
5. The size and spatial distribution of subordinate rock types is uncertain, especially for anomalously thick amphibolite bodies in domain RFM045.

*Rock Mechanics and Thermal model Uncertainties (*Section 3.3 of /SKB 2008c/)

1. Rock stress magnitudes are uncertain even if upper bounds can be provided. There is much less uncertainty in stress orientations and it is judged known within narrow bounds.
2. The thermal conductivity distributions, and especially their lower tails, are uncertain. Especially the occurrence of low conductivity amphibolite bodies in rock domain RFM045 has a large impact on the lower tail of the distribution and the typical length distributions for these are uncertain.

Hydrogeological Uncertainties (Section 3.4 of /SKB R-08-82/)

1. There are remaining uncertainties in the spatial variation of hydraulic properties of the rock mass (e.g. Hydro-DFN) in the potential repository volume, but the uncertainties are much reduced compared with previous model versions. A large number of PFL-tested boreholes show a consistent picture and confirms the existence and extent of the low permeability volumes at depths in FFM01 and 06.2.
2. The hydraulic properties of the deterministic deformation zones, their spatial variability, anisotropy and scaling inside the target volume are uncertain.
3. There is uncertainty in the hydraulic properties outside rock domains RFM029 and RFM045

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 2008c/ 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 2008c/ and the likelihood of the geohazard was independently assessed by an Advisory Expert Team (Figure 1-7).

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 2008c/. The likelihood descriptors were assigned based on the notion that the occurrence of the geohazard would be widespread throughout the design 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 8-2. 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 evaluated during the Design step D2.

	Geohazard: Geology model	
Identifier	Descriptor	Likelihood
G1	Distribution of rock types deviates from the design value	Unlikely
G2	Geological boundaries deviates from those used in the design	Likely
G3	Frequency of long fractures exceeds the values predicted by the Geo DFN Model	Extremely unlikely
G4	New Deformation zones between 1 km and 3 km long trace length	Unlikely
G5	New Deformation Zones requiring Respect Distance	Extremely unlikely
G6	Thickness of Minor Deformation Zones (MDZ<1 km) exceeds the estimated valued in SDM Site	Unlikely

	Geohazard: Hydrogeology model	
Identifier	Descriptor	Likelihood
H1	Frequency of water bearing fractures in FFM02 exceeds the Hydro-DFN prediction used in the design	Unlikely
H2	Frequency of water bearing fractures in FFM01 between FFM02 and the repository Elevation exceeds the Hydro-DFN prediction used in the design	Unlikely
H3	Frequency of discrete flowing fractures, with flows unsuitable for deposition holes or deposition tunnels, below 400 m in FFM01 exceeds the Hydro-DFN prediction used in the design	Extremely Unlikely

Identifier	Descriptor	Likelihood
	Geohazard: Rock Mechanics/In situ stress model	
R1	Properties of the major and minor deformation zones deviates from the design value	Unlikely
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

Identifier	Descriptor	Likelihood
	Geohazard: Thermal model	
T1	Geometrical distribution of thermal rock domains deviates from the design value	Unlikely
T2	Rock containing mafic (Amphibolite) dykes (low T properties) occurs more frequently causing the thermal conductivity distribution in the up-scaled model to be 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-hole 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 possible inadequacies and/or 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 104 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 Forsmark. 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 the site conditions, which is considered a more serious consequence.

Repository Area

The Repository layout was developed to provide space capacity for 6,000 canisters. For design purposes the loss of deposition-hole positions is expressed as a percentage or the percentage utilisation. For example, if the loss of deposition-hole positions is expected to be 30% (70% utilisation), then the number of deposition-hole positions required to accommodate this loss is 8,571 (8,571–6,000=2,571), i.e. 2,571 extra deposition-hole positions will be required to meet the 6,000 canister requirement. The reasons for rejecting a deposition-hole 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 2009a/: *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 2009a/, *the initially placed 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.* A deposition hole overbreak of maximum 5 cm 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-hole positions for each of these criteria in the D2 reference design is discussed in the following sections.

8.3.3 Potential loss of deposition-hole positions

Due to Long fractures

The loss of deposition-hole positions due to long fractures was evaluated using statistical approaches due to the stochastic and uncertain nature of the site description. SER / SKB 2008a/ 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/. The degree of utilisation varied between approximately 70 and 90% depending on which DFN model was used. The SER /SKB 2008a/ recommended a "most likely" utilisation value of 89% for design purposes. In summary, while a value of 11% is proposed as the "most likely" loss of deposition-hole positions due to long fractures, an "unlikely maximum" value of 30% was also evaluated in the layout studies to account for the uncertainty in the DFN model.

Due to inflows

The Forsmark SER /SKB2008a, Table 2-13/ concluded that at most about 6% of all potential deposition hole position have total transmissivity of intersecting fractures larger than $4 \cdot 10^{-9}$ m^2/s /Smith et al. 2007, Appendix B2/. As can be seen in Table 2-13 /SKB 2008a/, more than 6% of all deposition holes meet this criterion. However, most – if not all – of these 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. In summary, the most likely situation is that very few additional deposition holes will be lost due to high inflow, that are not already lost due to their intersection with long fractures. At the most extreme an additional 6% could be lost due to high inflows.

Due to spalling

The deposition holes at Forsmark will be located in Ground Type 1, which will be found primarily in the dominant rock types in RFM029 and RFM045. At the repository level the proportions of RFM029 and RFM045 is approximately 80% and 20%, respectively. The laboratory uniaxial compressive strength (UCS) of the main rock type in RFM045 (373 MPa) is significantly greater than the UCS of the main rock type in RFM029 (226 MPa). For the purposes of D2, only spalling analyse for the dominant rock type in RFM029 has been carried out since the layout is governed by findings from these analyses for this weaker rock. The analyses used to assess the potential for spalling in the deposition holes are provided in Appendix A. Only the findings from the analyses are summarised here.

Three dimensional elastic stress analyses were carried out for Forsmark with the deposition tunnel aligned perpendicular, 60 degree, 30 degree and parallel to the maximum horizontal stress. These analyses were carried out for the "most likely" and "unlikely maximum" in situ stress models. The maximum tangential stresses on the boundary of the deposition hole for each model was analysed and it was demonstrated that when the deposition tunnel is aligned parallel to the maximum horizontal stress the maximum tangential stress concentration on the wall of the deposition hole is at a minimum. For the "most likely" stress model only the deposition tunnels aligned greater than 30 degrees to the maximum horizontal stress will produce tangential stress concentrations that are greater than the spalling strength and in these situations the spalling will occur above the top of the canister. For this stress model the layout plan can utilize deposition tunnels that are aligned between and 0 and 30 degree. However for the "unlikely maximum" stress model the tunnels must be aligned with the maximum horizontal stress to reduce the risk of spalling and should spalling occur it will occur over essentially the entire length of the canister. These elastic stress analyses were used to establish the factor of safety for spalling and the potential depth-of-spalling.

The depth of spalling was calculated using the approach outlined in Appendix B for those cases where the factor of safety for spalling was less than 1. A deposition hole overbreak of maximum 5 cm relative to the nominal diameter is considered acceptable. Thus deposition holes that produce 5 cm of spalling or less would not contribute to the loss of deposition-hole positions from the perspective of achieving an acceptable buffer density. Figure 8-4 shows the loss of deposition-hole positions (out of 6,000) and the associated depth of spalling. Figure 8-4 illustrates that only approximately 100–200 deposition holes would sustain overbreak that exceeds the 5 cm criterion. This is considered an insignificant loss as the reference design currently has provision for potential loss of 1,818 deposition-hole positions. It should be noted that orientating the deposition tunnel parallel to the maximum horizontal stress could eliminate the potential loss 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-hole positions.

The Consequence categories for the loss of deposition-hole positions were developed based on the degree of utilisation. A loss of deposition-hole positions greater than the 1,818 currently provided for in the D2 layout is classed as a "Major Consequence" as a loss >1,800 would require access to areas outside the design target-area . A loss of deposition-hole positions between 1,000 and 1,800 is the maximum loss expected in the current design and this is classed as a "Moderate" consequence. A loss of deposition-hole positions between 500 and 1,000 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-hole positions <500 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-hole 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.

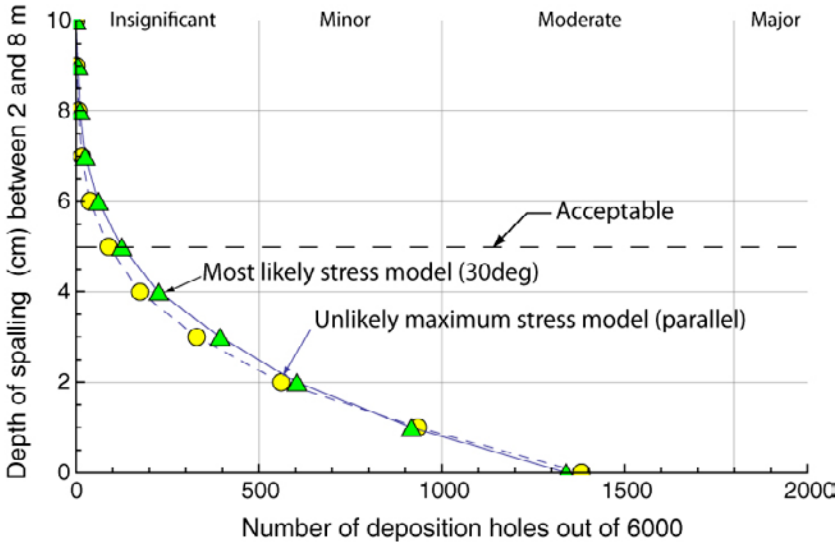


Figure 8-4. *Loss of deposition holes for the "Most likely" stress model with the deposition tunnels at 30 degrees to the maxim horizontal stress and the "Unlikely maximum" stress model with the deposition tunnel parallel to the maximum horizontal stress.*

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+ 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 SFR Facility to depths of approximately 140 m in similar rock quality to that expected at Forsmark. The consequences are evaluated against the performance experience for those facilities which have been reported in /Carlsson and Christiansson 2007a/.
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	<500 (Less than expected)	The current layout has a gross capacity of 7,818 deposition-hole positions. This capacity takes into account the loss of deposition-hole positions from all geohazards. Exceeding the gross capacity of the site would have major consequences.
C2	Minor	500–1,000 (Expected loss)	
C3	Moderate	1,000–1,800 (Maximum loss with current design)	
C4	Major	>1,800 larger loss than current design (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 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 8-5). 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 8-5 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.

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 8-5. *Illustration of the binning approach used to highlight risks using the qualitative Likelihood and Consequence risk matrix.*

It must be remembered that the risks described here are the risks that the reference design presented in D2 will need 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 in SR-Site. Risk Category N/A in Figure 8-5 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 8-3 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.

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 G2, G3, R1, R2, R3, G4, R4 and G5, while having varying classes 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 in FFM02 exceeds the Hydro DFN prediction used in the design) is ranked as “Unlikely” because of the confidence in the data from the large number of boreholes penetrating this fracture domain. However, if it occurs, the consequences are rated as “Minor” and could results in schedule delays that amount to 1 to 2 years due to the increased demands and complexities of grouting to reach acceptable levels of inflow. Geohazard H2 (Frequency of water bearing fractures in FFM01, between FFM02 and the repository elevation exceeds the Hydro DFN prediction used in the design) is also classed as “Unlikely” for the same reasons as H2 and also because the frequency of open flowing fractures is significantly reduced in FFM01. Even if the frequency is underestimated the consequence on the schedule is classed as “Minor” (1–2 years).

Geohazard R4 (Horizontal stress magnitudes exceed the Unlikely maximum stress model) is classed as “Extremely Unlikely”. However, should it occur it will be below 300 m depth and as shown by the construction of AECL’s shaft in comparable stresses /Kuzyk et al. 1991/, the consequences of stress magnitudes on the shaft construction schedule are classed as insignificant. The consequence of R4 on the construction of the access ramp is also classed as insignificant as the orientation and shape of the ramp can be modified should R4 be encountered. The likelihood of occurrence of geohazards G5 and G6 are rated as “Extremely unlikely and Unlikely”, respectively. There is simply no evidence in the geological model that would suggest the possibility of new deformation zones in the Target Volume. However, even if a new deformation zone would be discovered the effect of crossing such a zone on the construction schedule for the shaft access would be minor.

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

Repository Access					
Likelihood	Very likely				
	Likely	G2,			
	Unlikely	R1, R2, R3, G1,G4	H2,H1		
	Extremely Unlikely	R4, G5,G3			
		Insignificant (<1 year)	**Minor** (1–2 year)	**Moderate** (2–3 Years)	**Major** (>3 Year+)
		Consequences (Schedule Delays)			

Central Area					
Likelihood	Very likely				
	Likely	G2			
	Unlikely	G1,H2, R1, R2	G4, G6, R3		
	Extremely Unlikely	G3		R4, G5	
		Insignificant (Orientation adjustment)	**Minor** (Cavern Shape Modified)	**Moderate** (Concrete lining needed for stability)	**Major** (CA moved requiring land outside the building permit)
		Consequences (Stability & Location)			

Repository Area					
Likelihood	Very likely				
	Likely	G2			
	Unlikely	G1,G6, R1	R2 T1,T2	G4,R3	
	Extremely Unlikely			H3	G3,G5,R4
		Insignificant (<500)	**Minor** (500–1,000) (Most Likely)	**Moderate** (1,000–1,800)	**Major** (>1,800)
		Consequences (Loss of deposition-hole positions)			

Central Area

The primary concern for the Central Area is the stability of the caverns that vary in cross sectional area from 104 m^2 to 255 m^2. The consequence of geological geohazards G1 and G3, the hydrogeology geohazard H2, and the rock mechanics geohazard R1 and R2 on the stability of the caverns is considered "insignificant". Because the 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). Because of the very low open fracture frequency at the Central Area depth, water inflows are not expected to be an issue. This is also supported by the experience at the SFR /SKB 2008a/.

The likelihood of the geohazard R4 (Horizontal stresses exceed the "Unlikely maximum" Model) is rated "Extremely unlikely". However should R4 be encountered, the consequence is classed as "Moderate" as extensive reinforcement (shotcrete and bolts and possibly a concrete lining for the roof and walls) could be required for the 100-year life of the largest caverns. Again this situation would be known prior to construction of the caverns using the Observational Method. Geohazard G5 is also classed as "Extremely unlikely" as there is no evidence to support the possibility no matter how remote for a new deformation zone longer than 3 km. The consequence of R3 and

G4 are both rated as “Unlikely”. Should a new minor deformation zone be encountered (G4) the consequence on the cavern stability is rated as “Minor” as the deformation zones at Forsmark are described as Ground Type 3 (Good quality rock mass). Should the geohazard R3 (Horizontal stress magnitudes exceed “most-likely” model but not the “unlikely maximum” model) be encountered, the consequence will result in modification of the cavern shape, which is also classed as “Minor”. The caverns will also be aligned in the general direction of maximum horizontal stress, which will reduce the consequence of elevated stress magnitudes.

The geohazards will also impact the schedule but the construction of multiple caverns would provide ample opportunity to increase resources using multiple heading, which would reduce the impact of these geohazards on the schedule.

Repository Area

The primary consequence for the repository area is a loss of deposition-hole positions greater than that used for the reference design. The current design can accommodate 7,818 deposition-hole positions, i.e. 1,818 beyond the 6,000 deposition holes required for the reference design layout. The geohazards G1 (distribution of rock types), G2 (geological boundaries) and R1 (properties of deformation zones) are considered to have “insignificant” consequences to the deposition hole layout, that is the loss of deposition-hole positions due to these geohazards is expected to be less than 500.

The consequences of geohazard R2 (Orientation of maximum horizontal stress), T1 (Distribution of thermal rock domains) and T2 (Mafic dykes) are expected to be minor (loss of deposition hole 500–1,000). All of these geohazards are classed as “Unlikely” because of the uniformity and consistency of the data in the site descriptive model.

The consequences of geohazards G4 (new 1 km –3 km long deformation zones) R3 (horizontal stress magnitudes exceed the most-likely model but not the unlikely maximum model) and H3 (frequency of unsuitable flows for deposition-hole positions) are classed as “Moderate” (1,000–1,800 loss of deposition-hole positions). It should be noted that the open flowing fracture frequency at the repository level is less than 0.005/m, which would indicate a rock mass with very few open fractures and consequently the loss of deposition-hole positions may be very low. The relatively uniform geological conditions at Forsmark, makes the detection of deformation zones relatively easy. Hence it is extremely unlikely that these deformation zones would have gone undetected during the site investigations. The occurrence of Geohazard R3 is also classed as “Unlikely”. Should Geohazard R3 occur, the orientation of the deposition tunnels could be used to mitigate the consequences. This is described in more detail in Appendix C.

The occurrence of geohazard G3 (Frequency of long fractures based on the Geology DFN model), G5 (New deformation zones requiring a respect distance) and R4 (horizontal stresses exceeding the unlikely maximum magnitudes) are considered extremely unlikely. There is simply no evidence from the site descriptive model that these conditions could occur. Should they occur their consequence is classed as “major” since it implies a significant change in the SDM Site. Should these conditions occur alternative design arrangements would have to be evaluated.

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 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 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 programme 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 8-3 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 (in preparation).

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.

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
G2. Geological boundaries not spatially correct	Geological Descriptors	Borehole logging & Tunnel mapping	All investigative boreholes and underground excavations regardless of locations
G3. Predictive capability of the DFN Model inadequate	Geological Descriptors	Borehole logging & Tunnel mapping	All investigative boreholes and underground excavations regardless of locations
G4. New Deformation zones <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
Geohazard: Hydrogeology model			
H1. Frequency of water bearing fractures in FFM02 underestimated	Water Inflow to excavations	Probe hole drilling	All investigative probeholes and excavations within FFM02
H2. Frequency of water bearing fractures in FFM01 between FFM02 and the repository Elevation under estimated	Water Inflow to excavations	Probe hole drilling	All investigative probeholes and excavations between FFM02 and repository elevation
H3. Frequency of localised flowing fractures, with flows unsuitable for deposition holes or deposition tunnels, below 400 m in FFM01 under estimated	Water Inflow to excavations	Probe hole drilling	All investigative probeholes and excavations in the repository area
Geohazard: Rock Mechanics/In situ stress model			
R1. 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.
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 respository elevation.
R3/R4. Horizontal stress magnitudes	Shmax/Shmin	Convergence/ Overcoring	Measurements to be conducted in the shaft and ramp access below 150 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. Rock containing mafic (Amphibolite) dykes (low T properties) more frequent making up-scaling model wrong	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 programme indicate that the in situ conditions are outside the range of values used for the design. Figure 8-6 shows the tentative schedule for the construction of the underground excavations that will provide Repository Access and the Central Area. Also shown in Figure 8-6 are the geohazards listed in Table 8-3 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.

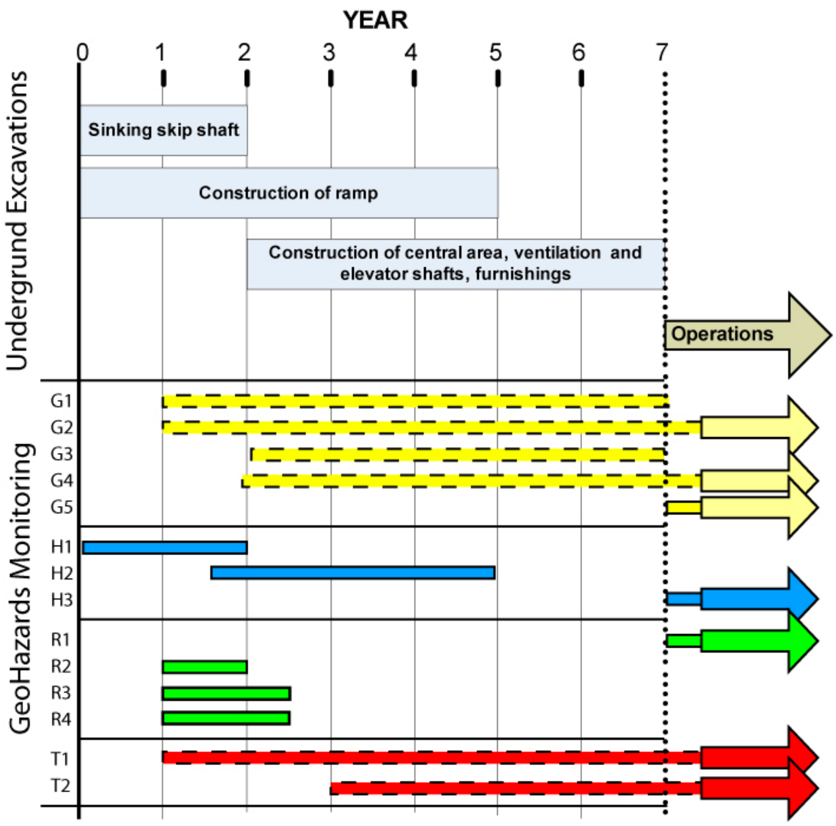


Figure 8-6. Tentative schedule for the construction of the Access ramp and shaft , and the Central Facility. Also shown are the monitoring periods for the geohazards listed in Table 8-3. Note that the stress model uncertainty would be established at the end of Year 2.

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 2008c/. 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-hole positions for the Repository layout. It should be noted that the consequences associated with the loss of deposition-hole 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.

An assessment of the impact of site uncertainties (geohazards) on the reference design was evaluated using a qualitative likelihood-consequence risk matrix. The risk matrix was developed for each of the functional areas and each of the geohazards individually assessed. None of the consequences from the geohazards assessed would render the repository unsuitable for the purpose intended. A preliminary implementation plan for the Observational Method was outlined that showed how the uncertainty in the design parameters could be reduced during construction of the repository.

It is clear from the analyses in this chapter that no amount of surface investigations or sophisticated design analyses will reduce the design risk and uncertainties to zero. It is simply not achievable for an underground project at the scale of a nuclear waste repository. However by using conservative design parameters in conjunction with various design tools, combined with uncertainty and risk analyses, there is a high confidence that a repository can be constructed that will meet all safety requirements. The likelihood of success is greatly enhanced by formally linking design procedures and assumptions to the Observational Method. This design approach requires that the Observational Method be fully integrated in the next design step and construction plans for the repository. In the next design step these implementation plans must be fully developed in conjunction with detailed Failure Modes and Effects Analyses.

9 Conclusions

The Complete Site investigations for the Forsmark site were completed in 2007 and the findings summarised in SDM Site. During the site investigation, several studies and design steps were carried out to ensure that sufficient space was available for the deposition requirement within the boundary of the tectonic lens. 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 within the tectonic lens at an Elevation of –470 m. The layout has a gross capacity of 7,818 deposition-hole positions, which provides for a loss of deposition-hole positions of approximately 23% (1,818 extra deposition-hole positions).
3. This layout incorporates all the deterministic deformation zones and a respect distance of 100 m to deformation zones longer than 3 km. No deposition tunnels 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 evaluated 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 has evaluated two methods for separation of the deposition and construction activities by sequencing the construction of the deposition tunnels: (1) Separation by linear development and (2) Separation by side-change. 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)	3.6	
Repository depth (m)	470	
Gross capacity (deposition-hole positions)	7,818	
Repository level:	Volume (…×10^3 m^3)	Length (km)
Transport tunnels	182	4.6
Main tunnels	384	6.4
Deposition tunnels	1,171	61
Deposition holes (6,000)	115	48
Central area and access (…x10^3 m^3)	324	
Total	2,178	120

9.2 Current Design Constraints

1. The location of the surface facilities was limited to the northern part of the target volume with the objective to minimise length of ramp and shafts through the highly transmissive fracture domain FFM02. Within this restricted area 3 surface locations were evaluated. The area at Söderviken was selected due to a small thickness (46 m) of FFM02 at this site, and with respect to existing infrastructure and future flexibility in the utilisation of the industrial area.
2. The layout was constrained to the Target Volume that was investigated and reported in SDM Site Forsmark. Other suitable volumes may exist adjacent to the boundaries of the Target Volume but these have not been considered in these studies.
3. 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. This approach does not optimise the footprint of the repository.
4. No deposition hole positions were located within the 100-m respect distance allocated to major deformation zones with >3 km trace length or the deterministic deformation zones <3 km trace length.

9.3 Expected site conditions

Repository Access

The excavations for the Repository Access will encounter the greatest frequency of open/water bearing fractures. Extra caution and probe drilling will be required for those excavations located between 0 and 150 m depth in these conditions. However, it should be noted that the SFR facility although located outside the tectonic lens was excavated to a depth of approximately 140 m, and did not encounter any construction difficulties and only minor water inflows in the "typical rock mass". The excavations through the Singö fault encountered more significant flows that required pre-grouting.

The excavation of the Repository Access ramp and shaft(s) will result in a groundwater drawdown. If grouting would not be sufficient to prevent negative environmental consequences, simple and robust infiltration measures should be evaluated as a means of meeting the environmental objectives.

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 be relatively massive with few widely spaced water bearing fractures (0.005/m). The cavern orientations have been aligned parallel to the maximum horizontal stress to minimise the tangential stresses on the excavations.

Repository Area

The rock mass within the tectonic lens at the repository horizon is expected to be relatively massive with few widely spaced water bearing fractures (0.005/m). The orientation of the deposition tunnels have been aligned parallel to the maximum horizontal stress to minimise 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 very good quality rock mass anticipated. This conclusion is also supported by the experience at the SFR Facility and other underground excavations at the Forsmark Nuclear Power Plant.

Groundwater Control and Grouting

The results from the analyses indicate that conventional cement grouting measures will generally be sufficient to meet the inflow criterion, but some fractures and deformation zones in deposition tunnels may also require grouting with new grouting technologies, e.g. silica sol. The most comprehensive grouting works are expected from the ground surface to a depth of approximately 50 m where near horizontal open fractures are expected. 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. An assessment of the impact of site uncertainties (geohazards) on the reference design was evaluated using a qualitative likelihood-consequence risk matrix. The risk matrix was developed for each of the functional areas and each of the geohazards individually assessed. None of the consequences from the geohazards assessed would render the repository unsuitable for the purpose intended.
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 the most likely stress model. However, there is uncertainty regarding this design parameter. Some evidence points to lower stress magnitudes while other evidence points to higher stress magnitudes. Evaluation of all possible stress models indicates that mitigation measures using deposition tunnel orientation and opening shape should be adequate in minimising the spalling to acceptable levels.
   Monitoring plans, as part of the Observational Method, should be developed during the next design to reduce the uncertainty in the stress magnitudes and orientations during the construction of the access ramp and shaft.
3. There is a general lack of confidence in predicted loss of deposition-hole positions using the Geo DFN for the repository Level. There is greater confidence in the Hydro-DFN model. The Hydro-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 less than 0.005/m or 1 water bearing fracture every 200 m.

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. Particular attention should be given to the frequency and hydraulic characteristics of the water bearing gently dipping fractures that are prominent in FFM02. 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 in FFM02 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. The deformation zone ENE060A in the current geological model has a surface trace length of 3,120 m based on 2 borehole intersections and surface lineament projections. Deformation zones with a trace length longer than 3,000 m require a "respect distance" and this respect distance for ENE060A has been accounted for in the reference design. The Detailed Design should determine the actual trace length of ENE060A, as the footprint of the repository could be significantly reduced if it could be shown that the trace length is <3,000 m.
5. Establish design rules for establishing the width of deformation zones <3,000 m that must be avoided for deposition-hole positions.
6. The gross capacity of the repository is governed by the boundaries of the tectonic lens within the target volume, the deformation zones requiring respect distance and minor deformation zones. These features can be refined from additional investigations from the surface and from underground excavations. Plans should be developed for defining these boundaries more precisely.
7. 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 20%.
8. 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.

References

Andersson C J, 2007. Äspö Hard Rock Laboratory. Äspö Pillar Stability Experiment, Final report. Rock mass response to coupled mechanical thermal loading. SKB TR 07-01, Svensk Kärnbränslehantering AB.

Arbetsmiljöverket, 2003. AFS 2003.2.

Australian Geomechanics Society, 2000. Landslide risk management concepts and guidelines. Australian Geomechanics Society. Sub-Committee on Landslide Risk Management, Australian Geomechanics 35.49–92.

Bergman SGA, Nord G, 1982. Täta tunnlar lagom. Stiftelsen Bergteknsik Forskning, BeFo Nr. 64.1/82.

Brantberger M, Zetterqvist A, Arnbjerg-Nielsen T, Olsson T, Outters N, Syrjänen P, 2006. Final repository for spent nuclear fuel. Underground design Forsmark, Layout D1. SKB R-06-34, Svensk Kärnbränslehantering AB.

Brantberger M, Jansson T, 2008. Underground design Forsmark. Layout D2. Grouting. SKB R-08-114, Svensk Kärnbränslehantering AB.

Carlsson A, Christiansson R, 2007a. Construction experiences from underground works at Forsmark. Compilation report. SKB R-07-10, Svensk Kärnbränslehantering AB.

Carlsson A, Christiansson R, 2007b. Construction experiences from underground works at Oskarshamn. Compilation Report. SKB R-07-66, Svensk Kärnbränslehantering AB.

Diederichs M S, Kaiser P K, Eberhard K E, 2004. Damage initiation and propagation in hard rock and influence of tunnelling induced stress rotation. Int. J. Rock Mech. Min. Sci. 41. 785–812.

Eriksson M, Petersson J, Danielsson P, Leander M, 2008. Rock mechanics and rock support, Forsmark. SKB R-08-115, Svensk Kärnbränslehantering AB.

Eurocode EN 1997-1, 2004. Eurocode 7. Geotechnical design – Part 1. General rules. European Committee for standardization, CEN, Brussels.

Follin S, Levén J, Hartley L, Jackson P, Yoyce S, Roberts D, Swift B, 2007. Hydrogeological characterisation and modelling of deformation zones and fracture domains, Forsmark modelling stage 2.2. SKB R-07-48, Svensk Kärnbränslehantering AB.

Funehag J, 2008. Injektering av TASS-tunneln. Delrapport tom september 2008. SKB R-08-123, Svensk Kärnbränslehantering AB.

Glamheden R, Fredriksson A, Röshoff, K, Karlsson, J, Hakami, H, Christiansson R, 2007. Rock mechanics Site descriptive modeling Forsmark Stage 2.2. SKB R-07-31**,** Svensk Kärnbränslehantering AB.

Gustafsson L-G, Gustafsson A-M, Aneljung M, Sabel U, 2009. Effects on surface hydrology and near-surface hydrogeology of an open repository in Forsmark. Results of modelling with MIKE SHE. SKB R-08-121, Svensk Kärnbränslehantering AB.

Hansson B, Magnusson J, Söderlund P, 2008. Underground design Forsmark. Layout and construction plan. SKB R-08-113, Svensk Kärnbränslehantering AB.

Hökmark H, Lönnqvist M, Fälth B, Christiansson R, 2008. Rock Mechanic Issues in the Swedish Waste Management Program. SKB ARMA 08-301, Svensk Kärnbränslehantering AB.

Hökmark H, Sundberg J, Kristensson O, Lönnqvist N, Hellström G, 2009. Strategy for thermal dimensioning of the final repository for spent nuclear fuel. SKB R-09-04, Svensk Kärnbränslehantering AB.

IEC, 1995. IEC60300-3-9, Risk Management – Part 3. guide to risk analysis of technological systems, International Electrotechnical Commission.

Kuzyk G W, Morris J R, Ball A E, 1991. Shaft extension design at the Underground Research Laboratory, Pinawa, Manitoba. CIM Bulletin 1991, 84 (945):42–50.

Martin C D, Chandler N A, Read R S, 1996. The role of convergence measurements in characterizing a rock mass. Canadian Geotechnical Journal,1996*,* 33, 363–370.

Martin C D, Cristiansson R, 2002. Rock mechanics considerations in the siting of Sweden's nuclear waste repository. In Proc. 5th North American Rock Mechanics Symposium and 17th Tunnelling Association of Canada Conference: NARM/TAC 2002 (Hammah R, Baden W, Curran J, Telesnicki M, editors) University of Toronto Press, 2002, Vol 2, 1199–1206.

Martin C D, 2007. Quantifying in situ stress magnitudes and orientations for Forsmark Design Step D2. SKB R-07-26, Svensk Kärnbränslehantering AB.

Munier R, 2006. Demonstrating the efficiency of the EFPC criterion by means of sensitivity analyses. SKB R-06-115, Svensk Kärnbränslehantering AB.

Palmström A, Stille H, 2007. Ground behaviour and rock engineering tools for underground excavations. Tunnelling and Underground Space Technology, 2007, 22 363–376.

Peck R B, 1969. Ninth Rankine Lecture: Advantages and limitations of the observational method in applied soil mechanics Geotechnique, 19.171–187.

Rausand M, Høyland A, 2004. System reliability theory: models, statistical methods, and applications. Wiley-Interscience, Hoboken, N.J.

Read R S, 2004. 20 years of excavation response studies at AECL's Underground Research Laboratory. Int. J Rock Mech 41.1251–1275.

Rojat F, Labiouse, V, Kaiser P K, Descoeudres F, 2008. Brittle rock failure in Steg Lateral Adit of the Lötschberg Base Tunnel. Rock Mechanics and Rock Engineering 2008 *in press*.

SKB, 2000a. Geovetenskapligt inriktat program för undersökning och utvärdering av platser för djupförvaret. SKB R-00-30, Svensk Kärnbränslehantering AB.

SKB, 2000b. Platsundersökningar, undersökningsmetoder och generellt genomförandeprogram. SKB R-01-10, Svensk Kärnbränslehantering AB.

SKB, 2000c. Geoscientific programme for nvestigation and evaluation of sites for the deep repository. SKB TR-00-20, Svensk Kärnbränslehantering AB.

SKB, 2001. Site investigations methods and general execution programme. SKB TR-01-29, Svensk Kärnbränslehantering AB.

SKB, 2005. Preliminary site description, Forsmark area – version 1.2. SKB R-05-18, Svensk Kärnbränslehantering AB.

SKB, 2006a. Long-term safety for KBS-3 repositories at Forsmark and Laxemar – a first evaluation. Main report of the SR-Can project. SKB TR 06-09, Svensk Kärnbränslehantering AB.

SKB, 2006b. Site descriptive modelling Forsmark stage 2.1.Feedback for completion of the site investigation including input from safety assessment and repository engineering. SKB R-06-38, Svensk Kärnbränslehantering AB.

SKB, 2006c. Preliminär anläggningsbeskrivning Forsmark. Layout D1. SKB R-06-33, Svensk Kärnbränslehantering AB.

SKB, 2006d. Preliminary site description.Laxemar subarea – version 1.2. SKB R-06-10, Svensk Kärnbränslehantering AB.

SKB, 2007. Final Repository Facility.Underground Design Premises, UDP/D2.SKB R-07-33, Svensk Kärnbränslehantering AB.

SKB, 2008a. Site Engineering Report, SER. Guidelines for underground Design Step D2. Forsmark. SKB R-08-83, Svensk Kärnbränslehantering AB.

SKB, 2008b. Site description of Forsmark at completion of the site investigation phase (SDM-Site Forsmark). SKB TR-08-05, Svensk Kärnbränslehantering.

SKB, 2008c. Confidence assessment Site descriptive modelling SDM-Site Forsmark SKB TR 00-12, SKB TR 08-82 Svensk kärnbränslehantering AB.

SKB, 2009a. Design Premises Long Term Safety. SKB TR-09-22, Svensk Kärnbränslehantering AB.

SKB, 2009b. Slutförvarsanläggning för använt kärnbränsle, Anläggningsbeskrivning layout D – Forsmark. SKB R-09-12, Svensk Kärnbränslehantering AB.

SKB, 2010. Design premises for a KBS-3v repository based on results from the safety assessment SR-Can and some subsequent analyses (currently only as internal SKB report SKB.doc 1188478). Svensk Kärnbränslehantering AB.

Smith P, Johnson L, Snellman M, Pastina B, Gribi P, 2007. Safety assessment for a KBS 3H spent nuclear fuel repository at Olkiluoto – Evolution report, Eurajoki, Finland. Posiva 2007-08, Posiva Oy and SKB R-08-37, Svensk Kärnbränslehantering AB. 298 pages.

Stille H, 1986. Experiences of design of large caverns in Sweden. Proceedings Int. Conf. on Large Rock Caverns, Helsinki, pp 231–241, Pergamon Oxford.

Stille H, Andersson J, Olsson, L, 2003. Information based design in rock engineering. SveBeFo Report 61, Swedish Rock Engineering Research.

Sundberg J, Back P E, Hellström G, 2005. Scale dependence and estimation of rock thermal conductivity. Analysis of upscaling, inverse thermal modelling and value of information with the Äspö HRL prototype repository as an example. SKB R-05-82, Svensk Kärnbränslehantering AB.

Svensson U, Follin S, 2009. Groundwater flow modelling of the excavation and operation periods – SR-Site Forsmark. SKB R-09-19, Svensk Kärnbränslehantering AB.

Terzaghi K, 1946. Rock defects and loads on tunnel supports. In Rock Tunnelling with Steel Supports, Commercial Shearing Inc, 99 pages. (Edited by Proctor RV and White TL).

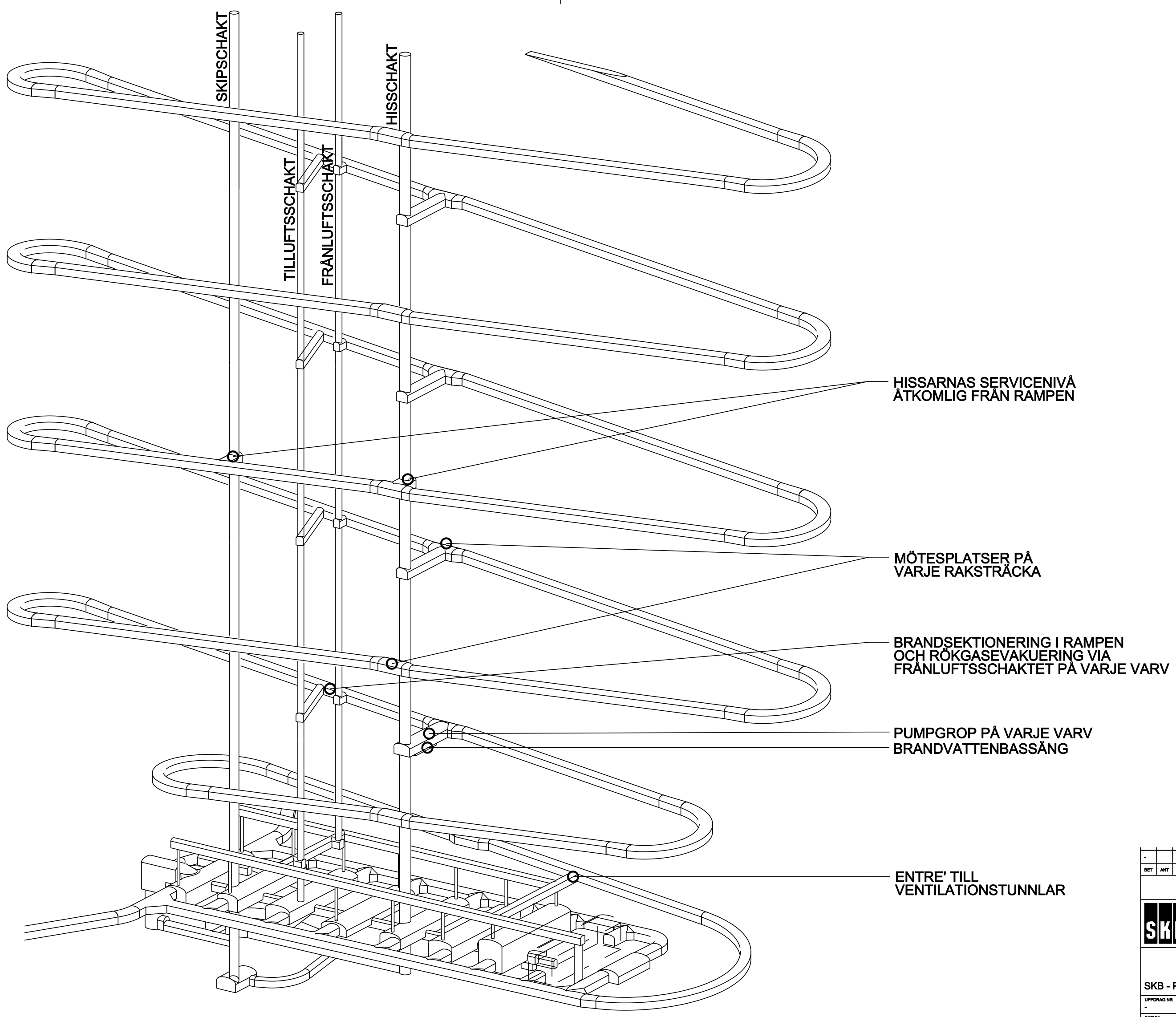
Terzaghi K, Peck RB, 1948. Soil mechanics in engineering practice, 1st edition. John Wiley & Sons, New York.

Vick S G, 2002. Degrees of Belief: Subjective probability and engineering judgement. American Society of Civil Engineers, Reston, Virginia.

Vose D, 2008. Risk Analysis: A quantitative guide. 3rd Edition. John Wiley & Sons, Ltd, Chichester, England.

Appendix A

Typical drawings of the underground openings



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
RAMP OCH SCHAKT
PERSPEKTIV
ÖVERSIKT

SKALA	NUMMER	BET
	9-C140-R-00-0001	-

9_C140R_00_0001.dgn

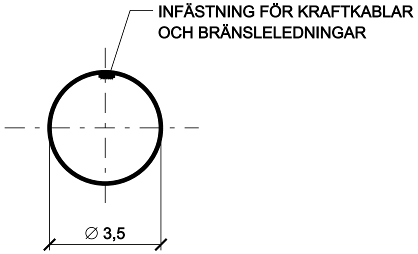
PLOT:2009-06-25 15:16

REF: 9_130LC000_9001.hln

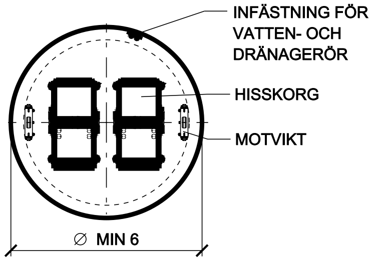
-

FÖRKLARINGAR

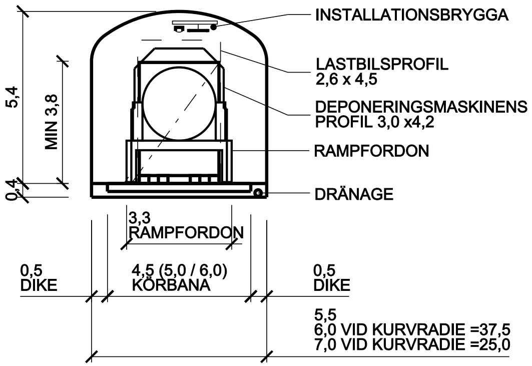
FRÅNLUFTSSCHAKT



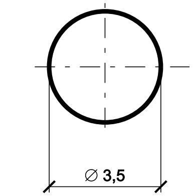
HISSCHAKT



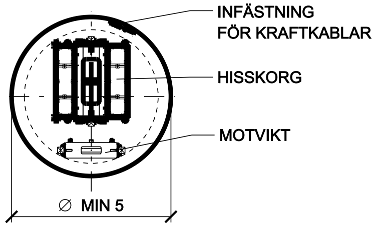
RAMP



TILLUFTSSCHAKT



SKIPSCHAKT



0 5 10m
1:200

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
RAMP OCH SCHAKT
TUNNEL- OCH SCHAKTPROFILER
MÅTT

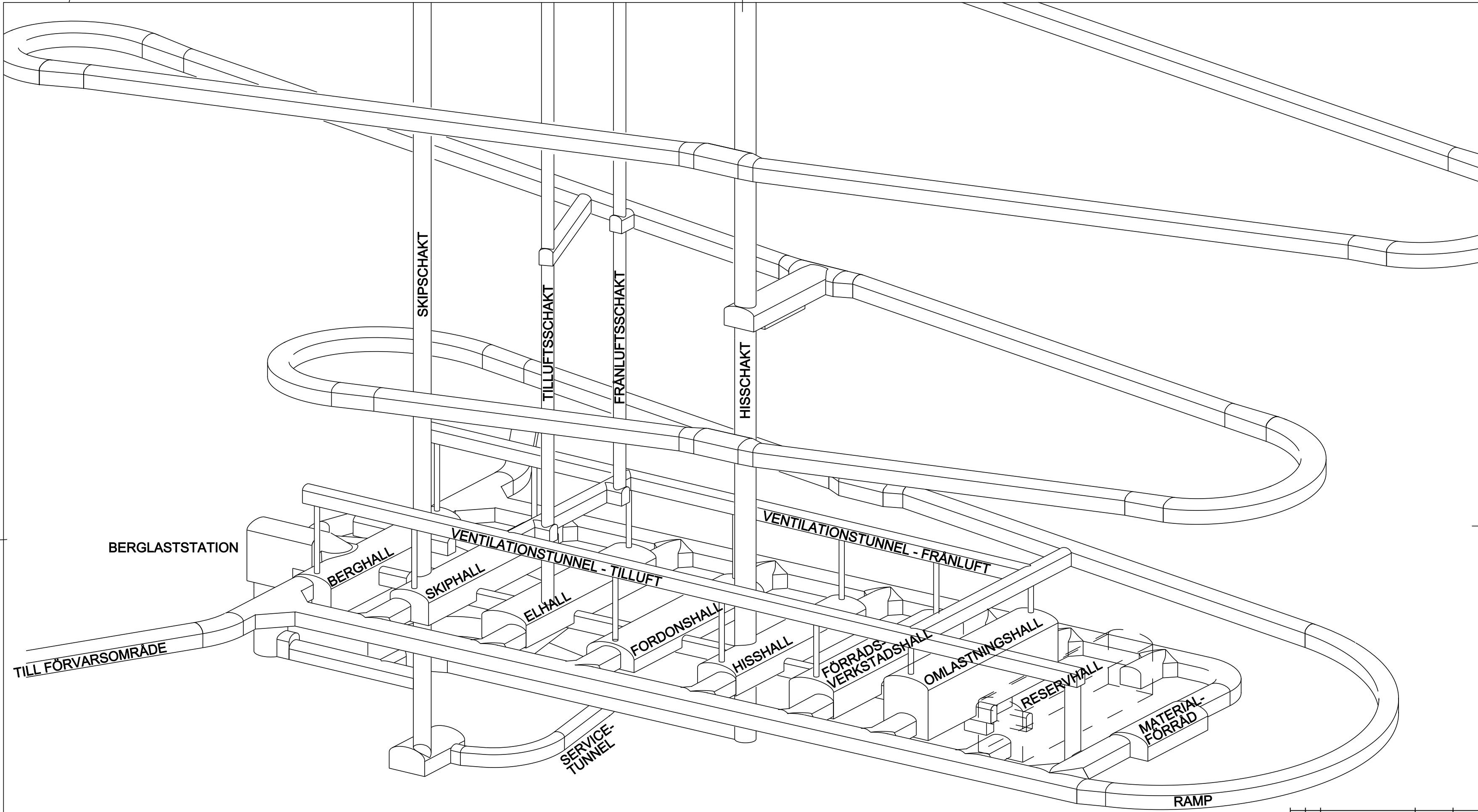
SKALA	NUMMER	BET
1 : 200	9-C140-R-00-0011	-

9_C140R_00_0011.dgn
PLOT:2009-06-25 15:17

REF: 9_140CAB00_0001.dgn

A3

A4



BET	ANT	ÄNDRINGEN AVSER	DATUM	SIGN
-		-	-	-

Slutförvar för använt kärnbränsle

SKB - PROJEKTERING SLUTFÖRVAR

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

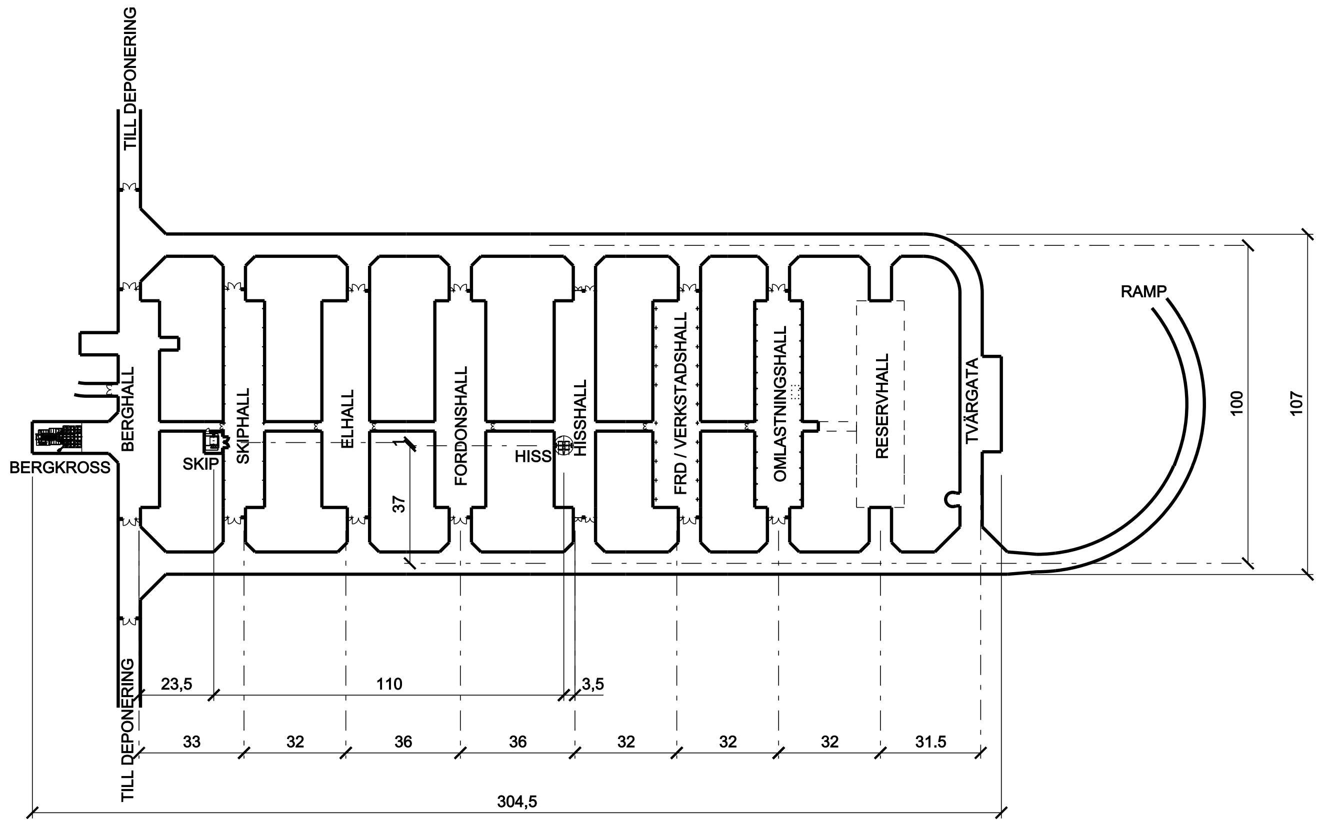
REFERENSUTFORMNING
CENTRALOMRÅDE
PERSPEKTIV
ÖVERSIKT

SKALA	NUMMER	BET
	9-C130-C-00-0001	-

9_C130C_00_0001.dgn

PLOT:2009-06-25 14:53

REF: 9_130CC000_9001.hln
-



BET	ANT	ÄNDRINGEN AVSER	DATUM	SIGN
-		-	-	-

Slutförvar för använt kärnbränsle

SKB - PROJEKTERING SLUTFÖRVAR

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

REFERENSUTFORMNING
CENTRALOMRÅDE
PLAN HUVUDNIVÅ
ÖVERSIKT

SKALA	NUMMER	BET
1:2000	9-C130-C-00-0011	-

9_C130C_00_0011.dgn
PLOT:2009-06-25 16:09

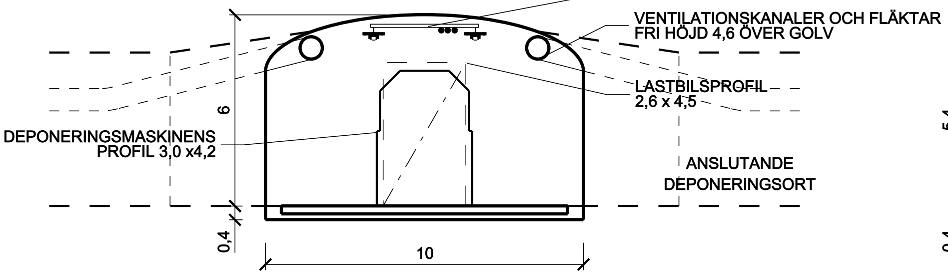
REF: 9_130CC_00_2001.dgn

A5

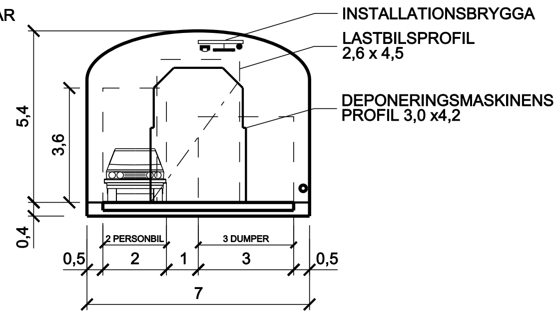
A6

108

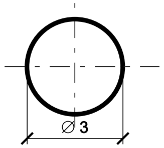
STAMTUNNEL



TRANSPORTTUNNEL



FRÅNLUFTSSCHAKT



0 5 10m
1:200

BET	ANT	ÄNDRINGEN AVSER	DATUM	SIGN
-	-		-	-

SKB Slutförvar för använt kärnbränsle

SKB - PROJEKTERING SLUTFÖRVAR

UPPDRAG NR - | RITAD SWECOOS | GRANSKAD -

DATUM 2009-06-25 | GODKÄND -

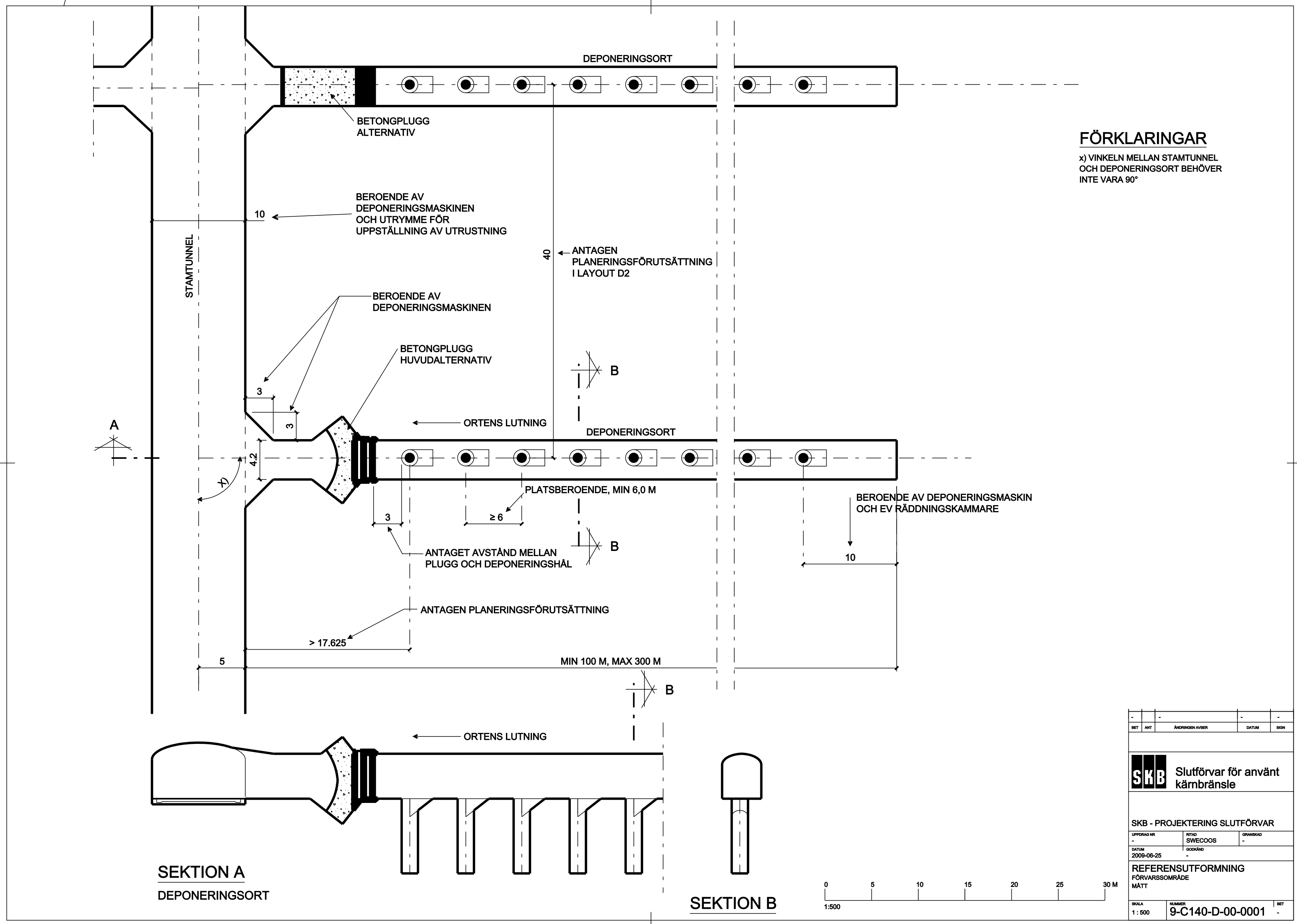
REFERENSUTFORMNING
FÖRVARSOMRÅDE
PROFILER AV TUNNLAR OCH SCHAKT

SKALA 1 : 200 | NUMMER 9-C140-D-00-0011 | BET -

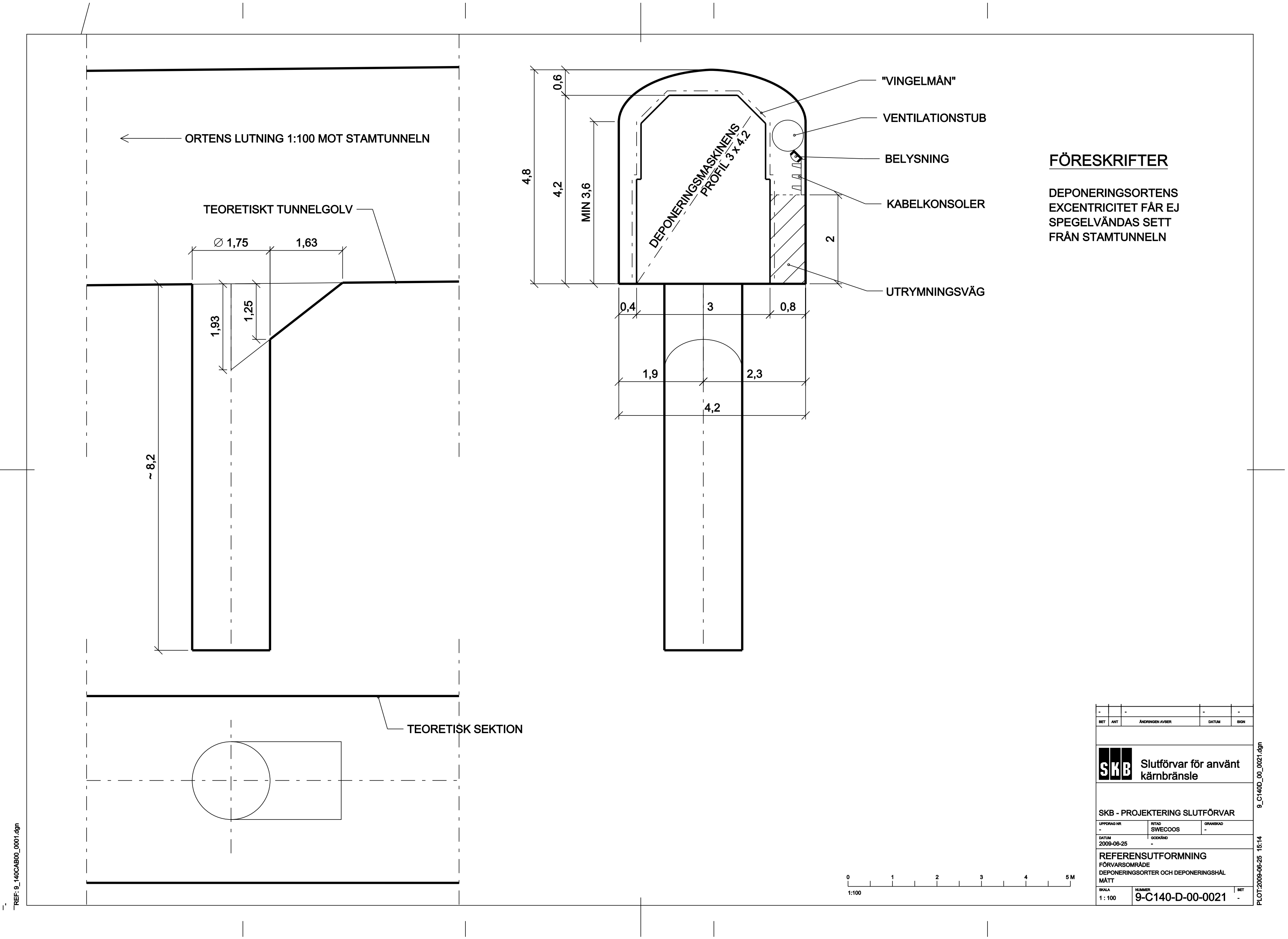
9_C140D_00_0011.dgn

PLOT:2009-06-25 15:12

REF: 9_140CAB00_0001.dgn

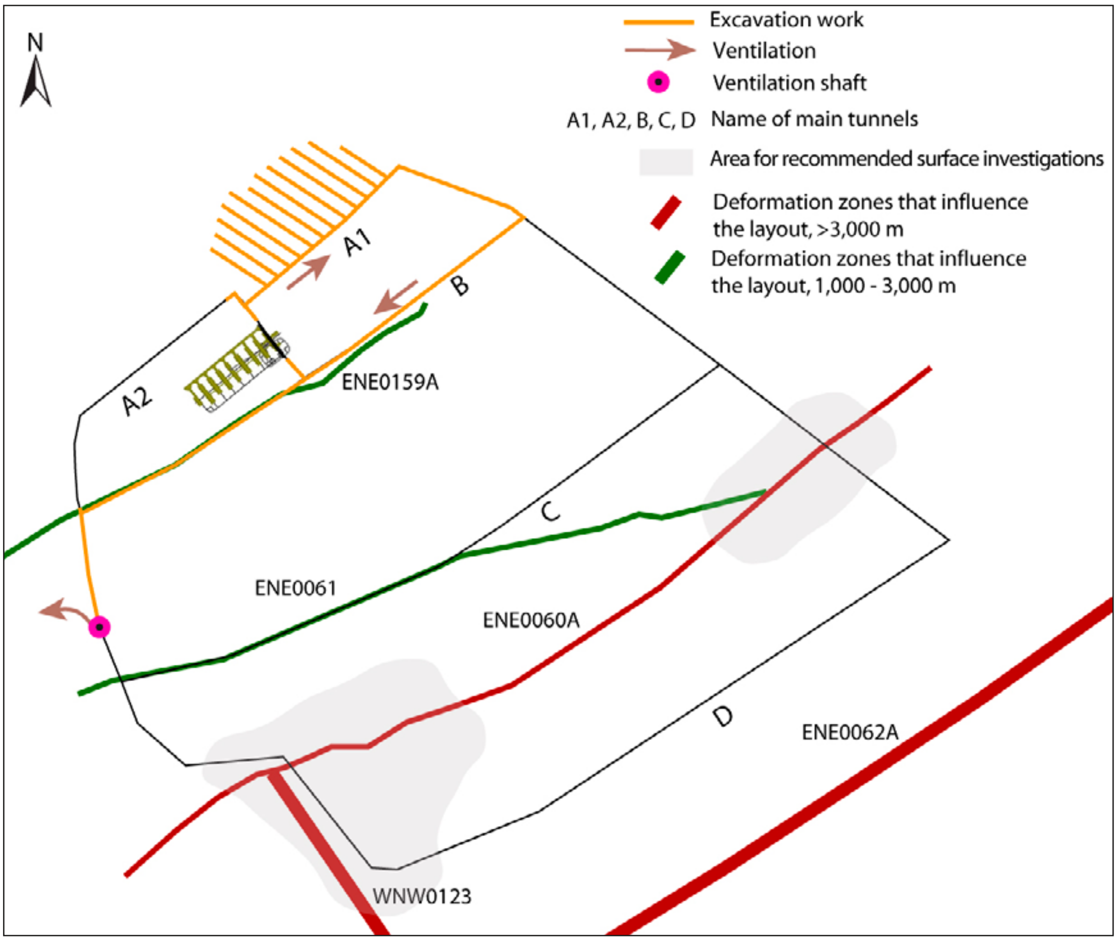

DEPONERINGSORT
BETONGPLUGG
ALTERNATIV
FÖRKLARINGAR
x) VINKELN MELLAN STAMTUNNEL
OCH DEPONERINGSORT BEHÖVER
INTE VARA 90°
BEROENDE AV
DEPONERINGSMASKINEN
OCH UTRYMME FÖR
UPPSTÄLLNING AV UTRUSTNING
10
STAMTUNNEL
40
ANTAGEN
PLANERINGSFÖRUTSÄTTNING
I LAYOUT D2
BEROENDE AV
DEPONERINGSMASKINEN
BETONGPLUGG
HUVUDALTERNATIV
B
3
3
A
ORTENS LUTNING
DEPONERINGSORT
4.2
x
PLATSBEROENDE, MIN 6,0 M
3
≥ 6
BEROENDE AV DEPONERINGSMASKIN
OCH EV RÄDDNINGSKAMMARE
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
Slutförvar för använt kärnbränsle
SKB - PROJEKTERING SLUTFÖRVAR
SWECOOS
2009-06-25
REFERENSUTFORMNING
FÖRVARSSOMRÅDE
MÅTT
1 : 500
9-C140-D-00-0001
REF: 9_147CD_00_0001.dgn
9_C140D_00_0001.dgn
PLOT:2009-06-25 15:09
A7

A8

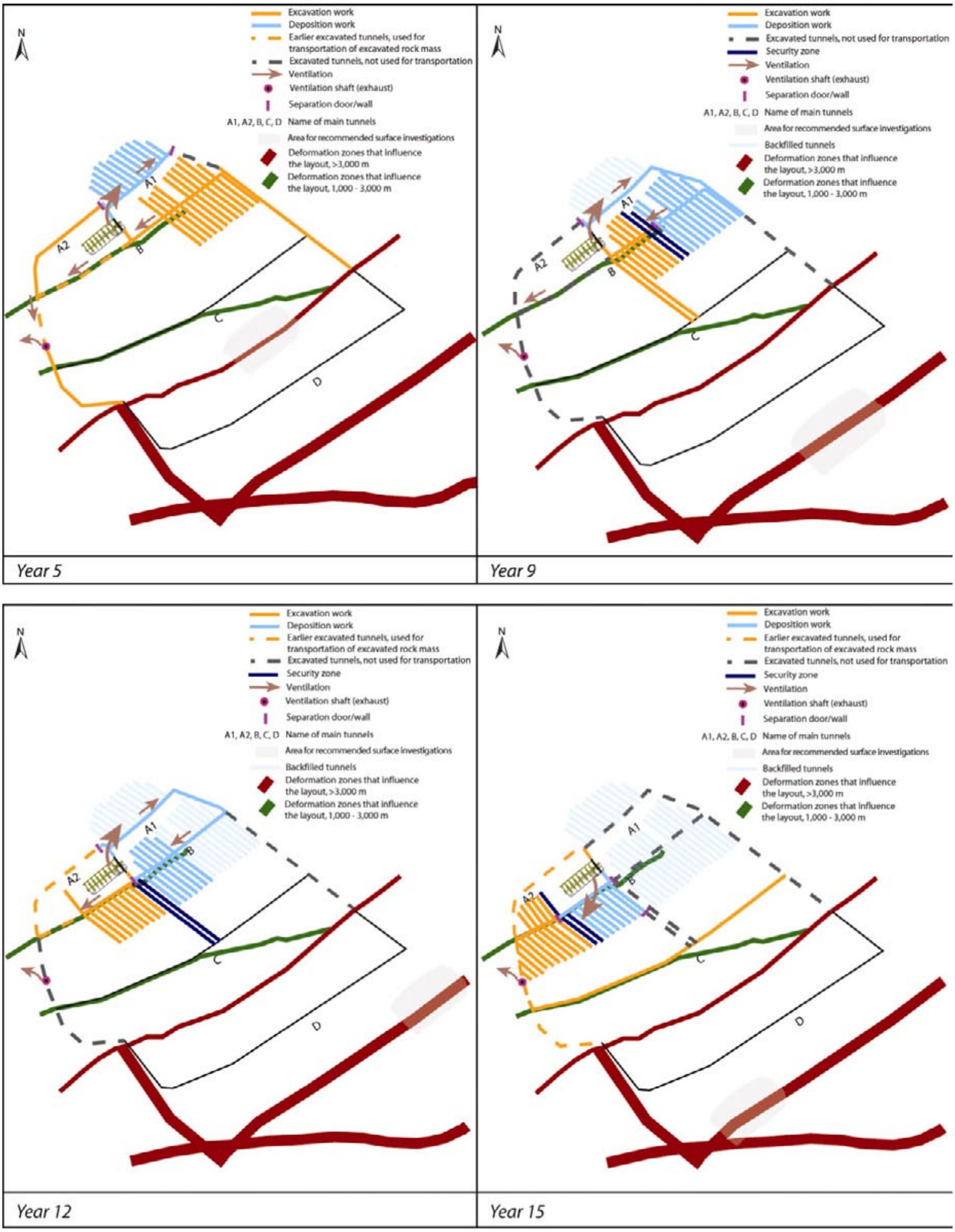


ORTENS LUTNING 1:100 MOT STAMTUNNELN
TEORETISKT TUNNELGOLV
∅ 1,75
1,63
1,25
1,93
~ 8,2
TEORETISK SEKTION
0,6
4,8
4,2
MIN 3,6
DEPONERINGSMASKINENS PROFIL 3 x 4,2
"VINGELMÅN"
VENTILATIONSTUB
BELYSNING
KABELKONSOLER
2
UTRYMNINGSVÄG
0,4
3
0,8
1,9
2,3
4,2
FÖRESKRIFTER
DEPONERINGSORTENS EXCENTRICITET FÅR EJ SPEGELVÄNDAS SETT FRÅN STAMTUNNELN
1:100
5 M
BET
ANT
ÄNDRINGEN AVSER
DATUM
SIGN
Slutförvar för använt kärnbränsle
SKB - PROJEKTERING SLUTFÖRVAR
UPPDRAG NR
RITAD
SWECOOS
GRANSKAD
DATUM
2009-06-25
GODKÄND
REFERENSUTFORMNING
FÖRVARSOMRÅDE
DEPONERINGSORTER OCH DEPONERINGSHÅL
MÅTT
SKALA
1 : 100
NUMMER
9-C140-D-00-0021
9_C140D_00_0021.dgn
PLOT:2009-06-25 15:14
REF: 9_140CAB00_0001.dgn

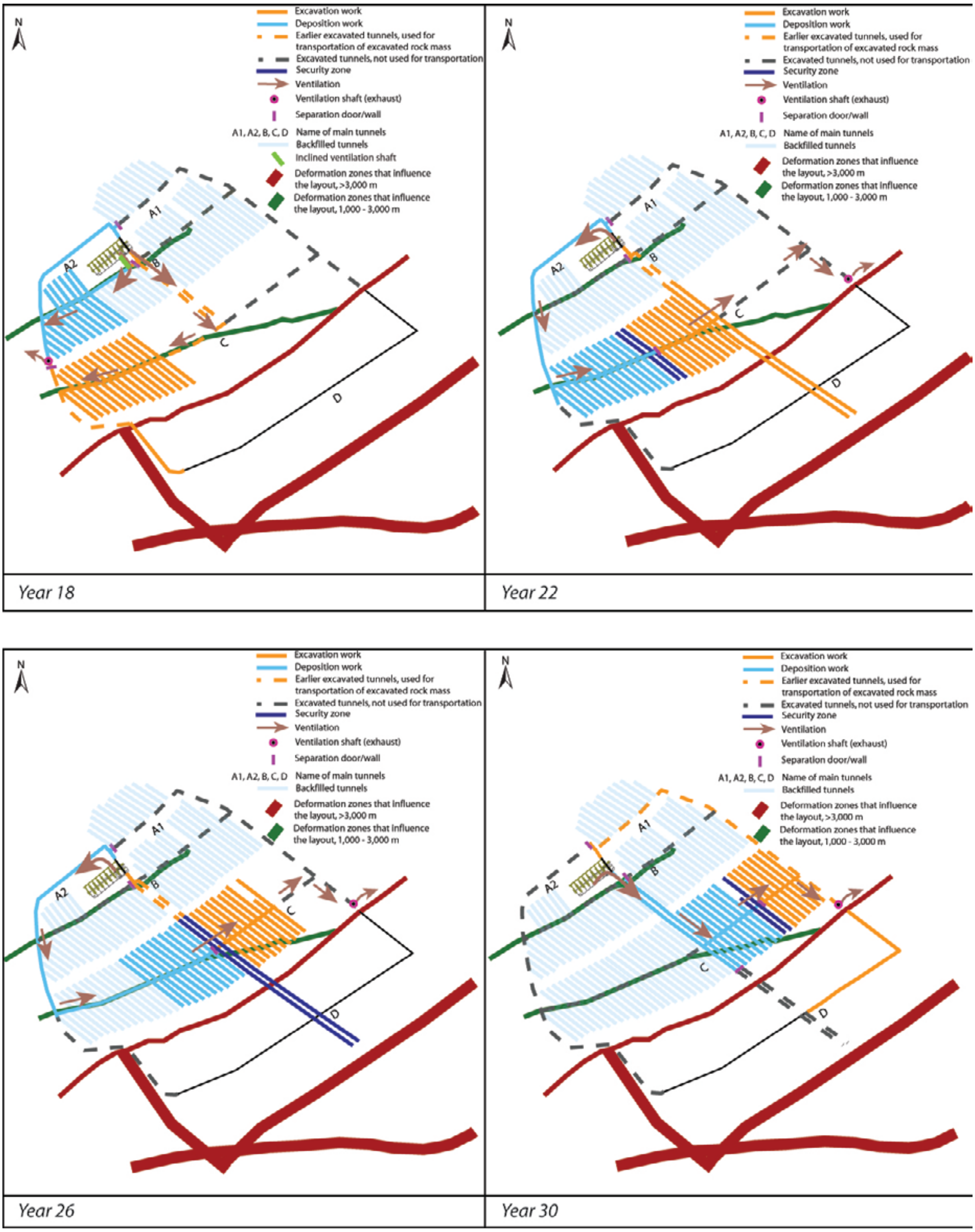
A development plan for construction and deposition



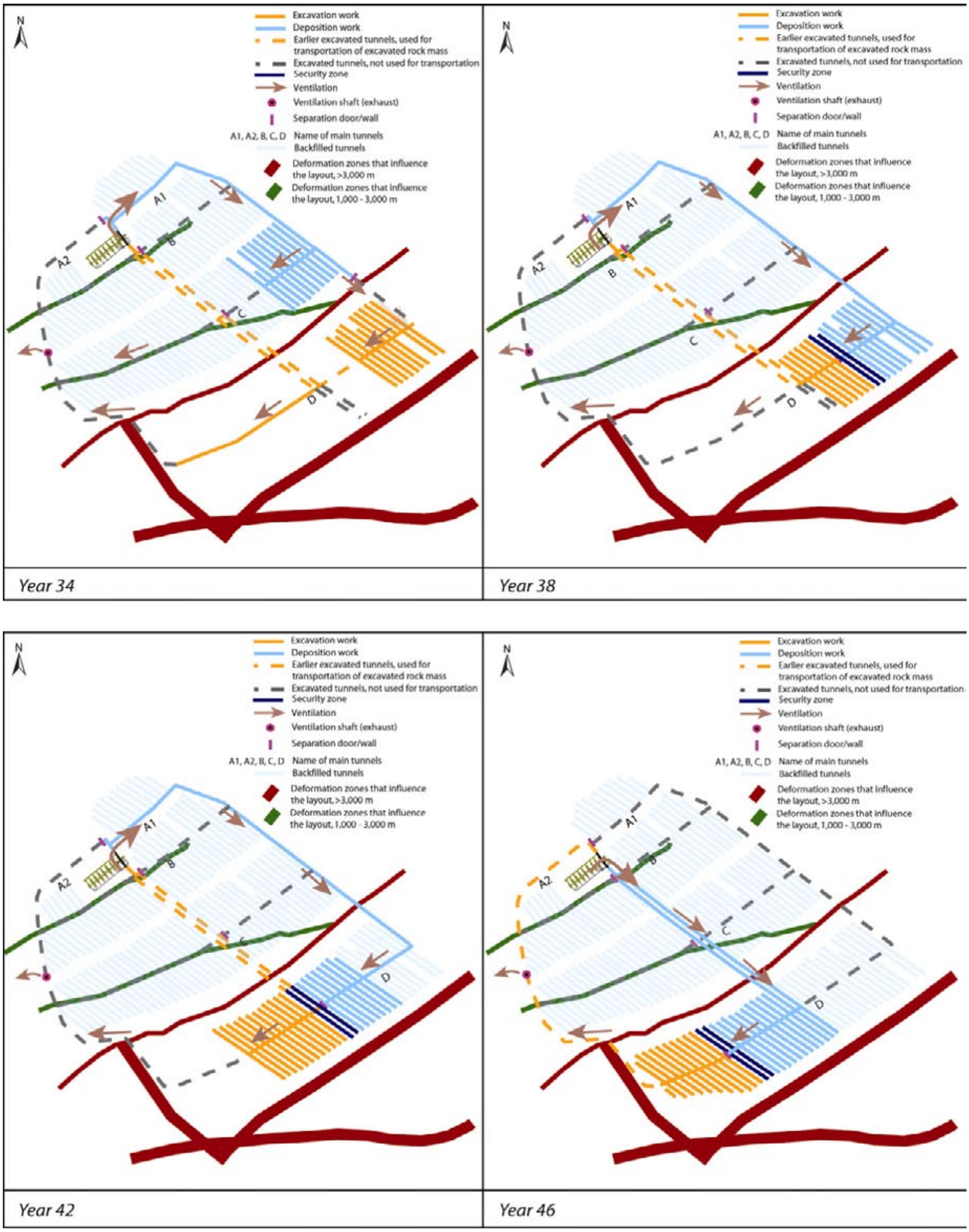
Construction schedule, year 0



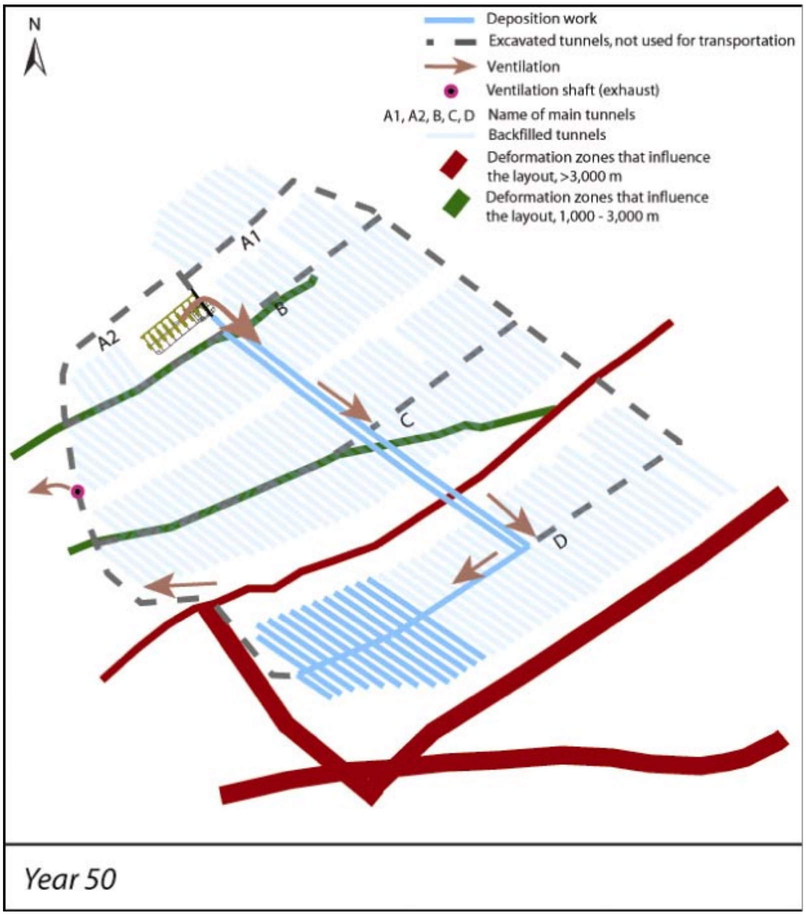
Construction schedule, year 5, 9 12 & 15.



Construction schedule, year 18, 22, 26 and 30.



Construction schedule, year 34, 38, 42 and 46.



Construction schedule, year 50.

Appendix C

An assessment of the potential loss of deposition-hole positions due to spalling

C. Spalling

The deposition holes at Forsmark will be located in Ground Type 1, which will be found primarily in the dominant rock types in RFM029 and RFM045. At the repository level the proportions of RFM029 and RFM045 is approximately 80% and 20%, respectively. The laboratory uniaxial compressive strength (UCS) of the main rock type in RFM045 (373 MPa) is significantly greater than the UCS of the main rock type in RFM029 (226 MPa). For the purposes of D2, only spalling analyse for the dominant rock type in RFM029 has been carried out since the layout is governed by findings from these analyses for the this weaker rock.

C.1 Definitions

Yielding of the rock mass around underground openings in hard rocks is a function of the in situ stress magnitudes and the characteristics of the rock mass, i.e. the intact rock strength and the fracture network . At low in situ stress magnitudes, the failure process is controlled by the continuity and distribution of the fracture network in the rock mass. However as in situ stress magnitudes increase, the failure process is dominated by new stress-induced fractures in intact rock growing parallel to the excavation boundary. This form of stress-induced progressive fracturing is generally referred to as spalling. Spalling may occur during excavation of the deposition holes and/or during heating of the rock mass once the waste has been placed. Once the waste package is placed, the buffer will take up water and generate a swelling pressure. This swelling pressure if developed before the heat increase is sufficient to induce spalling, may be adequate to suppress the development of the spalled zone. The potential for thermally induced spalling in deposition holes is assessed as part of the safety assessment, and is not further discussed here.

The development of spalling during the drilling of a deposition hole and from the application of thermal loading around a deposition hole was examined by /Andersson 2007/. The in situ experiment carried out by /Andersson/2007/ demonstrated that if spalling occurs during drilling it will be concentrated in the upper portion of the borehole when tangential stresses are concentrated in a similar manner to those illustrated in Figure C-1. This observation can be related to the distribution of tangential stress along the deposition hole caused by the interaction of the deposition tunnel and the deposition hole (Figure C-2). As shown by many researchers spalling occurs when the tangential stress on the boundary of the excavation reaches the spalling strength. As illustrated in Figure C-1, /Andersson 2007/ using the observed spalling and the calculated tangential stress showed that the spalling strength for Äspö Diorite ranged from 114 to 133 MPa, with a mean strength of 124 MPa.

C.2 Spalling assessment methodology

In order to evaluate the impact of spalling on the repository design two issues must be addressed: (1) will spalling occur, and (2) what will be the maximum depth of spalling should it occur. The methodology used to address these issues is described in /Martin and Christiansson 2009/. The three key parameters used in this methodology are (1) the in situ stress magnitudes and orientations, (2) the rock mass spalling strength, and (3) the relationship between stress magnitude and depth of spalling. The uncertainties associated with these parameters for the Forsmark spalling analyses are briefly discussed below.

C.2.1 In situ stress magnitudes and orientations

The in situ stresses for Forsmark are described in SDM Site and were established by integrating direct measurements, and indirect data and observations /Martin 2007/. /Martin 2007/ and /Glamheden et al. 2007/ reduced the uncertainty in the *in situ* state of stress using technical auditing of measurement results, statistical data analysis and numerical modelling. The lack of significant stress-induced damage to cores recovered from exploratory near vertical boreholes (microcracking or core disking) or borehole walls (borehole breakouts) to 1,000 m depth supports the notion that the in situ stress magnitudes do not exceed the elastic limit of the unloaded rock mass. This implies that stable vertical boreholes can be excavated at Forsmark to great depths.

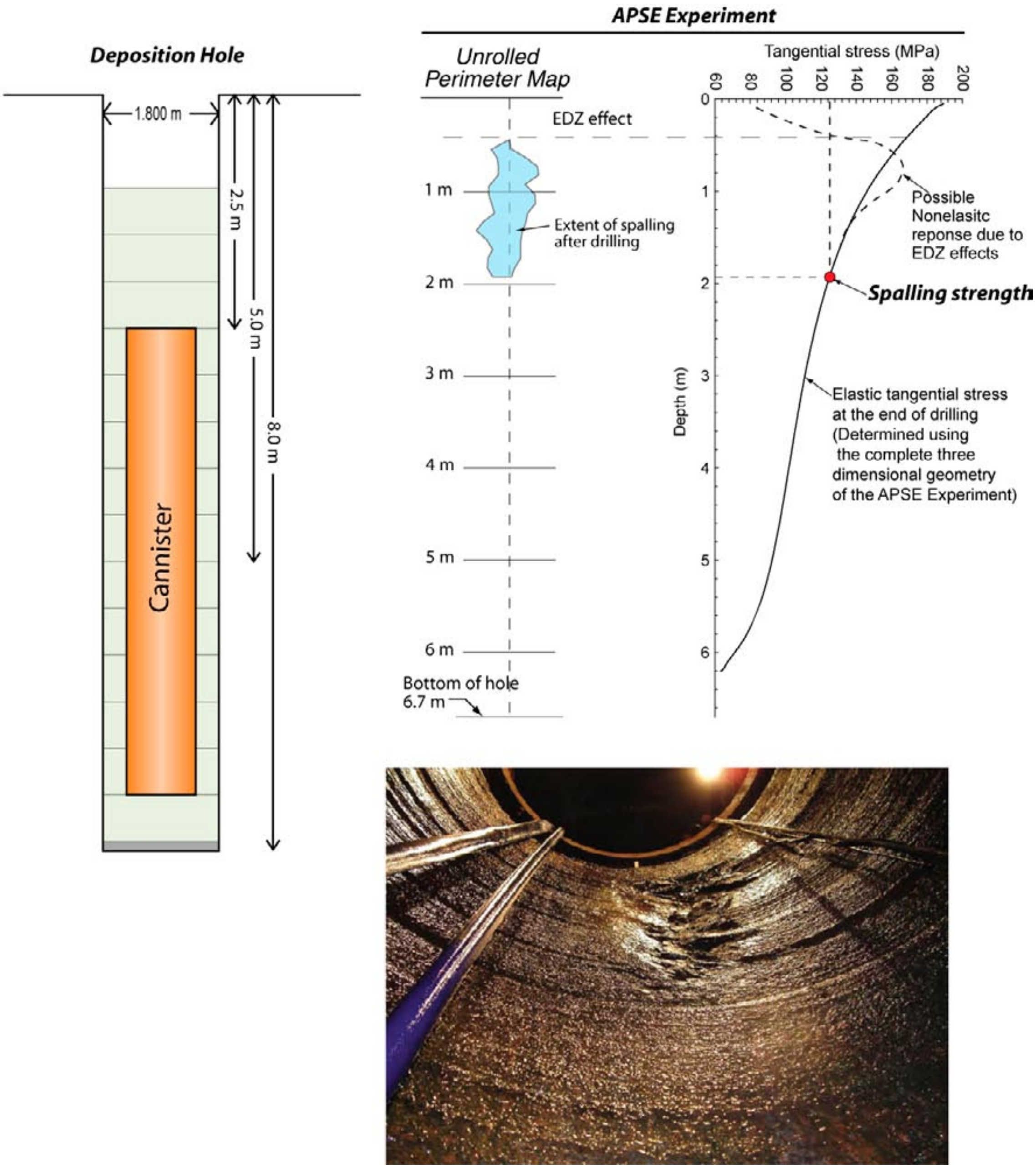


Figure C-1. *Tangential stress on the borehole used in the APSE Experiment and the measured stress-induced spalling at the end of excavation. The total induced stress that induced the spalling was composed of the far-field stress plus the excavation-induced stress.*

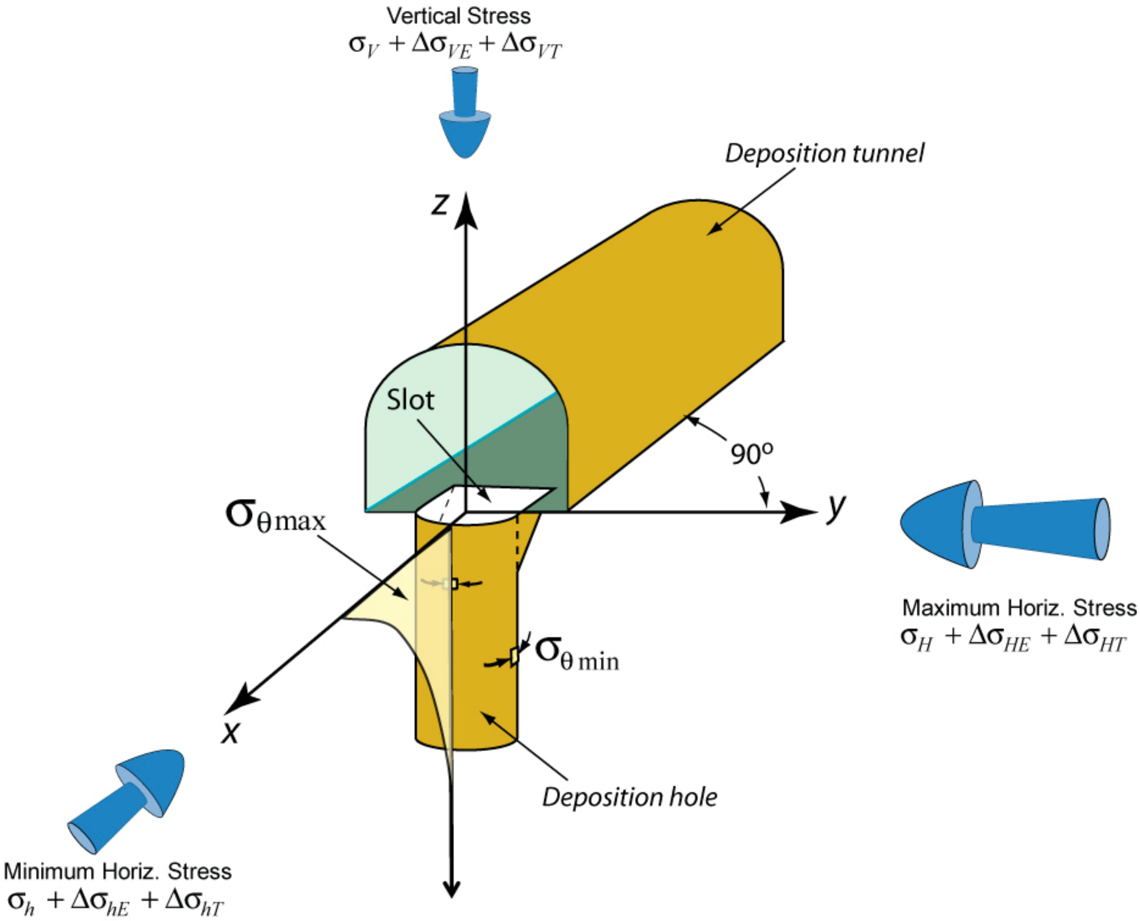


Figure C-2. *Tangential stress distribution on the boundary of a deposition hole. The elastic stresses actng on the deposition hole are a function of the far-field stress (σ), the excavation induced stress (ΔσE) and the thermally induced stress (ΔσT), and the geometry of the deposition tunnel.*

The orientation of the principal stresses and the magnitude of the vertical stress component of the stress tensor are judged to have the highest confidence at Forsmark. The basis for the confidence in the orientation is the conformity in the results between measuring methods and indirect observations at different scales, and its agreement with regional seismic studies. The confidence in the vertical stress magnitude is based on the agreement between measured values from both hydraulic fracturing and overcoring methods, and theoretical values based on the weight of the overlying rock cover. The uncertainty in the stresses at Forsmark resides in the magnitudes of the horizontal stresses. Consequently, three in situ stress models for Forsmark have been used in the design studies (Table C-1). The three models are summarised in SER and for purposes of this report are labelled:

(1) "most likely" case,

(2) "unlikely maximum" case and

(3) "unlikely minimum" case.

The orientation of the stress tensor and the vertical stress magnitudes are the same in all models and hence the differences in the models represent differences in the estimates of the horizontal stress magnitudes. The "most likely" model was developed by constraining stress measurement data with indirect observations made during the site investigations. The "most likely" model should not be considered as an "optimistic" model but as the model that best fit the data gathered during the site investigations. The "unlikely maximum" case is based entirely on horizontal stress magnitudes obtained from measurements carried out during the development of the Borre probe in the late 1970's. While these data are considered "low quality and suspect" they have been retained as an upper bound model. The "unlikely minimum case" was developed from the measurements made

Table C-1. Summary of the in situ horizontal stress models used to assess the potential for spalling. The vertical stress in all models is equivalent to the weight of overlying rock.

Parameter	Unit	Most Likely	Unlikely maximum	Unlikely minimum
σh min	MPa	20	27	8.6
σh average	MPa	25	35	10.2
σh max	MPa	30	43	11.8
σH min	MPa	34	50	21.6
σH average	MPa	40	56	22.7
σH max	MPa	46	62	23.8
Orient σH (average)	°	145	145	124
		k should lie between k= 1.4 to 2	k should lie between k= 1.4 to 2	

during the hydraulic fracturing program carried out during the site investigations. It is well known that hydraulic fracturing cannot be relied on to provide horizontal stress magnitudes when both horizontal stresses are greater than the vertical stress. While there is little confidence in this "unlikely minimum" model it has been retained to provide a lower bound model.

C.2.2 Rock mass spalling strength

The rock mass spalling strength can only be measured by conducting full-scale in situ tests. Such a test was carried out to determine the spalling strength for the 1.8-m-diameter deposition holes as part of the Äspö Pillar Stability Experiment (APSE, /Andersson 2007/). /Andersson 2007/ concluded that the rock mass spalling strength for Äspö diorite was 124 MPa which was nearly identical to the 120 MPa reported by /Read 2004/ for various tunnel diameters and shapes excavated in Lac du bonnet granite. /Martin and Christiansson 2009/ evaluated both of these in situ experiments and concluded that in the absence of in situ data the spalling strength could be estimated using the crack initiation stress from unconfined laboratory compression tests. /Diederichs et al. 2004/ also concluded that crack initiation stress from laboratory tests represented a lower bound estimate for the in situ spalling strength.

Figure C-3 shows the distribution of the crack initiation values normalised to the peak uniaxial strength obtained from the laboratory testing program carried out during the site investigation program for Forsmark. The 116 values range from 0.41 to 0.64 with a mean value of 0.53 (Figure C-3). The measured spalling strength from the APSE experiment is also shown for comparison. As suggest by /Diederichs et al. 2004/ Figure C-3 illustrates that the laboratory crack initiation stress provides a lower bound estimate for the rock mass spalling strength.

C.2.3 Depth of spalling

Once the stresses on the boundary of the excavation reach the rock mass spalling strength (Factor of safety for spalling =1) and spalling initiates, the severity of the hazard must be assessed, i.e. how deep will the spalling extend. The depth of spalling can be estimated using the empirical correlations described in /Martin and Christiansson 2009/. These data were compiled from published case histories in a wide range of rock mass conditions and in situ stresses. The results from the APSE experiment have been added to Figure C-4 and Figure C-4 shows that the empirical correlations predicted the depth of spalling for the APSE reasonably well. In Figure C-4 the depth of spalling is normalised to the tunnel radius and is measured from the centre of the tunnel. The expression for the depth of spalling given in Figure C-4 for approximately circular openings can be rewritten as:

$$S_d = a\left[0.5\frac{\sigma_\theta}{\sigma_{sm}} - 0.52\right] \qquad \text{(Eq. C-1)}$$

where the S_d is measured from the boundary of the tunnel. /Rojat et al. 2008/ used Equation C-1 to estimate the depth of failure for the Lötschberg base Tunnel in granites and gneiss and concluded that "good agreement was found between the observed and predicted".

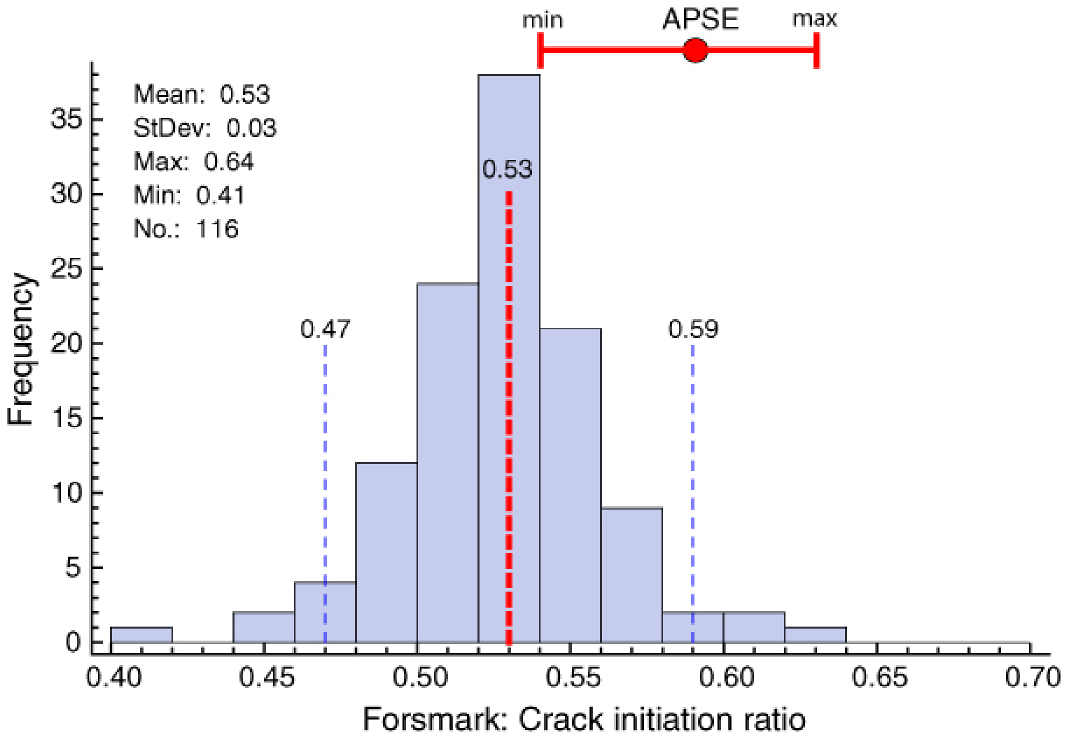


Figure C-3. *Crack initiation stress from laboratory uniaxial compression tests normalised by the peak uniaxial strength. The minimum, mean and maximum in situ spalling strength from the APSE experiment is also shown for comparison.*

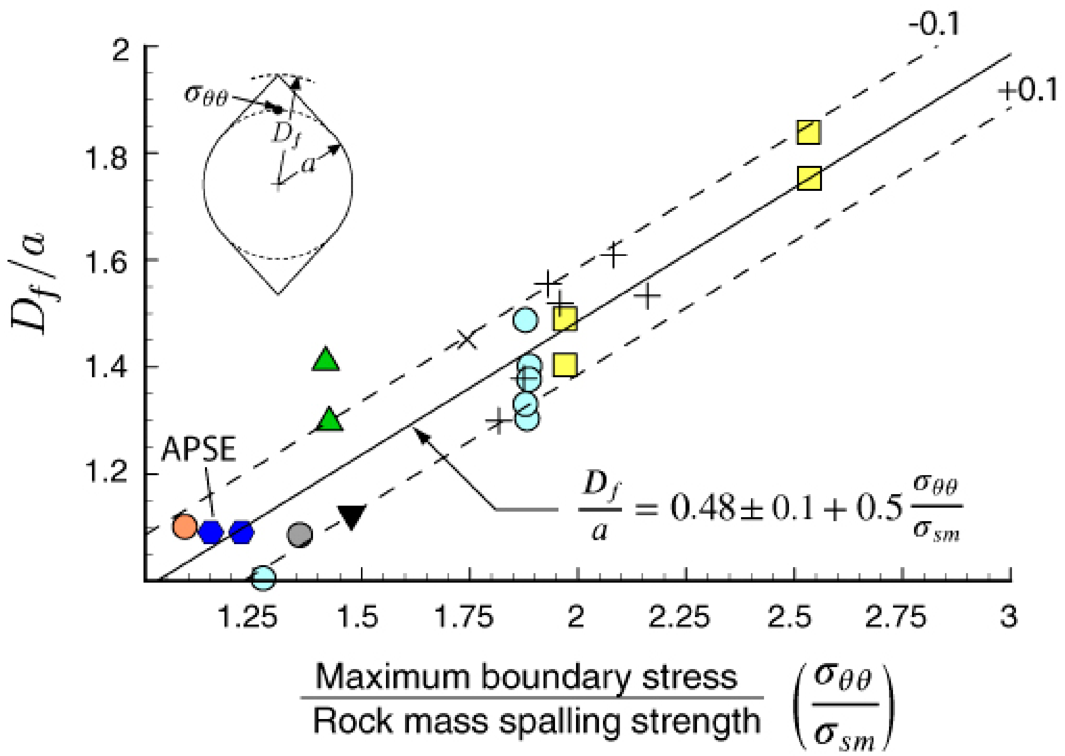


Figure C-4. *Empirical relation between depth of spalling and the calculated maximum elastic tangential boundary stress.*

C.2.4 Probabilistic analyses

The spalling potential for Forsmark was assessed using the probabilistic methodology outlined by /Martin and Christiansson 2009/ and summarised in Figure C-5. The methodology uses the two-dimensional plane strain Kirsch solution to establish the maximum tangential stress on a circular opening and to explore the sensitivity of the depth of spalling to the stress models. As shown in Figure C-2, the stresses acting on the circular deposition hole are a function of the far-field stress, the excavation induced stress and the thermally induced stress, and the shape of the deposition tunnel.

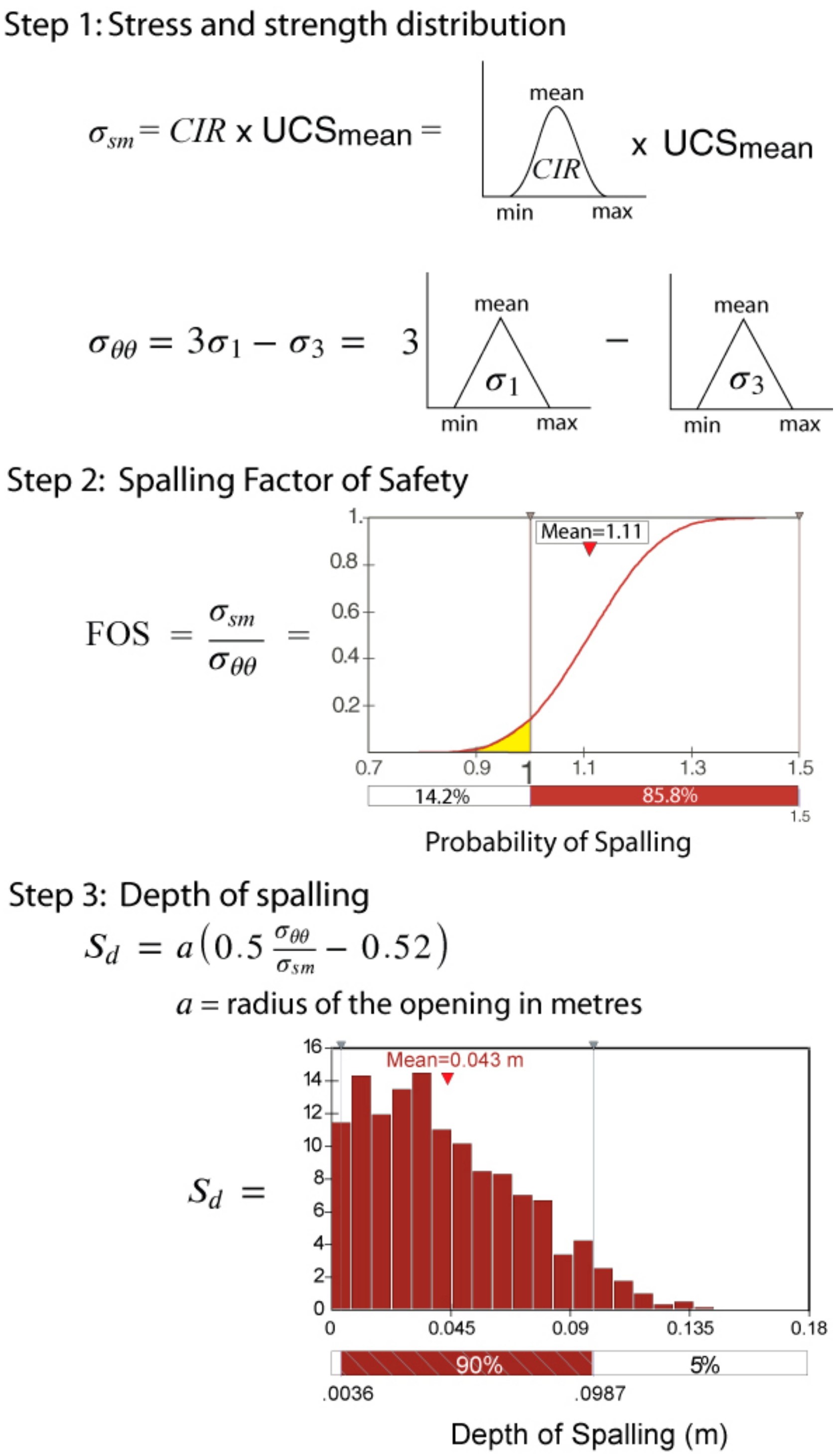


Figure C-5. *Overview of the methodology used to establish the probability of spalling and the depth of spalling, adapted from /Martin and Christiansson 2009/.*

When considering the elastic stresses on the boundary of a deposition hole, a three dimensional stress analyses is required to establish the maximum tangential stress magnitudes with deposition hole depth (Figure C-2). This 3D tangential stress was then normalised to the 2D tangential stress to establish a correction factor that was then used in the spalling probabilistic methodology.

C.3 Results from the spalling analyses

C.3.1 Three dimensional elastic stress analyses

Three dimensional elastic stress analyses were carried out for Forsmark with the deposition tunnel aligned perpendicular, 60 degree, 30 degree and parallel to the maximum horizontal stress. These analyses were carried out for the "most likely" and "unlikely maximum" in situ stress models. The maximum tangential stresses on the boundary of the deposition hole for each model are summarised Figure C-6. From Figure C-6 it is clear that when the deposition is tunnel aligned parallel to the maximum horizontal stress the maximum tangential stress concentration on the wall of the deposition hole is at a minimum. Also shown on Figure C-6 is the spalling strength used in design step D2. For the "most likely" stress model only the deposition tunnels aligned greater than 30 degrees to the maximum horizontal stress will produce tangential stress concentrations that are greater than the spalling strength and in these situations the spalling will occur above the top of the canister. Hence for this stress model the layout plan can utilise deposition tunnels that are aligned between and 0 and 30 degrees to the maximum horizontal stress. However for the "unlikely maximum" stress model the

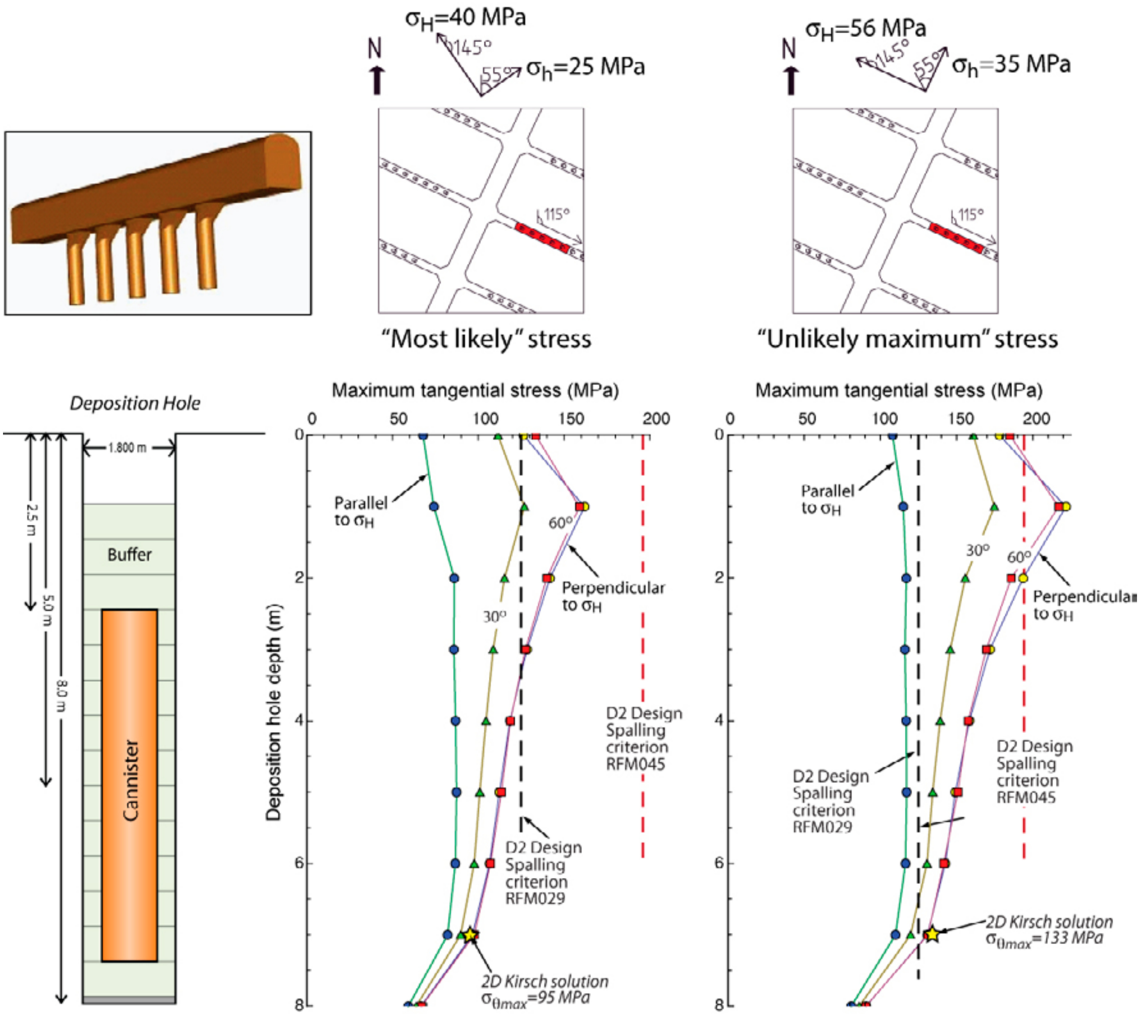


***Figure C-6.** Maximum tangential stress on the deposition hole as a function of the deposition tunnel orientation relative to the orientation of the maximum horizontal stress for the "most likely" and "unlikely maximum" stress models.*

tunnels must be aligned with the maximum horizontal stress to reduce the risk of spalling and should spalling occur it will occur over essentially the entire length of the canister (Figure C-6). From these analyses, the probabilistic methodology described in the previous section was used to establish the possible depth of spalling for:

1. The "most likely" stress model with the deposition tunnels aligned parallel and at 30 degree to the maximum horizontal stress, and
2. The "unlikely maximum" stress model with the deposition tunnels aligned parallel to the maximum horizontal stress.

The results from these probabilistic depth of spalling analyses are discussed in the following section.

C.3.2 Factor of safety for Spalling

The spalling factor of safety is used as a screening tool to assess if the potential for spalling is significant. The three cases identified above were analysed using the methodology given in /Martin and Christiansson 2008/ but incorporating the spalling strength based on the Forsmark laboratory crack initiation distribution described previously. Each depth was analysed using 5,000 simulations. Figure C-7 presents all the results from the factor of safety calculations at each 1 m depth down the deposition hole. Figure C-7 shows that some of the simulations for the deposition tunnel aligned 30 degree to the maximum horizontal stress using the "most likely" stress models and the deposition tunnel aligned parallel to the maximum horizontal stress using the "unlikely maximum" stress models indicate a potential for spalling. None of the simulations for the deposition tunnel aligned parallel to the maximum horizontal stress using the "most likely" stress models indicated a potential for spalling at any deposition hole depth.

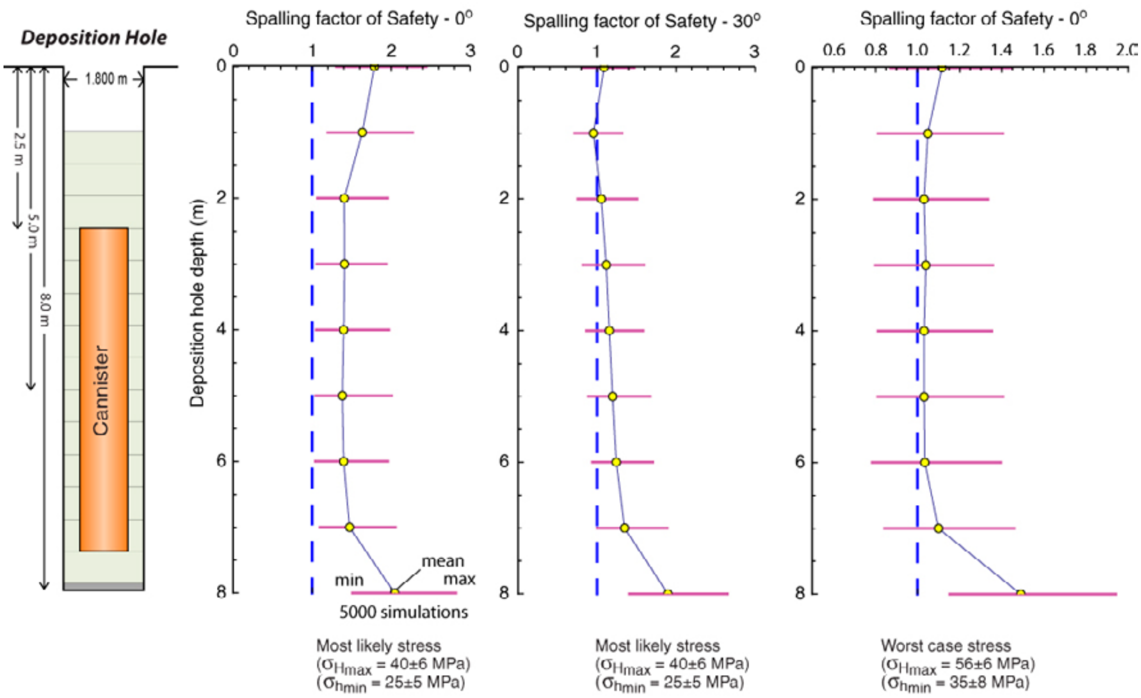


Figure C-7. *Factor of safety for spalling using the most likely and unlikely maximum stress models. The orientation of the deposition tunnel relative to the maximum horizontal stress is given. The range in the factor of safety was obtained from 5,000 simulations using the methodology outlined in /Martin and Christiansson 2009/.*

C.3.3 Depth of spalling

The depth of spalling was calculated using Equation C-1 for the simulations in Figure C-7 that had a factor of safety less than 1. The simulations for any given deposition hole depth resulted in a distribution of spalling depths illustrated in Figure C-8. These distributions are summarised using box-and-whisker plots in Figure C-9 and the depth of spalling criterion of 7 cm that was used for design D2. Figure C-9 illustrates that for the "most likely" stress model with the deposition tunnel oriented 30 degrees to the maximum horizontal stress, the depth of spalling only exceeds the 7 cm design criterion above the top of the canister position and that the median of the simulations at any depth never exceeds the design criterion. For the "unlikely maximum" stress model with the deposition tunnel oriented parallel to the maximum horizontal stress, only the outliers in the simulations (those simulations beyond the whiskers – 95th percentile) exceed the 7 cm design criterion. All the simulations for this "unlikely maximum" stress scenario indicate the median depth of spalling ranged between 1 and 2 cm over the deposition hole depth.

The analyses suggest that the depth of spalling may locally exceed the design criterion but that it is only the depth of spalling for the "unlikely maximum" stress scenario where spalling is likely to be encountered over the full depth of the deposition hole. However, even for this situation the depth of spalling is likely to remain below the design criterion at the end of excavation.

C.3.4 Loss of deposition holes due to spalling

A deposition hole overbreak of maximum 5 cm is considered to not have a detrimental impact to the performance of the compacted buffer blocks. However a depth of overbreak greater than 5 cm will require remedial measures to the overbreak geometry prior to buffer placement. Hence by this criterion deposition holes that produce 5 cm of spalling or less would not contribute to the loss of deposition holes. As shown in Figure C-9 the depth of spalling will vary with depth down the borehole, particularly for the "most likely" stress model oriented at 30 degrees to the maximum horizontal stress. To evaluate the number of deposition holes that could cause detrimental impact to the performance of the buffer, the number of deposition holes for depths of spalling ranging from 0 to 10 cm was evaluated for the two stress models. These results are given in Figure C-10 for differ-

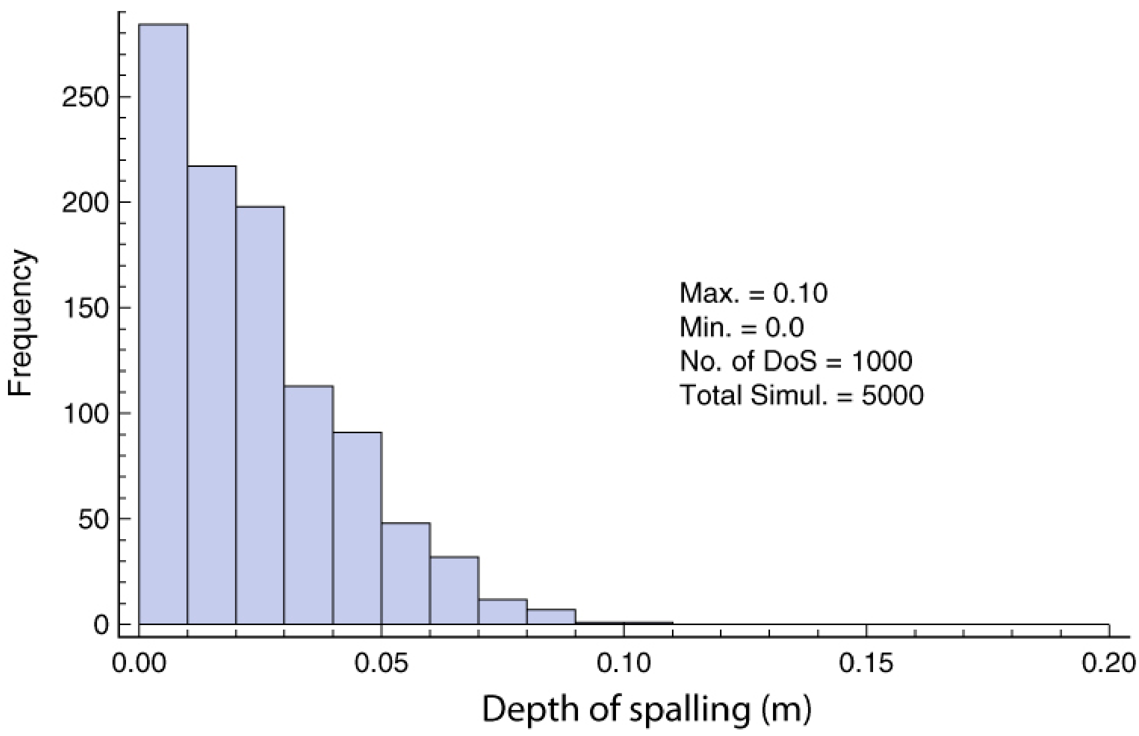


Figure C-8. *Example of the depth of spalling obtained using Equation 9-1 and the procedure outlined in /Martin and Christansson 2009/.*

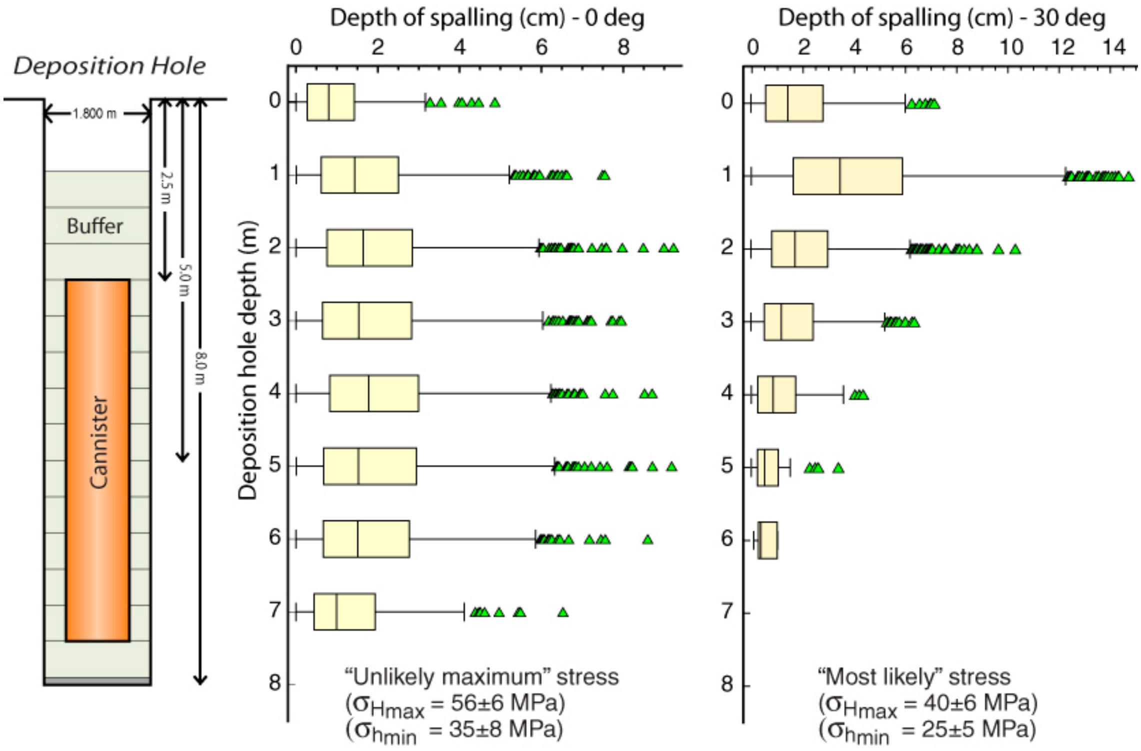
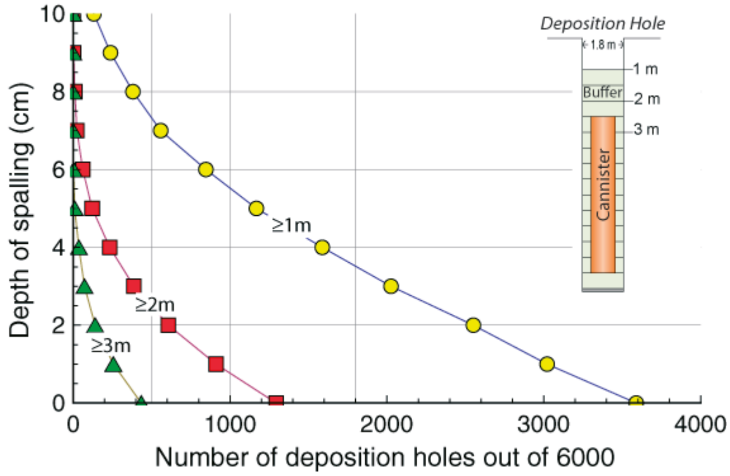


Figure C-9. *The depth of spalling calculated using the methodology given in /Martin and Christiansson 2009/ and expressed as box-and-whisker plots for the two cases identified in Figure C-7. The box captures 25th and 75th percentiles and the whiskers captures the 5th and 95th percentile. The green triangles represent the outliers beyond the whisker.*

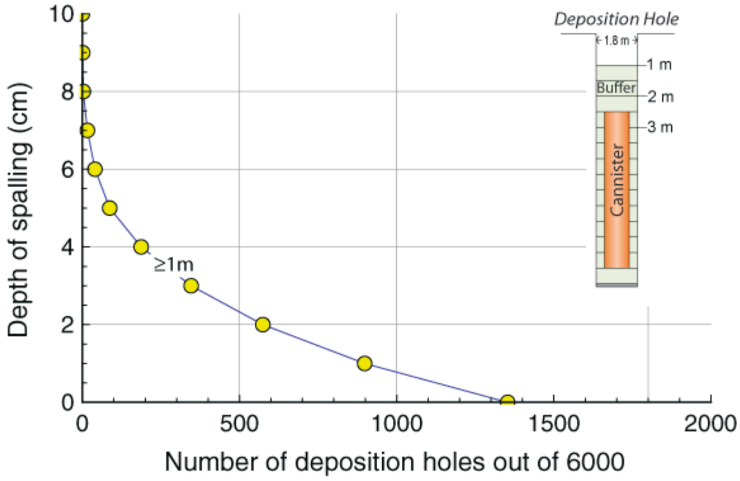
ent depths down the deposition hole. For example, using the "most likely" stress model, the number of deposition holes and the associated spalling depths were determined along the deposition hole at 1 m and below. This plot is noted as ≥1 m in Figure C-10 (a) and will have the maximum number of deposition holes with spalling since it evaluates all possible combinations ≥1 m depth. Because the tangential stresses varies with depth down the deposition hole, depths of 2 m and 3 m were also determined (see Figure C-10 (a)). As shown in Figure C-10 (a) the number of deposition holes with spalling below 2 m and 3 m decreases significantly because of the reduction in tangential stresses with deposition hole depth. However, because the tangential stress for the "unlikely maximum" stress model oriented parallel to the maximum horizontal stress remains essentially constant with depth there is no change in the number of deposition holes with spalling. It should be noted that based on the results of the APSE experiment, spalling is unlikely to be encountered in the deposition hole depth range of 0 to 1 m due to the effects of stress redistribution caused by the slot at the top of the hole and possible excavation-induced disturbed zone around the tunnel.

Figure C-10 b illustrates that the number of deposition holes with spalling for the "unlikely maximum" stress model at all depths is essentially the same for the "most likely" stress at depths ≥2 m. However at depths ≥3 m, the number of deposition holes with spalling (412) significantly reduces for the "most likely" stress model with the deposition tunnels oriented at 30 degrees to the maximum horizontal stress. This implies that if 5 cm of overbreak can be tolerated the number of deposition holes that exceed this criterion will be:

- approximately 1,000 for the "most likely" stress model with the deposition tunnels oriented at 30 degrees to the maximum horizontal stress, and
- approximately 100 for the "unlikely maximum" stress model with the deposition tunnels oriented parallel to the maximum horizontal stress.



a) Most likely stress model – deposition tunnel oriented 30 degrees to maximum horizontal stress



b) Unlikely maximum stress model – deposition tunnel oriented parallel to maximum horizontal stress

Figure C-10. *The number of deposition holes versus the depth of spalling calculated at different depths (≥1 m) along the deposition hole for the "most likely" and "unlikely maximum" stress model. The number of deposition holes for the "unlikely maximum" stress model is unchanged regardless of deposition hole depth.*

It should be noted that for the most likely stress model, no deposition holes will be lost if the deposition holes are aligned parallel to the maximum horizontal stress. Hence if the stress magnitudes at the repository level exceed the "most likely" stress model minor design changes in the layout for reference design could be used to mitigate the loss of deposition holes due to spalling.

C.4 Requirements during Detailed Design and construction

The analyses carried out for assessing the spalling hazard for design step D2 highlights the need for verifying the orientation of the maximum horizontal stress and the magnitudes of the maximum and minimum horizontal stress prior to finalizing the layout of the deposition tunnels. However, such verification can only be carried out when the repository excavation has reached relevant repository depths. /Martin et al. 1996/ showed that convergence measurements can be used as a reliable method for determining stress orientations and magnitudes in sparsely fractured crystalline rock. Such technology should be suitable for reducing the uncertainty in horizontal stress magnitudes and confirming the orientation of the maximum horizontal stress. In addition, once underground traditional overcoring techniques can also be used to verify the stress tensor. In the unlikely event that these verifications shows that the stress level is higher than suggested by the "most likely" stress, the assessment shows that this could be handled by e.g. changing the orientation of the deposition tunnels for the final design.

While the majority of the deposition holes will be placed in the dominant rock types there may be a need to utilize subordinate rock types. Additional laboratory UCS tests should be carried out during the next design stage to establish the crack initiation stress for these subordinate rock types that may be suitable for deposition holes.

For design Step D2, the rock mass spalling strength was established from laboratory tests. Once the repository level has been reached the APSE experiment should be repeated to establish the in situ rock mass spalling strength.

References – Appendix C

Andersson C J, 2007. Äspö Hard Rock Laboratory. Äspö Pillar Stability Experiment, Final report. Rock mass response to coupled mechanical thermal loading. SKB TR 07-01, Svensk Kärnbränslehantering AB.

Diederichs M S, Kaiser P K, Eberhard K E, 2004. Damage initiation and propagation in hard rock and influence of tunnelling induced stress rotation. Int. J. Rock Mech. Min. Sci. 41. 785–812.

Glamheden R, Fredriksson A, Röshoff, K, Karlsson, J, Hakami, H, Christiansson R, 2007. Rock mechanics Site descriptive modeling Forsmark Stage 2.2. SKB R-07-31, Svensk Kärnbränslehantering AB.

Martin C D, Chandler N A, Read R S, 1996. The role of convergence measurements in characterizing a rock mass. Canadian Geotechnical Journal, 1996, 33, 363–370.

Martin C D, 2007. Quantifying in situ stress magnitudes and orientations for Forsmark Design Step D2. SKB R-07-26, Svensk Kärnbränslehantering AB.

Martin C D, 2009. Estimating the potential for spalling around a deep nuclear waste repository in crystalline rock. Int. J. Mech. Min. Sci. A6:219–228.

Read R S, 2004. 20 years of excavation response studies at AECL's Underground Research Laboratory. Int. J Rock Mech 41.1251–1275.

Rojat F, Labiouse, V, Kaiser P K, Descoeudres F, 2008. Brittle rock failure in Steg Lateral Adit of the Lötschberg Base Tunnel. Rock Mechanics and Rock Engineering 2008 *in press*.