R-06-34

Final repository for spent nuclear fuel

Underground design Forsmark, Layout D1

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April 2006

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Förord

Denna rapport utgör en sammanställning av resultaten från den bergprojektering som utförts under skede D1 inom projekt Projektering Slutförvar, Djupförvarsprojektet, för platsen Forsmark. Motsvarande rapporter tas även fram för platserna Simpevarp och Laxemar.

Huvudsyftet med skede D1 är att svara på frågan om ett slutförvar kan inrymmas inom den anvisade platsen, men även att testa designmetodiken och ge en återkoppling till modelleringsprojektet.

Projekteringen har utförts av Ramböll Sverige AB i samarbete med underkonsulterna I&T Olsson AB, Golder Associates AB och Gridpoint Oy. För två avsnitt har Computer-aided Fluid Engineering AB och Prof. Derek Martin, University of Alberta genomfört utredningar i separata uppdrag åt SKB.

Projekteringen har genomförts i enlighet med den metodik som beskrivs i UDP (Underground Design Premises), SKB rapport R-04-60 och projekteringen har baserats på preliminära data från olika discipliner i platsmodelleringsprojektet. De preliminära indata som använts har sedan stämts av mot data i den slutliga platsbeskrivningen SDM v.1.2 och väsentliga avvikelser har inarbetats i projekteringen.

Projekteringsresultaten från respektive projekteringsfråga har presenterats av projektören vid presentationsmöten för SKB:s projekteringsledning och de granskare som SKB engagerat för den specifika frågan. Efter presentationsmötet har projektören färdigställt arbetsrapporterna för respektive fråga. Arbetsrapporterna har sedan granskats av SKB:s granskningsgrupp. Resultatet av granskningen har sammanställts i ett utlåtande som lämnats till projektören för åtgärd. I utlåtandet har projektören dokumenterat vilka kommentarer som åtgärdats och hur de åtgärdats.

Den 3D-layout med koordinatlistor för deponeringshål och tunnlar, som tagits fram inom uppdraget för att illustrera en möjlig utformning, har använts i PSE-analysen och hydromodelleringen av Open Repository, båda aktiviteter inom Djupförvarsprojektet.

Stockholm, 2006-04-24

Eva Widing

Summary

Introduction

This report comprises the design step D1 related to the underground design for a deep repository located at the Forsmark site. The design is based on the Site Descriptive Model Forsmark v1.2 /SKB 2005a/. All studies have been focussed at an area southeast of the Forsmark nuclear plant, which has been considered to be the most promising area for hosting the repository.

According to current plans for the Swedish nuclear programme the minimum required number of canister positions in the repository is determined to be 4,500. However, in order to accommodate the uncertainty in geological conditions and tentative future extensions of the nuclear plants operation period, SKB has for this study applied a required capacity of 6,000 canisters.

SKB has developed guidelines, entitled “Underground Design Premises” /SKB 2004a/, for the design of the repository, which further describes the methodology applied for the studies. From these guidelines the following basic objectives for the design step D1 are summarized:

- to determine whether the final repository can be accommodated within the studied site,
- to identify site-specific facility critical issues,
- to test and evaluate the design methodology described in /SKB 2004a/,
- to provide feedback to:
  - the design organisation regarding additional studies that needs to be done,
  - the site investigation and modelling organization regarding further investigations required,
  - the safety assessment team.

During the execution of the studies findings from other parallel ongoing studies and R&D work have initiated some deviations from /SKB 2004a/, which are further summarised and explained in Chapter 2 of the report.

Possible locations and preliminary assessment of the potential to accommodate the repository

The possible locations for a tentative deep repository are analysed in Chapter 3 of the report. The most promising area for the repository (denoted “priority site”) has been defined by SKB to be located southeast of the Forsmark nuclear plant and northwest of the gently dipping deformation zone ZFMNE00A2. Preferably the repository should be located southeast of the inlet canal for cooling water close to the nuclear power plant. The studied area is shown in Figure 3-1 of the report.

Regarding the repository depth, present knowledge acquired from the site investigations indicates that it is possible to locate the repository at all stipulated depths according to /SKB 2004a/, that is between 400 m and 700 m depth. However, from the initial site investigations it has been concluded that the rock stresses within the Forsmark area are

high, which implies that a shallow depth will be preferable. Since the site investigation results so far have not indicated any major differences in other properties of the rock mass between 400 m and 700 m depth, SKB decided that the design work should focus on repository levels of 400 m and 500 m. A shallow depth is also of obvious advantage from both economic and environmental points of view.

The preliminary assessment made in Chapter 3 clearly demonstrates that the repository can be accommodated within the "priority site". The potential to accommodate the repository is essentially the same for both 400 m and 500 m depths.

Design of deposition areas

The design of the deposition areas is reported in Chapter 4, which includes the design of layout features for all tunnels and deposition holes, orientation of tunnels, calculation of anticipated loss of deposition holes due to the applied design criteria given in /SKB 2004a/ as well as a recommendation on repository depth.

For design step D1 tunnel geometries and dimensions were recommended to be in accordance with Layout E /SKB 2002/, which also indicates a distance between deposition tunnels of 40 m, and a distance between deposition holes of 6 m. Even though the very favourable thermal properties of the rock mass in Forsmark would allow for shorter distances (approximately 5.5 m) between deposition holes, the Layout E recommendations remained unaltered in this report.

The studies of the orientation of deposition tunnels clearly reveal that all factors except the horizontal stresses are of minor importance for the orientation. All deposition tunnels have consequently been oriented parallel to the major horizontal stress, and the main- and transport tunnels have, where possible, been located in a skew direction to the major horizontal stress in order to minimise the risk of rock spalling. The selected design thus results in an insignificant risk of spalling for all tunnels.

The performed analysis of loss of deposition holes reveals that the only factor of importance is the applied design criteria given in /SKB 2004a/ for the minimum required distance from a deposition hole to stochastically determined fractures. The calculated loss of deposition holes was 9%, and is independent of depth.

The executed analyses summarized above indicate no significant difference between the 400 m and 500 m levels. Consequently the 400 m level was recommended as reference alternative for the layout studies and for the safety assessment studies.

Layout studies

In Chapter 5 the layout studies are reported, and two alternative layouts for each repository level at 400 and 500 m depths have been prepared. The layout studies were based on findings reported in previous chapters, and all presented layouts are designed for a minimum of 6,000 canisters, including allowance for the calculated loss of deposition holes. The layout also provides for a separate area for initial operation of approximately 200–400 canisters, which are included in the design capacity of 6,000 canisters.

The prepared layouts include two alternative locations for the surface facilities and the position where the access ramp meets daylight. One site for the surface facility is located close to the current SFR office building, and the other site is located at the residential area southeast of the nuclear power plant (named “Infarten”). Apart from the location of the access ramp and location of the surface facilities, all presented layouts are quite similar for the two studied repository levels.

For the recommended layout, assuming the “Infarten” alternative and a repository depth of 400 m (named the base layout), the anticipated volume for the underground facilities is approximately 2 million m^3 (excluding deposition holes), including 55 km of tunnels and 6,210 available canister positions (including allowance for a loss of deposition holes of 9%).

Identification of passages through deformation zones

Studies of identified passages through deformation zones are presented in Chapter 6.

The studies concluded that no major problems are expected during tunnelling through the deterministically determined deformation zones, and that only standard grouting and rock support methods would be required. However, extensive probe drilling, grouting and special excavation requirements and rock support are expected to be needed when the ramp and shafts pass through sub-horizontal near-surface fracture zones (approximately in the upper 200 m).

Seepage and hydrogeological situation around the repository

Chapter 7 of the report deals with the seepage and the hydrogeological situation into and around the repository with respect to the distance of influence and the salinity (TDS).

Both analytical and numerical methods have been utilised for the analyses. For the anticipated seepage into the repository both calculation methods indicate small amounts of leakage, and typical total seepage estimates for the whole repository level (excluding ramp and shafts) is calculated to be in the range 1–4 l/s depending on the applied sealing levels (corresponding to an hydraulic conductivity of the grouted zone, K_t, of 1×10^{-9} m/s or 1×10^{-7} m/s). The access ramp and shafts will in the upper parts (approximately the upper 200 m) pass water bearing sub-horizontal fracture zones with high hydraulic conductivity, which will render seepage values of the same magnitude as mentioned above for the repository level. The total inflow to the repository is thus expected to be approximately 2–8 l/s for an open repository, depending on the applied sealing level.

The small quantity of seepage also implies that the distance of influence regarding the groundwater level will be limited, and the numerical calculations indicate almost no discernable impact at all at the surface.

However, for the analysis regarding the salinity, the analytical and numerical methods provide contradictory results. Whereas the numerical model indicates a low probability of an increased salinity, the analytical method indicates a high probability but a low salinity due to the impervious rock mass.

Assessment of rock grouting need

Assessment of the rock grouting need is reported in Chapter 8. The assessment of rock grouting need included the design of grouting procedures and an estimation of the grout quantity. The estimated total grout quantity (including grout hole filling) injected into the rock mass for the repository is given in the table below. Due to uncertainties in the underlying parameters, variations in the calculated quantities have been estimated. It is assumed that the work is based on a standard pre-grouting technique and that cement based grouts are used.

	Quantity (m^3)
Total grout quantity for the repository Sealing level 1: $K_t = 1\times10^{-7}$ m/s	60 to 210
Total grout quantity for the repository Sealing level 2: $K_t = 1\times10^{-9}$ m/s	620 to 1,550
Grout quantity in deposition tunnels Sealing level 2: $K_t = 1\times10^{-9}$ m/s	460 to 1,120

The grout quantity for sealing level 1 corresponds to grouting of the subhorizontal fracture zones only.

It is important that the pH value in the rock mass around the repository, in the KBS-3 concept, is not too high due to the function of the bentonite buffer. In the safety analysis it is assumed that grout with a pH value < 11 is used. In order to comply with this assumption, a preliminary low pH grout was proposed by SKB, and the grout was implemented as an alternative grout in the designed grouting procedures.

Assessment of rock support need

In Chapter 9 an assessment of the rock support need is presented. The assessment of rock support need included the design of rock support and an estimation of the quantity of different rock support elements. The estimated total quantity of bolts and shotcrete for the repository is given in the table below. Due to uncertainties in the underlying parameters, variations in the calculated quantities have been estimated.

	Quantity
Total number of rock bolts for the repository	49,000 to 69,000
Total shotcrete area for the repository (m^2)	190,000
Number of rock bolts in deposition tunnels	18,000 to 30,000

The rock quality is generally very good and no major stability problems are expected. The rock support is installed primarily to ensure that no isolated blocks or smaller pieces of rock fall out. Most of the rock reinforcement will be installed as minimum support, including spot bolting and a 50 mm thickness of steel fibre reinforced shotcrete in the roof. This rock support will be installed irrespective of the rock quality.

Technical risk assessment

A technical risk assessment has been performed and is dealt with in Chapter 10 of the report. The main objective of the technical risk assessment was to quantify an answer to the question "Can the repository be accommodated within the 'priority site'?". A model considering variations in different factors, which influence the available area for the repository (such as the dip of deformation zones), was developed and an analysis was carried out using Monte Carlo simulations.

Due to the high rock stresses at the Forsmark area, a "what-if scenario" regarding even higher stress levels was also analysed more in detail.

The most important results from the technical risk assessment are:

- There is a very high (approximately 99%) probability that 6,000 canisters can be accommodated within the studied area (the "priority site").
- The most important factors identified to have the largest impact on the uncertainty of calculated results, and given in order of priority were:
  - The length of deformation zone ZFMNE0060 (whether the length of the zone is > 3 km or not).
  - The dip of deformation zone ZFMNE0060.

Other factors were considered to be of minor importance.

- Rock stress modelling for rock domain RFM029, which is utilised as input for the estimate of the risk of spalling in deposition tunnels and deposition holes, are of major importance. If using a conservative estimate of the major horizontal stress (based on old data), horizontal stresses up to 65 MPa might be prevailing at repository depth. This condition would in turn result in a high probability of spalling and the loss of deposition holes may accordingly be high.

Sammanfattning

Inledning

Föreliggande rapport beskriver bergprojekteringen i projekteringssteg D1, avseende ett djupförvar i Forsmark. Projekteringen baseras på den platsbeskrivande modellen version 1.2 /SKB 2005a/.

Fokus för projekteringen har varit området sydöst om Forsmarks kärnkraftverk. Detta område har valts av SKB, baserat på resultat från de inledande platsundersökningarna.

Enligt gällande plan för det svenska kärnkraftprogrammet, krävs en kapacitet för djupförvaret motsvarande 4 500 kapselpositioner. Med hänsyn till rådande osäkerhet i geologiska förhållanden samt möjlig förlängning av kärnkraftverkens drifttid, har SKB beslutat att projekteringen ska baseras på ett kapacitetskrav motsvarande 6 000 kapsel-positioner.

För projekteringen har SKB utarbetat projekteringsanvisningar /SKB 2004a/, vilka i detalj beskriver krav på projekteringsmetodiken avseende bl a indata, verifiering och dokumentation av olika projekteringsmoment. I /SKB 2004a/ anges också de huvudsakliga målen för projekteringssteg D1:

- Att fastslå huruvida djupförvaret ryms inom det studerade området.
- Att identifiera platsspecifika kritiska faktorer.
- Att testa och utvärdera den projekteringsmetodik som beskrivs i /SKB 2004a/.
- Att ge återkoppling till:
  - SKB:s projekteringsorganisation avseende behovet av ytterligare utredningar.
  - Platsorganisationen i Forsmark avseende behovet av ytterligare undersökningar.
  - SKB:s organisation för säkerhetsanalys.

Under den pågående projekteringen har parallella utredningar och utvecklingsarbete inom SKB resulterat i ett antal avsteg från projekteringsanvisningarna /SKB 2004a/, vilket behandlas i kapitel 2 av rapporten.

Möjliga lägen och djup inom platsen samt platsens potential att rymma förvaret

Möjliga områden för ett djupförvar beskrivs i kapitel 3. Det prioriterade området har angetts av SKB till sydöst om Forsmarks kärnkraftverk och nordväst om den flacka deformationszonen ZFMNE00A2. Om möjligt bör även djupförvaret lokaliseras sydöst om kärnkraftsverkets kylvattenkanal (se även figur 3-1).

Baserat på den kunskap som erhållits från de inledande platsundersökningarna och de krav som anges i /SKB 2004a/, gjordes bedömningen att det är möjligt att lokalisera djupförvaret på alla djup inom intervallet 400–700 m. Då bergspänningarna inom Forsmarksområdet är höga, bör djupförvaret lokaliseras så ytligt som möjligt. Då inga större skillnader i bergmassans egenskaper mellan 400 m och 700 m har indikerats av platsundersökningarna, beslutade SKB att fokus för projekteringen skulle vara djupen 400 m och 500 m. Ett ytligt lokaliserat djupförvar är också fördelaktigt med avseende på kostnad och miljöpåverkan.

En preliminär bedömning av möjligheten att djupförvaret ryms inom det prioriterade området har utförts i kapitel 3. Slutsatsen blev att förvaret med stor marginal kan rymmas och att potentialen att rymma djupförvaret bedömdes vara ungefär lika för de båda djupen 400 m och 500 m.

Utformning av deponeringsområden

Utformningen av deponeringsområdena redovisas i kapitel 4. Detta kapitel redovisar utformning av tunnlar och deponeringshål, orientering av deponeringstunnlar, bedömning av bortfall av deponeringshål samt en rekommendation avseende förvarsdjupet.

I projekteringssteg D1 användes geometrier och dimensioner för tunnlar och deponeringshål enligt Layout E /SKB 2002/, vilken även anger ett avstånd mellan deponeringstunnlar på 40 m och 6 m mellan deponeringshål. Avståndet 6 m mellan deponeringshålen har använts i projekteringssteg D1, trots att utförda beräkningar visade på att ett kortare avstånd kan vara möjligt (ca 5,5 m) pga bergmassans gynnsamma termiska egenskaper i Forsmarksområdet.

Analyserna av lämplig orientering av deponeringstunnlarna visade tydligt att den enda faktorn som har någon större betydelse för valet av orientering är den höga horisontalspänningen och den därmed förenade risken för spjälkning av bergmassan i tunnlarna. Med hänsyn till bergspänningarna har deponeringstunnlarna därför orienterats parallellt med den största horisontalspänningen och stam- och transporttunnlar har om möjligt orienterats med en spetsig vinkel mot den största horisontella spänningen. Med denna tunnelorientering bedöms risken för spjälkning av bergmassan i tunnlarna vara liten.

Den utförda analysen av bortfallet av deponeringshål visade att den enda faktorn av betydelse för bortfallet av deponeringshål är kriterierna i /SKB 2004a/ avseende avståndet mellan deponeringshål och stokastiskt bestämda sprickor. Det beräknade bortfallet av deponeringshål blev 9% samt var oberoende av djupet.

De utförda analyserna som beskrivs ovan, visade inte på någon skillnad i resultat mellan djupen 400 m och 500 m. Djupet 400 m rekommenderas därför som huvudalternativ för det fortsatta layoutarbetet och säkerhetsanalyserna.

Layoutstudier

I kapitel 5 redovisas två alternativa layouter för varje djup, 400 m och 500 m. Layoutstudierna baserades på resultat från de analyser, som beskrivits i föregående avsnitt, samt kravet på en kapacitet för 6 000 kapslar med beaktande av ett bortfall av deponeringshål på 9%. Layouterna möjliggör deponering av ca 200–400 kapslar för den inledande driften. Kapslarna för den inledande driften är inkluderade i kapacitetskravet på 6 000 kapslar.

De framtagna layouterna inkluderar två olika alternativ avseende ovanmarksanläggningen samt tillfartsrampens utformning. Ett område för ovanmarksanläggen är beläget vid kontorsbyggnaden för SFR och det andra området, kallat ”Infarten”, är beläget vid bostadsområdet sydöst om kärnkraftverket. I övrigt är de framtagna layouterna relativt lika för de båda djupen 400 m och 500 m.

Den layout som förordas (kallad ”the base layout”) är belägen på 400 m djup med läge ”Infarten” som ovanmarksalternativ. Denna layout omfattar ca 2 miljoner kubikmeter bergschakt (exklusive deponeringshål), 55 km tunnlar och 6 210 möjliga kapselpositioner (med beaktande av ett bortfall av deponeringshål på 9%).

Identifiering av passager genom deformationszoner

Analyser av passager genom deformationszoner presenteras i kapitel 6.

Slutsatsen från analyserna är att inga större problem förväntas vid tunneldrivningen genom deformationszonerna på förvarsdjup och att konventionella injekterings- och förstärkningsmetoder kan tillämpas. Vid passagen av ytliga (0–200 m djup) sub-horsiontella sprickzoner, som kan vara mycket vattenförande, krävs en mer omfattande sonderingsborrning och injekteringsinsats samt en anpassad tunneldrivning och förstärkning. Dessa sub-horisontella sprickzoner passeras vid byggandet av rampen samt schakten.

Inläckning och hydrogeologisk situation runt förvaret

I kapitel 7 redovisas analyser av möjligt inläckage av grundvatten till förvaret samt den hydrogeologiska situationen runt förvaret med avseende på influensavstånd och salthalt (TDS).

Både numeriska och analytiska metoder har använts för analyserna. Samstämmiga resultat avseende inläckning och influensavstånd erhölls med båda metoderna. När det gäller inläckaget till förvaret, erhölls ett inläckage i storleksordningen 1–4 l/s till djupförvaret (exklusive ramp och schakt) beroende på vilken täthet som erhålls i bergmassan runt tunnlarna efter injektering. I de utförda analyserna användes två tätningsnivåer, motsvarande en konduktivitet på den injekterade zonen, K_t, på 1×10^{-9} m/s och 1×10^{-7} m/s. Inläckaget till rampen och schakten bedöms vid passagen av de ytliga (0–200 m djup) vattenförande sub-horisontella sprickzonerna ge ett tillskott i samma storleksordning som inläckaget på förvarsdjupet. Det totala inläckaget bedöms således bli ca 2–8 l/s för ett öppet fullt utbyggt förvar.

Då inläckaget till förvaret bedöms bli litet, förväntas influensavståndet avseende grundvattenytan också bli litet med en obetydlig ytpåverkan som följd.

Motstridiga resultat erhölls däremot vid analyserna av salthalten. Den numeriska metoden resulterade i en liten sannolikhet för en förhöjd salthalt runt djupförvaret, medan den analytiska metoden resulterade i en stor sannolikhet men en låg salthalt.

Uppskattning av tätningsinsats

Uppskattningen av tätningsinsats redovisas i kapitel 8, där design av injekteringsmetodik för olika anläggningsdelar samt en bedömning av mängden injekteringsmedel behandlas. Bedömningen av mängden injekteringsmedel (inklusive hålfyllnad) för djupförvaret redovisas i nedanstående tabell. På grund av osäkerheter avseende injekteringsbrukets spridning i bergmassan, redovisas ett intervall för mängden injekteringsmedel. För bedömningen av mängden injekteringsmedel har det antagits att en konventionell injekteringsteknik med cementbaserade injekteringsmedel används.

	Mängd (m^3)
Total mängd injekteringsmedel för djupförvaret Tätningsnivå 1: $K_t = 1 \times 10^{-7}$ m/s	60–210
Total mängd injekteringsmedel för djupförvaret Tätningsnivå 2: $K_t = 1 \times 10^{-9}$ m/s	620–1 550
Mängd injekteringsmedel i deponeringstunnlar Tätningsnivå 2: $K_t = 1 \times 10^{-9}$ m/s	460–1 120

Mängden injekteringsmedel för tätningsnivå 1 motsvarar att endast de ytliga sub-horisontella sprickzonerna injekteras.

För ett djupförvar baserat på KBS-3 konceptet är det viktigt att pH-värdet i bergmassan runt förvaret inte blir för högt, vilket kan försämra bentonitbuffertens funktion. För säkerhetsanalyserna har det därför antagits att injekteringsbruk med ett pH < 11 används. För detta ändamål har SKB tillhandahållit en sammansättning för ett preliminärt injekteringsbruk med ett lägre pH. Detta injekteringsbruk har inarbetats som ett alternativt bruk i den framtagna injekteringsmetodiken.

Bedömning av förstärkningsinsats

En bedömning av förstärkningsinsats redovisas i kapitel 9. Bedömningen av förstärkningsinsats omfattande design av bergförstärkning för olika anläggningsdelar samt en bedömning av mängder för olika förstärkningselement. På grund av osäkerheter i omfattningen av förstärkningsinsats redovisas ett intervall för mängden förstärkning. Bedömningen av mängden förstärkning redovisas i den nedanstående tabellen.

	Mängd
Bergbultar, totalt för djupförvaret (st)	49 000–69 000
Sprutbetongarea, totalt för djupförvaret (m^2)	190 000
Bergbultar, deponeringstunnlar (st)	18 000–30 000

Bergkvaliteten är generellt mycket bra och inga större stabilitetsproblem förväntas, vilket innebär att bergförstärkningen installeras huvudsakligen för att förhindra utfall av enstaka block. Denna förstärkning installeras som minimiförstärkning och består av selektiva bergbultar och 50 mm fiberarmerad sprutbetong i taket på tunnlarna. Denna förstärkning installeras oberoende av bergkvaliteten.

Teknisk risk bedömning

En teknisk riskbedömning redovisas i kapitel 10. Det huvudsakliga syftet med riskbedömningen var att svara på frågan ”Kan förvaret rymmas inom det prioriterade området i Forsmark?”. En analys av effekten av olika faktorers variation (t ex variationen i deformationszonernas stupning) på det möjliga antalet kapselpositioner utfördes med Monte Carlo simulering.

På grund av de höga bergspänningarna inom Forsmarksområdet, analyserades också effekten av ytterligare högre spänningar.

De viktigaste slutsatserna från den tekniska riskbedömningen var att:

- Sannolikheten är mycket hög (ca 99%) att 6 000 kapselpositioner kan rymmas inom det prioriterade området i Forsmark.
- De faktorer som har störst inverkan på osäkerheten avseende om förvaret ryms eller inte är i prioritetsordning:
  - Längden på deformationszonen ZFMNE0060 (om längden på zonen är > 3 km eller inte).
  - Stupningen på deformationszonen ZFMNE0060.

Övriga faktorer har endast en mindre betydelse.

- Bedömningen av bergspänningarna inom bergdomän RFM029 har stor betydelse för bedömningen av risken för spjälkning av bergmassan runt tunnlar och deponeringshål. Om en konservativ bedömning av den största horisontalspänningen används (baserat på tidigare spänningsmätningar i Forsmarksområdet), kan spänningar på upp till 65 MPa vara möjliga på förvarsdjupet. Denna spänningsnivå resulterar i en hög sannolikhet att spjälkning av bergmassan kan inträffa och bortfallet av deponeringshål kan således förväntas bli stort.

Contents

1 Introduction

1.1 Objectives

SKB is currently planning for the construction of a final repository for disposal of spent nuclear fuel from the Swedish nuclear power plants. Geological investigations are ongoing at the municipalities of Oskarshamn and Östhammar. This design study has been carried out by a design team, including Ramböll Sweden AB, Rambøll Denmark A/S, Golder Associates AB, Gridpoint Oy and I&T Olsson AB, to meet the goals for design step D1 of a final repository at the Forsmark site.

SKB's guiding principles are to contribute to a safe radiation environment by protecting the environment and human health in both the short and long term perspective. SKB's objective is to conduct all works in strict observance of all statutory and regulatory requirements, and to recognize environmental awareness, high quality and cost-effectiveness.

During the site investigation phases the general objectives of the design work for a final repository are to:

- Prepare a facility description with a proposed layout for the final repository facility's surface and underground parts as a part of an application for concession according to applicable Swedish laws. The description shall present baseline data for the constructability, technical risks, costs, environmental impact and reliability/effectiveness. The underground layout will be based on information from the Complete Site Investigations (CSI) phase and serves as a basis for the long term Safety Assessment made in support to the application to build the final repository.
- Provide a basis for the Environmental Impact Assessment (EIA) and consultation regarding the site of the final repository facility's surface and underground parts. This includes proposed ultimate locations of ramp and shafts, and a description of the assessed environmental impact of construction and operation.
- Outline the design work for the final repository facility in adequate detail in order to satisfy the fundamental conditions for the forthcoming detailed design and preparation of documents for the construction phase.

SKB has developed guidelines entitled "Underground Design Premises" (UDP) /SKB 2004a/ for the design of the repository, and from these guidelines the following main objectives of rock engineering during the design step D1 can be summarized:

- determine whether the final repository can be accommodated within the studied site,
- identify site-specific facility critical issues and provide feedback to:
  - the design organisation regarding additional studies that need to be done,
  - the site investigation and modelling organization regarding further investigations, required,
  - the safety assessment team.
- provide illustrative tentative layouts for public consultations as required by Swedish environmental laws, comprising:
  - the location of surface facilities,
  - the location and extent of underground facilities,
  - baseline data for the environmental impact assessment.

- provide prerequisites for Preliminary Safety Evaluation (PSE) regarding:
  - theoretical extent of deposition areas,
  - estimation of the quantity of grouting, rockbolts and other artificial materials.
- prepare supporting documentation for the preliminary facility description,
- test and evaluate the design methodology described in /SKB 2004a/.

1.2 Strategy

The site investigations for the final repository started in 2002 and are scheduled to continue until 2007. The design procedures will proceed in parallel steps as results from the investigations are analysed and reported. Consequently, the design of the final repository will be developed in steps as the knowledge of underground conditions increase.

The design procedure is further described in Table 1-1.

This report comprises the design step D1, which is developed based primary on the investigation phase Initial Site Investigations (ISI), which later will be followed by the design step D2 based on the Complete Site Investigations (CSI). In design step D1 three different sites for the repository, Simpevarp, Forsmark and Laxemar, are investigated. After completing design step D2 the most suitable site will be selected for the application for concession as stipulated by the environmental laws and regulations of Sweden.

In design step D1 the overall focus of the studies is concentrated on the following key issues:

- To identify suitable areas for the repository within the studied site, and to provide input for the parallel studies whether the selected site can fulfil the safety requirements.
- To confirm that the site is large enough to accommodate the required size of a final repository.
- To test the developed design method in Underground Design Premises /SKB 2004a/.

A secondary objective, however not included in this report is:

- To perform a first study to implement environmental requirements on actual site conditions.

Table 1-1. Final Repository Project during the site investigation phase – relationships between different stages, design steps etc.

Final Repository Project during the site investigation phase (SI)				
Stage in Site Investigation (SI)	**Initial site investigation (ISI)**		**Complete site investigation (CSI)**	
Step in SI	1.1	1.2	2.1	2.2
Model version	1.1	1.2	2.1	2.2
Design step	D0	D1	D2	
Output of the design work in the Final Repository Project	Sketches of the surface facility (internal study material)	Preliminary facility description, Layout D1	Facility description, Layout D2	

The site investigation data are submitted in consecutive batches ("data freezes") and each part is evaluated and assessed into a site descriptive model (SDM). However, in order to gain time the design team has worked in close co-operation with the site investigation organization and modelling teams in order to establish preliminary results to be used for the design, i.e. before the publishing of the SDM. The preliminary results provided by each working group within the Site Descriptive Modelling team are later compared to the approved /SKB 2005a/. The possible risk that preliminary model information data might be modified, and consequently require revision of various design tasks, is acknowledged by SKB for the design step D1.

The working strategy for the design team to partly use reports that are not fully reviewed and approved by experts, and partly use not yet fully verified preliminary information, calls for thorough planning and management, frequent meetings and an open attitude between modellers and designers. This process is documented through Minutes of Meetings. Deviations between preliminary and final results in the /SKB 2005a/ are summarised in Chapter 2, Table 2-1. The consequences of changed parameter values are finally evaluated from the perspective how it would influence the final results of the design work carried out. If change in data is not unfavourable to the overall objectives of the design step D1, the analysis is not revised.

The UDP /SKB 2004a/ defines several design tasks for various technical issues (c.f. Section 2.1), and after each task a seminar has been arranged for presentation and discussion of results and for decisions on the prerequisites for future design tasks.

All reporting has been reviewed by external experts, who also have participated in the presentations made by the design team, with the objective to obtain a quick response and an opportunity for direct comments on presented findings. Within a few weeks after each presentation the design team submitted their task report to be reviewed by the engaged experts. At submission of the final report a final review of the completed report was performed.

1.3 Design Methodology

The design methodology adopted for this study is in detail described in the UDP (Underground Design Premises) /SKB 2004a/, which includes the necessary instructions for the design team to execute the design work. The methodology stipulates a stepwise progress of the work intercepted by meetings for decisions on the continuing design tasks. A more detailed description of the design tasks and the design methodology logical framework is given in Section 2.1.

1.4 Organisation

The design work has been carried out by an external design team performing the day-to-day work and a SKB representative as Project Manager. The Project Manager has been supported by various expertises within SKB as well as by independent reviewers (external resources). Coordination with other parts of the Final Repository Project, such as for example site investigations, site modelling and environmental impact studies, has been administrated by the project management.

The design team was organised with the objective of having resources for the different disciplines involved in the design tasks such as rock mechanics, hydrogeology, DFN-analyses, risk assessment, rock engineering and 3D-CAD design. The following individuals from Ramböll and other companies have contributed to the design work:

Ramböll:

- Martin Brantberger: Project leader, rock engineering, assessment of grouting and rock support, technical risk assessment, editor of reports.
- Anders Zetterqvist: Layout and design studies, CAD-operator (2D/3D).
- Torben Arnbjerg-Nielsen, Pernille Finn: Sensitivity analysis and technical risk assessment.
- Håkan Sandstedt: QA, technical review and review of English language.
- Peder Thorsager: Technical support rock mechanics.
- Per-Lennart Karlsson: Review of English language.

I & T Olsson: Tommy Olsson: Hydrogeological analyses.

Golder Associates:

- Nils Outters: DFN-analyses.
- Anders Fredrikssson: Technical review of DFN-analyses, hydrogeological- and rock mechanical analyses.

Gridpoint Oy: Pauli Syrjänen and Petteri Somervuori: Rock mechanical analyses.

Paul Summers: Review of English language.

The design work has been carried out with support of systems for quality assurance from Ramböll. These support systems are in accordance with SS-EN ISO 9001:2000.

1.5 Definitions and abbrevations

1.5.1 Abbreviations

Abbreviations used are explained below.

CSI	Complete site investigation. CSI is a stage during the site investigation phase.
ISI	Initial site investigation. ISI is a stage during the site investigation phase.
DFN	Discrete fracture network (stochastic distribution).
PSE	Preliminary safety assessment.
SDM	Site descriptive model.
SDM v1.2	Preliminary site description Forsmark area – version 1.2. /SKB 2005a/.
SI	The site investigation phase comprises the construction and detailed characterization phase and includes the required period for the authorities to process the site application.
UDP	The document “Underground Design Premises, Edition D1/1”.

1.5.2 General

Definitions for general terms are given below.

Client	SKB Project Manager for the Final Repository Project is Client for this Study.
Stage	A clearly defined part of a phase. *The site investigation phase includes the stages ISI, CSI and Application Review.*
Independent reviewer	Resource contracted by SKB for independent review of the project results.
Candidate area	Area within a municipality which has been judged in the feasibility studies to contain possible site(s) for a final repository.
Layout	The spatial disposition of the constituent parts.
Site	A prioritized part of a candidate area, i.e. the area required to accommodate with good margin a final repository and its immediate environs, roughly 5–10 km^2 /SKB 2001/.
Final Repository Project	The project including all site investigations and other activities ending at submission of the application for concession.
Design	All the work of preparing system- and construction-documents including a site description.
Design coordinator	Unit within SKB that is responsible for execution and coordination of the design of the final repository system. The design coordinator is unit TU.
Designer	Resource that executes a defined design assignment.
Safety assessment	Evaluation of long–term post closure safety.
Investigations	Measurements, surveys, samplings and tests aimed at determining properties and mechanisms. *In SI, this refers to the measurements, surveys, samplings and tests that are carried out in the field and that comprise a basis for the site description.*

1.5.3 Parts

Different parts are defined below (see also Figure 1-1 and 1-2)

Hard rock facility	The facilities below ground for the final repository.
Buffer	Diffusion barrier of bentonite surrounding the canister.
Central area	The part of the facility below ground in which caverns for operation and maintenance are located, e.g. storage and workshop cavern, elevator cavern, ventilation cavern, etc.

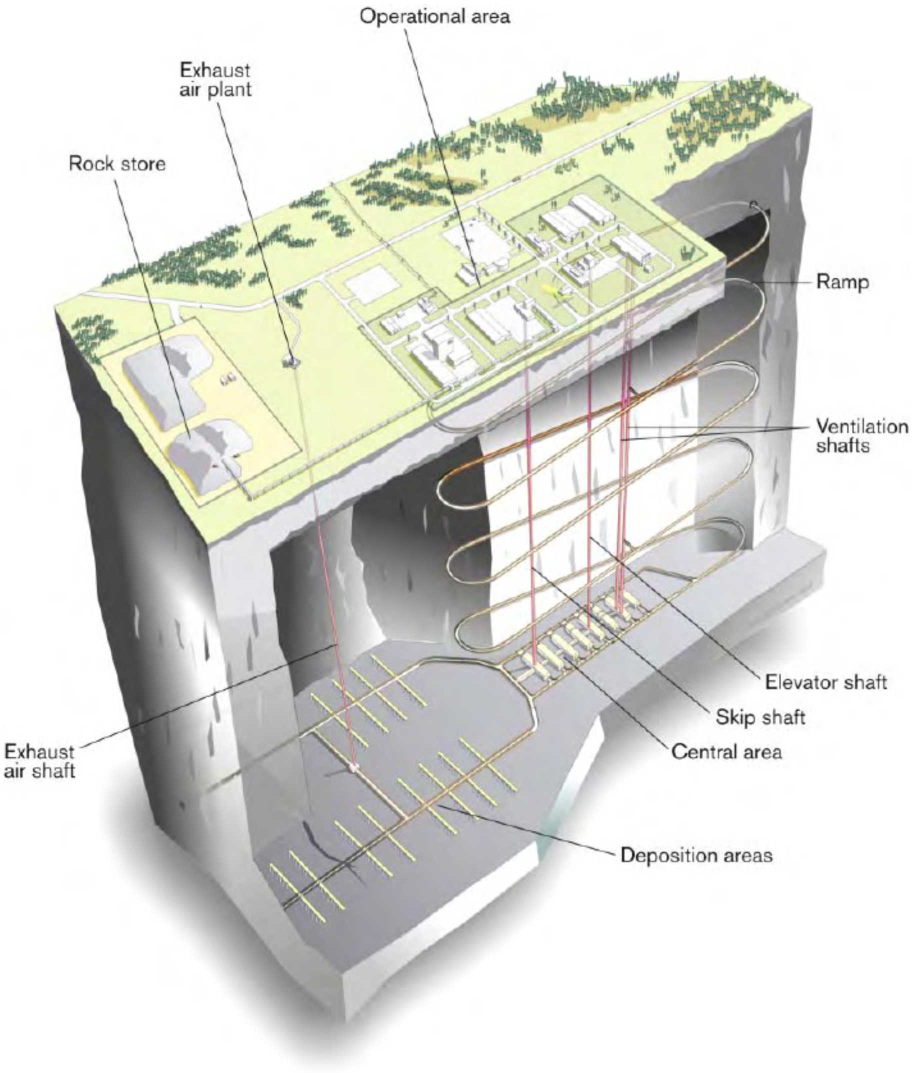


Figure 1-1. *3D-illustration of surface and underground facilities.*

Deposition area	The part of the hard rock facility in which canister deposition will take place. The deposition area includes main tunnels, deposition tunnels, deposition holes, and the rock mass immediately surrounding these openings.
Final repository	Final repository for spent nuclear fuel designed according to the KBS-3 method. The reference design is KBS-3V, with vertical deposition of canisters beneath the tunnel floor.
Final repository facility	The final repository and the facility parts that are required to construct, operate and seal the final repository. Can be roughly subdivided into a surface part and an underground part.
Surface part of final repository facility	The surface part comprises facilities above ground for the construction and operation of the final repository.

Underground part of final repository facility	The underground part comprises ramp – shafts – transport tunnels, central area, deposition areas, technical systems and furnishings under ground.
Temporary plug	Facility part that is used during the construction and operating phases to temporarily separate or seal various underground openings in the hard rock facility. Temporary plugs normally consist of reinforced concrete structures.
Canister	Load bearing steel container with copper shell in which spent nuclear fuel is placed for deposition.
Permanent plug	Facility part that is used to permanently separate or seal various underground openings in the hard rock facility.
Backfill	Backfill refers to the material that is placed in deposition tunnels and the rock caverns in the central area as deposition proceeds.
Backfilling	Backfilling refers to the activity.

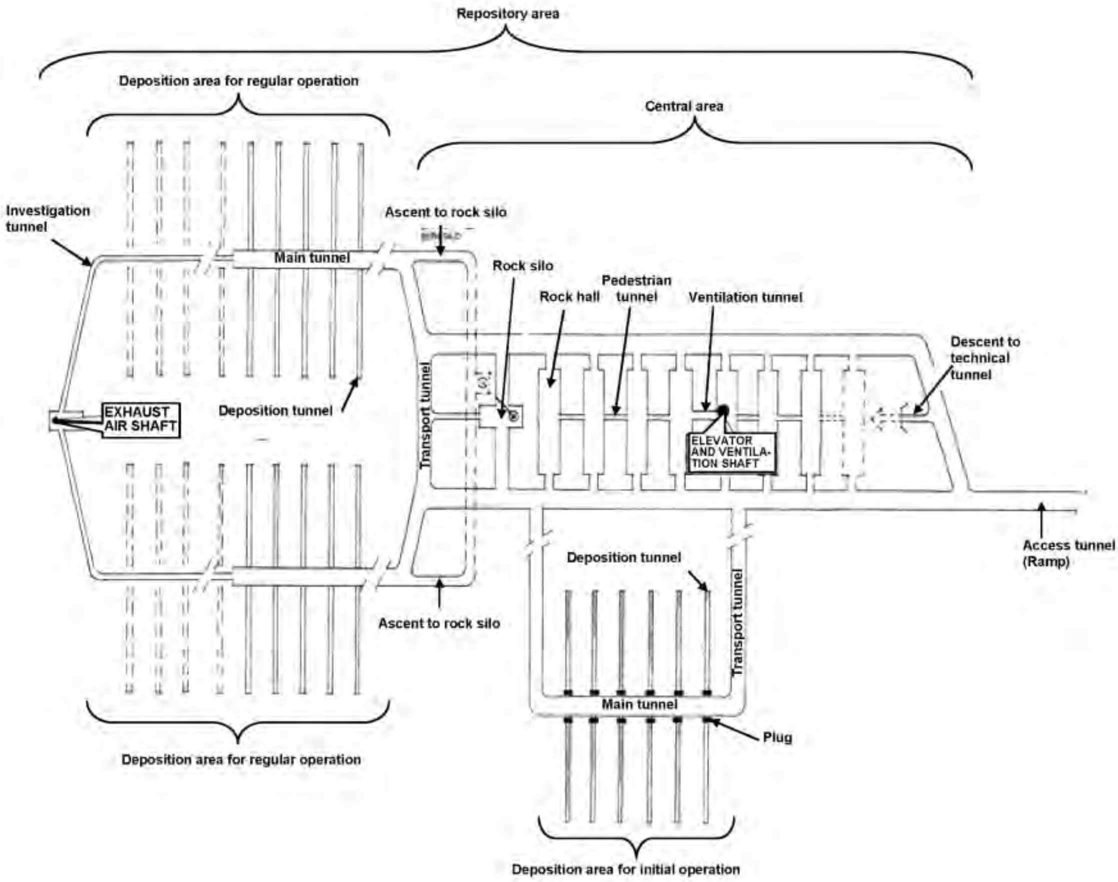


Figure 1-2. *Schematic plan showing certain parts and underground openings.*

1.5.4 Underground openings

The various openings in the hard rock facility are defined below (see also Figure 1-2) .

Rock cavern	Underground opening intended to contain caverns for personnel and visitors, technical systems, other equipment or for loading/unloading that is required for construction and operation.
Rock silo	Cavern for interim storage of rock spoil from blasting.
Central area's rock caverns	Caverns necessary for operation of the final repository.
Deposition hole	Hole for deposition of canisters containing spent nuclear fuel. Besides canisters, deposition holes also contain the buffer.
Deposition tunnel	Tunnel from which deposition holes are bored.
Pedestrian tunnel	Connecting passageway between the rock halls in the central area.
Ramp	Inclined transport tunnel providing access for vehicles between ground surface and repository level.
Shaft	Vertical or steeply inclined opening connecting ground surface and repository level.
Main tunnel	Tunnel leading directly to the deposition tunnels and connecting deposition tunnels with other underground openings.
Transport tunnel	Tunnel between different deposition areas.
Installation tunnel	Tunnel for technical systems.
Other rock cavern	Cavern that is not deposition tunnel or deposition hole.

1.5.5 Documents

Different documents are defined below.

Facility description	The facility description presents the layout of the final repository facility, the sequential construction of the facility, systems for construction and operation activities, etc.
Site Descriptive Model (SDM)	The site description is an integrated description of a site (geosphere and biosphere) and its regional surroundings with respect to current state and naturally ongoing processes.
Preliminary Safety Assessment Report (PSE)	The Preliminary Safety Evaluation report describes the analyses and assessments of the post-closure radiological safety of the final repository.

1.5.6 Other definitions

Other definitions are given below.

Aggressive water	Water which, when analyzed according to the method description "Determination of corrosive properties of water" (National Road Administration), exhibits one or more of the following properties: • pH < 6.5, • hardness < 20 mg Ca/l (total hardness), • alkalinity < 1 meq/l, • conductivity > 100 mS/m.
Rock domain	A region of rock containing rock units whose properties can be considered to be statistically uniform.
Respect Distance (RD)	The minimum permissible distance between a deposition hole and a zone with a trace length of 3,000 m or more, due to anticipated future seismic events on canister integrity /SKB 2004b/.
Margin for Excavation (MFE)	The minimum distance a deposition tunnel or cavern excavation should be from a particular deformation zone from the point of view of ease of construction.
Rock contour	Actual rock surface surrounding a tunnel, rock cavern, shaft, etc, i.e. outside support, drains, etc.
Internal contour	Actual envelope surrounding the free space in a tunnel, rock cavern, shaft, etc, i.e. inside concrete structure, support, drains, etc.
Theoretical internal contour	Theoretical envelope surrounding the free space in a tunnel, rock cavern, shaft, etc, i.e. inside concrete structure, support, drains, etc.
Theoretical rock contour	Theoretical rock surface surrounding a tunnel, rock cavern, shaft, etc, i.e. outside support, drains, etc.
Design working life	The assumed period for which a structure is to be used for its intended purpose with anticipated maintenance and repair.

2 Design premises and site conditions

2.1 Design Methodology

The design methodology adopted in this study is in detail described in the UDP (Underground Design Premises) /SKB 2004a/, and below general principles and the logical stepwise design process is explained.

For each site the design methodology calls for dealing with a number of design tasks, which are:

A. What locations and depths within the site may be suitable for locating the final repository, considering the conditions and status of the site?

B. Is it reasonable that the repository can be accommodated at the site, considering assumed preliminary respect distances to deformation zones and loss of deposition holes?

C. How can the deposition areas be designed with regard to sufficient space and long-term safety?

   C1. How can deposition tunnels, deposition holes and main tunnels be designed with regard to the proposed deposition procedure equipment, and the activities they are supposed to accommodate also considering stability and location of temporary plugs?

   C2. What distance may be required between deposition tunnels and between deposition holes given maximum permissible temperature at the canister surface?

   C3. What orientation may be suitable for deposition tunnels with respect to water seepage and stability in deposition tunnels and deposition holes?

   C4. What number of deposition holes may be unusable considering the minimum permissible distance to stochastically determined fractures, excessive water inflow and rock instability? How is the loss of deposition holes affected by different criteria?

   C5. At what depth or depth range may it be suitable to construct the final repository? Is there a site specific depth dependence?

D. How can the other underground openings, especially the central area's rock caverns, be designed with respect to rock stability and functional requirements?

E. How should the layout of the entire hard rock facility be configured?

F. What deformation zones might be intersected by different types of tunnels and what difficulties could be expected to arise?

G. How could the repository be affected by the hydrogeological conditions around the repository with respect to: (1) migration of saline water from below, and (2) lowering of the water table?

I. How much grouting might be required?

J. How much rock support might be required?

K. What consequences can different design requirements, criteria and parameters be expected to have on the design of the hard rock facility with respect to perimeter of utilized deposition area, utilization ratio and excavated rock volume? What studies and investigations need to be done before or during the next design step?

L. Documentation of performed design work (this report).

The design methodology is described in Figure 2-1, where the different design tasks and the logical framework and re-iterating loops for the various tasks are illustrated. After design tasks B, E, G and I, SKB and the review team has checked and evaluated the design results and approved and/or given instructions for the subsequent design work.

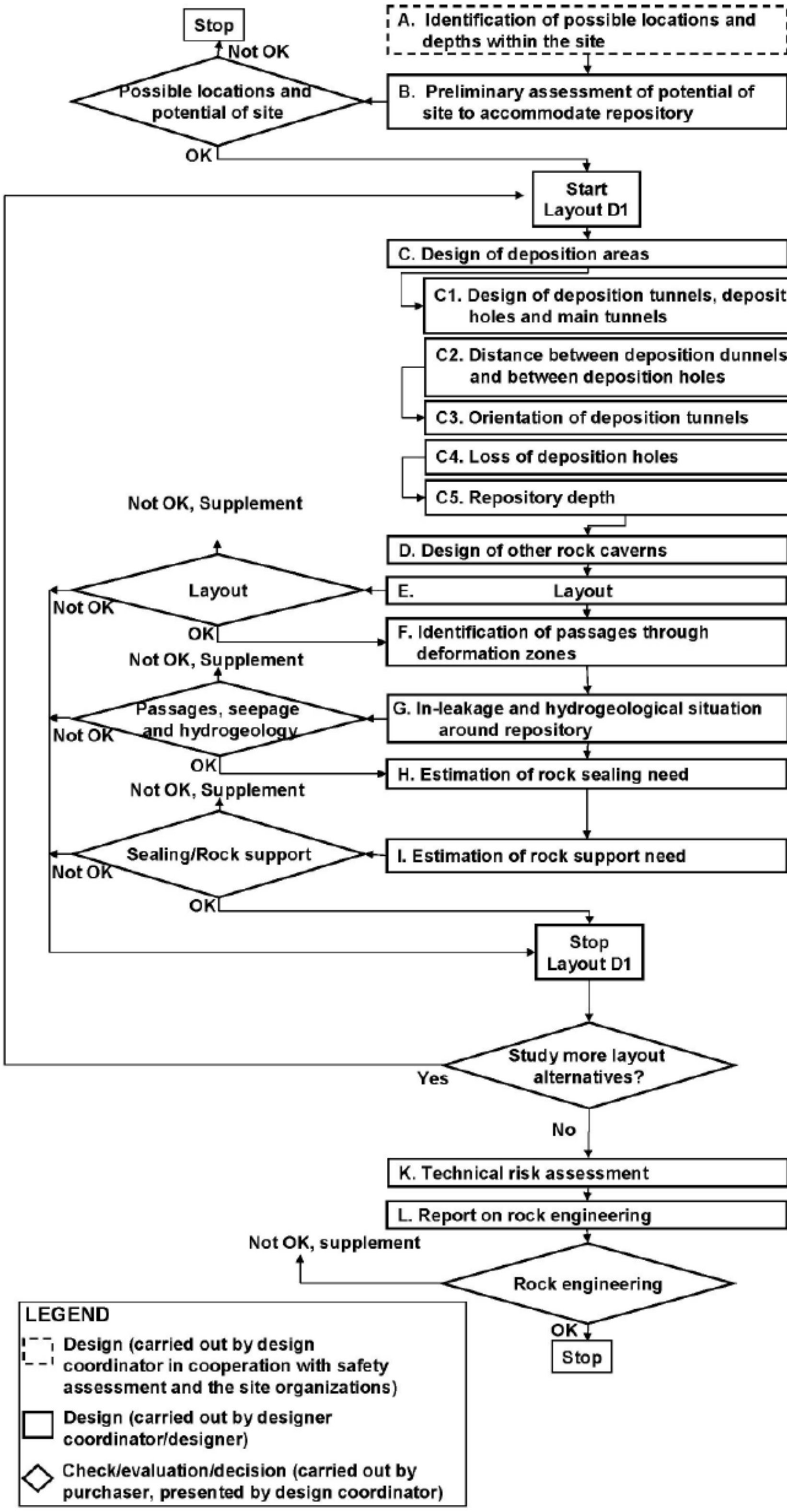


Figure 2-1. *Design methodology, logical framework.*

2.2 Site specific key issues

For the Forsmark site the following main site specific key issues were identified prior to commencement of the design step D1:

- High rock stresses at repository level, indicating risk for rock spalling.
- Extraordinary low inflow of water into the investigation boreholes at depths below 300 m, i.e. at repository level.
- High inflow of water in boreholes at depths above 200 m, to be considered for the design of access ramp and shafts.
- High thermal conductivity properties for the rock mass.

Site specific key issues are further identified and analysed in the individual design tasks, and in the technical risk assessment presented in this report.

2.3 Overview of input data for the design

2.3.1 Input from site investigations

It is postulated that the SDM v1.2 /SKB 2005a/ shall be the basis for the design step D1 /SKB 2004a/. However, as described in Chapter 1, the design work presented in this report was based on preliminary site modelling results, and not until a late phase of the design work, final SDM results could be compared with the preliminary results used. Identified discrepancies are listed in Table 2-1, and it was intended to rectify the analysis only if it was assessed that the final /SKB 2005a/ results would not be conservative. The influence on the respective design task concerning new data not applied in analyses are assessed and shown in Table 2-1.

Table 2-1. Major differences between "preliminary data" used in design step D1 and input data from the SDM v1.2 /SKB 2005a/.

Design task	Chapter in this report	Preliminary data used	Final /SKB 2005a/ data	Estimation of influence from new data	Analysis rectified Yes/No
Orientation of deposition tunnels and loss of deposition holes	4.3 4.4	Minimum horizontal stress approximately 20 MPa at depth 400–500 m	Minimum horizontal stress about 30 MPa at depth 400–500 m	Conservative	No
– " –	4.3 4.4	Orientation of maximum horizontal stress 145 degrees (mean value)	Orientation of maximum horizontal stress 140 degrees	Neutral	No
– " –	4.3 4.4	Fracture intensity, P_{32}, for DFN-model	DFN-model with lower fracture intensity for four fracture sets and higher intensity for one fracture set.	Neutral/ Conservative	No
Layout studies	5	Analyses based on "preliminary data" resulted in a loss of deposition holes of 11% for 400 m depth and 14% for 500 m depth.	Data from /SKB 2005a/ resulted in a loss of deposition holes of 9% for both 400 m and 500 m depth.	Conservative (the layout was designed with a surplus of canister positions)	No

Design task	Chapter in this report	Preliminary data used	Final /SKB 2005a/ data	Estimation of influence from new data	Analysis rectified Yes/No
Identification of passages through deformation zones/ Estimation of rock grout need	6 and 8	Estimation of hydraulic conductivity for deformation zones	Hydraulic conductivity values lower for some deformation zones.	Conservative	No
Seepage and hydrogeological situation around repository	7	Hydraulic conductivity for the rock mass between deformation zones is given for different orientations	Hydraulic conductivity for the rock mass between deformation zones is not given for different orientations and is also presented in a somewhat different way.	Neutral	No
Rock support need	9	"Preliminary data" resulted in an assessment of rock support need	Compared to the "preliminary data" the assessment of rock quality was the same, but the minimum horizontal stress was higher.	Neutral	No

2.3.2 Input from SKB

Based on the results from previous studies and investigations, SKB has given specific premises regarding the location and depth of the underground part of the repository. A more detailed presentation of the premises and motives for the premises are given in Chapter 3.

The minimum required number of canister positions in the repository is, according to current plans for the Swedish nuclear programme, determined to 4,500. However, in order to accommodate the uncertainty in geological conditions and tentative future extensions of the nuclear plants operation period, the deposition area should according to SKB be designed for a capacity of 6,000 canisters.

With the objective not to deviate too much from the reference Layout E /SKB 2002/ of the repository, SKB has decided that the minimum distance between canisters should be 6 m, even though the temperature properties of the rock mass might allow for a shorter distance, see Section 4.2. During completion of this design report, findings in parallel ongoing studies, Preliminary assessment of long-term safety for KBS-3 repositories at Forsmark and Laxemar /SKB 2006/, also revealed that the temperature criteria for the canister and buffer could be changed from 100°C at the canister surface to max 100°C inside the buffer. This indicated the possibility to allow for 10°C higher temperature when evaluating the canister spacing according to Figure 5-4 in UDP /SKB 2004a/. However, SKB decided not to utilise this opportunity, and consequently not to revise the study at this late stage.

Due to almost no measurable inflows from the site investigations at depths below approximately 360–400 m, no hydro-DFN model was presented as "preliminary data" regarding the model volumes relevant for the repository. Consequently SKB decided to omit the planned hydraulic numerical analysis for the Forsmark area.

Loss of deposition holes due to stochastically determined fractures is as described in Section 2.2 calculated according to /Hedin 2005/. The analytical calculations were prepared by SKB, and the design team was instructed to adopt a loss rate of approximately 9% for stochastically determined fractures.

Orientation of deposition tunnels and loss of deposition holes due to the risk of spalling was analysed and reported in /Martin 2005/. The report also included analysis of potentially unstable wedges, and was delivered by SKB to the design team, who included the results in their design work.

2.4 Deviations from the design premises

The design work in design step D1 presented in this report has primarily been based on /SKB 2004a/. However, some amendments have for various reasons been introduced. For example the ongoing R&D work within SKB has given new insight and understanding of studied tasks, such as the analytic method for estimating the probability of canister/fracture intersections in a KBS-3 repository /Hedin 2005/ that overrule suggestions on this matter in /SKB 2004a/. In other cases parallel studies within the design activities of SKB have given sufficient information already at this early design stage, such as for example /Martin 2005/, in which rock mechanical issues were analysed. Due to obtained site specific information it has also been obvious that the proposed analysis in /SKB 2004a/, as for example hydro-DFN studies at large depths or ground water drawdown at shallow depths, is not meaningful, or ought to be carried out differently. All deviations from the strategy outlined in /SKB 2004a/ are summarised in Table 2-2.

Table 2-2. Deviations from /SKB 2004a/ in this design report.

Design task	Chapter in this report	Premises according to /SKB 2004a/	Deviation from /SKB 2004a/	Justification
Distance between deposition tunnels and deposition holes	4.2	Distance between deposition holes based on result from calculations.	Longer distance between holes than calculated was used (6 m instead of about 5.5 m).	A better agreement with reference layout is obtained.
Orientation of deposition tunnels and loss of deposition holes	4.3 4.4	Inflow of water to deposition tunnels and deposition holes should be analysed.	No analyses regarding the water inflow were made.	No anisotropy in data were presented in /SKB 2005a/. Descriptions of flow characteristics in /SKB 2005a/ also indicate that the inflow could be neglected.
Orientation of deposition tunnels and loss of deposition holes	4.3 4.4	Both design tasks should include an analysis of potential unstable wedges.	No analyses of potentially unstable wedges were made.	Small risk of outfall of wedges /Martin 2005/.
Loss of deposition holes due to stochastically determined fractures	4.4	A numerical DFN-method for stochastically determined fractures with 100 m < r < 500 m should be used. The distance between the deposition hole and the closest fracture must be < 2 m if the fracture has a radius, 100 m < r ≤ 200 m. If the fracture has a radius, r > 200 m, the distance must be < 0.01 × fracture radius.	An analytical method as proposed in /Hedin 2005/ for stochastically determined fractures with 50 m < r < 600 m were used. No minimum permissible distance as set out in /SKB 2004a/ was used. Instead the deposition hole was assumed to be lost if it is intersected by a fracture or fracture zone.	Re-evaluated limits for fractures discernible during the construction period. Less time consuming calculation method.

Design task	Chapter in this report	Premises according to /SKB 2004a/	Deviation from /SKB 2004a/	Justification
Orientation of deposition tunnels and loss of deposition holes	4.4	Loss of deposition holes due to the risk of spalling should be based on numerical calculations using a three dimensional model including one deposition tunnel and four deposition holes.	Loss of deposition holes was calculated assuming one circular opening and a uniform stress distribution.	A separate study (see /Martin, 2005/) was exectuded by SKB, considering new findings regarding regarding rock mechanical issues.
Layout	5	Respect distances for deterministically determined deformation zones should be recognized when locating deposition tunnels.	Deterministically determined deformation zones with a length < 3,000 m are allowed to pass deposition tunnels. However, no deposition holes are allowed within the “margin for construction”, normally 2×5 m (5 m at each side of the zone) for these zones.	Premise from SKB before start of the design.
Layout	5	Sensitivity analysis shall be made for all layouts proposed.	Sensitivity analysis was made for one layout (considered to be the base layout)	Premise from SKB before start of the design.
Seepage and hydrogeological situation around repository	7	Numerical analyses should be made with both of the software tools Darcy Tools and Connect Flow.	Analyses have been made only with Darcy Tools.	Decision by SKB.
Seepage and hydrogeological situation around repository	7	Analysis of ground water draw down should be made.	No analysis of draw down was made.	Draw down assumed not to be an issue due to probable contact between fractured superficial rock mass and the sea.

3 Possible locations and preliminary assessment of the potential of the site to accommodate the repository

3.1 Introduction

The design commenced with an identification of suitable locations within the Forsmark site. The identification, which was executed by SKB based on the results of previous studies and investigations, resulted in an area of priority for the forthcoming design work. This area of priority was by SKB denoted the “priority site”. Following the identification of the “priority site”, SKB issued premises to the design team regarding the location of the repository within the “priority site”. An overview of the identification work and a presentation of the design premises, is given in Section 3.2.

A preliminary assessment of the potential to accommodate the repository was made in a subsequent step, i.e. whether the required number of canisters can be deposited within the identified “priority site”. This assessment of the potential of the “priority site” to accommodate the repository was made in principle with respect to:

1. Loss of deposition area due to preliminary respect distances to deterministically determined fracture zones.
2. A preliminarily assumed loss of deposition holes.

Owing to safety considerations it has been assumed that deposition of canisters is not allowed within a specified distance to deterministically determined deformation zones. This defined “respect distance” will reduce the area available for deposition of canisters.

Within a specified area some canister positions will not be suitable for deposition of canisters owing to local rock mechanical and/or hydrogeological conditions. Loss of deposition holes implies that additional space will be required for deposition of sufficient canisters within a specified area.

After completion of the preliminary assessment of the potential of the site to accommodate the repository, the result was reviewed together with SKB.

3.2 Possible locations

The identification of possible locations, including depths for deposition areas and other parts of the hard rock facility, was undertaken in accordance with /SKB 2004a/, taking into account:

1. The geological setting.
2. Thermal properties of the rock.
3. Hydrogeological properties of the rock.
4. Mechanical properties of the rock and initial (in situ) stresses.
5. Groundwater composition.
6. Municipal planning of land use and environmental conditions at the surface.

The identification of possible locations and depths with regard to points 1–5 was carried out by comparing the properties and conditions of rock domains with requirements and preferences according to /Andersson et al. 2000/ which are summarised as follows:

- Regional, plastic shear zones must be avoided.
- The rock mass within the deposition area must not have any ore potential.
- A repository must be accommodated and given a reasonable design with regard to the extension of the repository.
- The rock mechanical conditions must not result in significant stability problems in deposition holes or deposition tunnels.
- The groundwater at the repository level should not include dissolved oxygen.
- The total salinity (TDS) of groundwater at repository level must not exceed 100 g/l.

According to /SKB 2004a/, the comparison should be performed for depths between 400 and 700 m in each rock domain which may constitute a potential volume for location of deposition areas or other parts of the hard rock facility.

SKB has also postulated some specific requirements for the utilisation of each deposition area to be used during design step D1:

- Length of deposition tunnels shall be in the range 100 m $\leq$ L $\leq$ 300 m.
- Number of deposition tunnels shall be $\geq$ 5 in each rock unit between deformation zones.

3.2.1 Overview of the Forsmark site

An overview of the Forsmark site is presented in Figure 3-1. Different areas of the site are defined in this figure together with a presentation of completed and planned cored boreholes. A more detailed description of features controlling the location of the repository within the site is provided in Figure 3-2 and an illustration of rock domains and deformation zones according to /SKB 2005a/ is presented in Figure 3-3.

Descriptions of the “candidate area”, “priority site” and “preferable repository area” including the premises for design are provided in Sections 3.2.2, 3.2.3 and 3.2.4.

3.2.2 Candidate area

Based on earlier investigations SKB has defined an area to the south-east of the nuclear power plant suitable for the location of the repository. The area is designated the candidate area and marked in red in Figure 3-1 (in Figure 3-1 the candidate area is denoted the candidate site).

The candidate area is approximately 6 km long and 2 km wide. The rock mass in the candidate area is composed essentially of different types of granitic rock. The prevailing rock domain according to the nomenclature in /SKB 2005a/ is RFM029 (refer to Figure 3-2).

According to /SKB 2005b/ the main arguments for the selection of the candidate area were that:

- The area is well defined with potential favourable geological conditions.
- A location close to the Forsmark nuclear power plant and the SFR facility is achieved (see Figure 3-2).

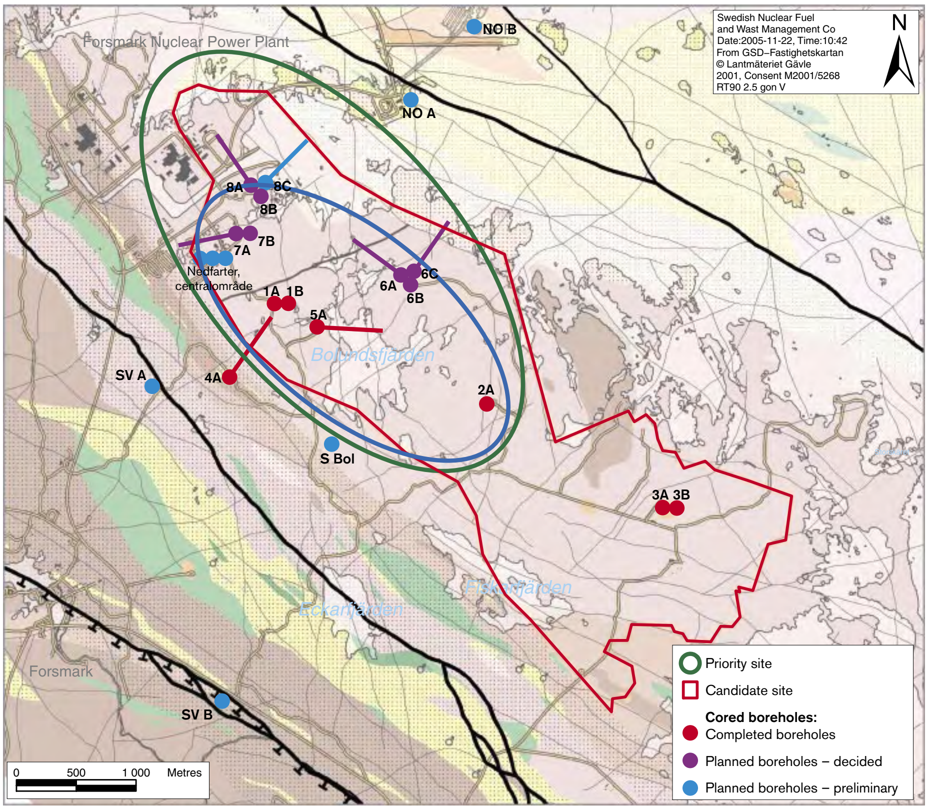


Figure 3-1. *Overview of the Forsmark site. The "Candidate area"(candidate site) is marked in red and the "priority site" is marked in green (from /SKB 2005a/). If possible the repository should be located within the area marked in blue (defined as the "preferable repository area"). The numbers 1, 2 etc at the dots representing the cored boreholes, denote the different drill sites, drill site 1, drill site 2 etc. Descriptions of the "candidate area", "priority site" and "preferable repository area" are provided in Sections 3.2.2, 3.2.3 and 3.2.4.*

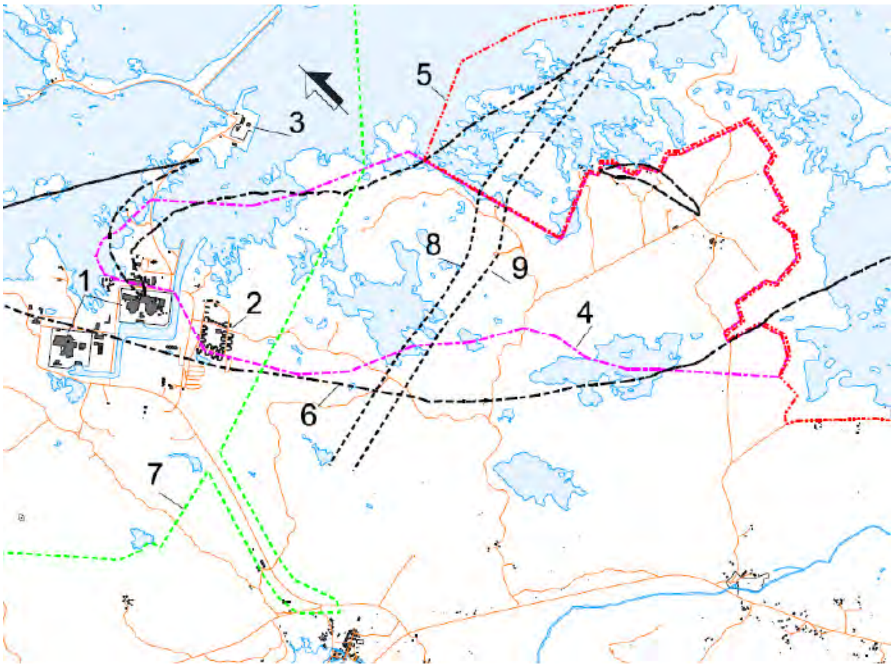


Figure 3-2. *1/ Nuclear power plant, 2/ Residential area, 3/ SFR office building, 4/ Candidate area, 5/ Nature reserve, 6/ Rock domain RFM029, 7/ Municipal detailed industrial planning area, 8/ Location of deformation zone ZFMNE00A2 at 400 m depth, /9 Location of deformation zone ZFMNE00A2 at 500 m depth.*

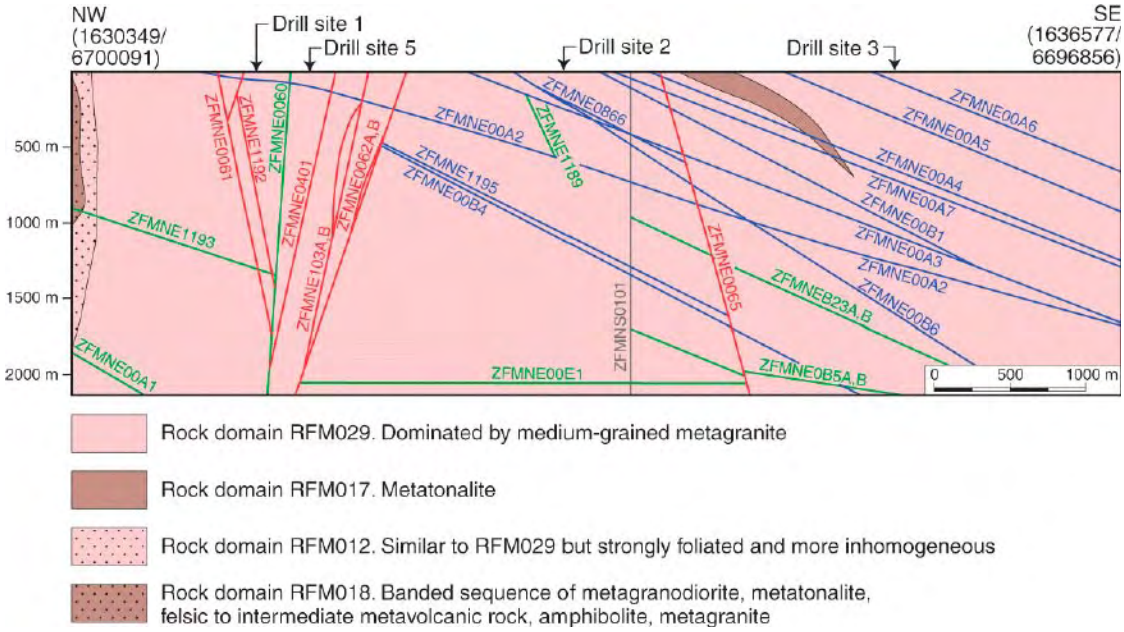


Figure 3-3. *NW-SE cross-section that passes close to drill site 1, 2, 3 and 5 inside the candidate area (see Figure 3-1). The figure shows the steeply dipping deformation zones that strike NE and the gently dipping zones that dip to the south-east and south. The zones coloured in red shades are vertical and steeply dipping zones with high confidence, the zones coloured in blue shades are gently dipping zones with high confidence, the zones coloured in green shades are medium confidence zones irrespective of their dip and the zone coloured in grey shade is a vertical zone with low confidence (from /SKB 2005a/).*

A location close to the nuclear power plant and the SFR facility will result in the following benefits:

- Existing harbour and other infrastructure could be used.
- Transport distances to the repository would be short.
- Part of the area is planned for industrial use and thus the need of utilising areas of environmental interest will be small (see Figure 3-2).

3.2.3 Priority site

Based on results from the initial site investigations, a suitable location for a tentative deep repository was identified by SKB, the so called “priority site” in the northern part of the candidate area. The “priority site” is marked green in Figure 3-1. The ongoing site investigation is concentrated on the “priority site”.

The choice of “priority site” was based on the existence of a potential wide and water-bearing sub-horizontal fracture zone (ZFMNE00A2) which is assumed to intersect the candidate area at the repository depth approximately in the location of borehole 2A (see Figure 3-1 and Figure 3-2). Based on the currently available site investigations results it was concluded in /SKB 2005b/ that the requirements according to /Andersson et al. 2000/ are fulfilled north of deformation zone ZFMNE00A2 and that consequently complete site investigations may continue in this area. Moreover, initial layout studies indicate that it is likely that the repository may be accommodated in this area.

From a technical point of view there are also several advantages in locating the repository within the "priority site", for example it is assumed that the fracture intensity, the number of sub-horizontal fracture zones and the hydraulic conductivity of the rock mass increase south-east of the "priority site" and deformation zone ZFMNE00A2.

3.2.4 Preferable repository area and premises for design

For the design work and layout studies it was decided by SKB and the design team that the repository should, if possible, be located within the southern part of the "priority site" closer to the deformation zone ZFMNE00A2, keeping the area below the nuclear power plant and the sea at Asphällsfjärden as spare area for future extension. This preferred area for the repository is marked in blue in Figure 3-1. The main reason for this decision was that the repository should not be located too close to the nuclear power plant nor too far from the shoreline. Reasons for this decision were:

- The uncertainty whether the deposition tunnels can be located below the nuclear power plant. As no geological investigations have so far been carried out below the power plant and the legal premises are not fully understood, it has been assumed in design step D1 that no deposition tunnels should be located northwest of the inlet canal for cooling water.
- In order to reduce uncertainties based on existing geological interpretations, it was decided to limit the extension of the deposition area below the sea, Asphällsfjärden.

A further premise given by SKB regarding the location of the repository was that it is not necessary for it to be located strictly within the borders of the candidate area. The deposition area must, however, be located within rock domain RFM0029, which is the dominant rock domain within the candidate area, and outside the border of the nature reserve (see Figure 3-2). Based on an initial evaluation of the rock mass conditions, transport tunnels may, if necessary, be located in adjacent rock domains.

According to SKB the surface part of the repository should be located within the industrial area close to the nuclear power plant and consequently also within the municipal detailed industrial planning area. This design requirement has an effect on the repository layout since the location of the central area of the repository is restricted by this prerequisite. According to SKB the surface part of the repository and the ramp should be located with respect to the following two main alternatives:

1. The surface facilities should be located to the so-called residential area, directly south of the power plant (see Figure 3-2).
2. The main part of the surface facility including the adit of the access tunnel should be located close to the SFR office building. A minor part should be located to the so-called residential area (see Figure 3-2).

3.2.5 Depth of the repository

Present knowledge acquired from the site investigations indicates that it is possible to locate the repository at all stipulated depths between 400 and 700 m according to /SKB 2004a/. However it has been concluded from the initial site investigations that the rock stresses are high within the Forsmark area, which implies that a shallow depth will be preferable. Since the site investigation results have not so far indicated any major differences in other properties of the rock mass between 400 and 700 m depth, SKB decided that the design work should focus on the repository levels of 400 m and 500 m.

3.3 Preliminary assessment of the potential of the site to accommodate the repository

3.3.1 Execution

The potential of the site to accommodate the repository was assessed by:

1. Marking of preliminary respect distances given by SKB from deterministically determined deformation zones.
2. Preliminary calculation of the potential to accommodate the required number of canisters.

According to /SKB 2004a/, the potential of the site to accommodate the repository is given by Equation 3-1.

$$P = \left(1 - \frac{K}{100}\right) \cdot \frac{A_T}{N \cdot A_S} \qquad \text{Equation 3-1}$$

where

P = The potential of the site to accommodate the repository

K = Assumed preliminary percentage loss of deposition holes

N = Number of canisters

A_T = Area available for deposition, i.e. sum of areas for rock domains at given depth after reduction for respect distance

A_S = Preliminary specific area required for each deposition hole

A value of P < 1 determined from Equation 3-1 indicates that it may not be possible to accommodate the required number of canisters within the available area, while a value of > 1 indicates that there may be surplus capacity.

The analyses were made for the "priority site" at 400 and 500 m depth.

3.3.2 Input data and assumptions

For the marking of preliminary respect distances (defined as the distance from the centre of the zone to deposition tunnels) the following input was used:

- Table of properties for deformation zones from /SKB 2005a/.
- Digital models regarding the geometry of rock domains and deformation zones (provided by SKB).

For deformation zones with a length equal to or exceeding 3,000 m the respect distance is equal to the width of the zone, but not less than 100 m /SKB 2004b/.

For deterministically determined zones of length < 3,000 m no respect distance is defined according to /SKB 2004b/. For these zones SKB has proposed to apply a "margin for construction" in accordance with Equation 3-2 as follows.

Margin for construction = (Thickness of zone + variation)/2 + safety margin (SM)

Equation 3-2

where

SM = 10 m if stability problems are expected

SM = 20 m if water problems are expected

SM = 5 m, default value, if stability or water problems are not expected

The margin for construction is thus to be interpreted as the distance from the centre of the zone to the periphery of the deposition holes. No deposition holes are permitted within the borders of the “margin for construction”.

The specification of the properties of the deformation zones given in /SKB 2005a/ was used in determining the value of the safety margin (SM) for zones shorter than 3,000 m. From /SKB 2005a/ it can be concluded that the most water-bearing zones are the sub-horizontal zones (dip < 45°) and those trending north-west. Since the deterministically determined deformation zones that are shorter than 3,000 m in the “priority site” trend to the north-east, no water problems are thus expected. From Q-classifications based on boreholes KFM01–04A (see Table 6-4 in /SKB 2005a/) it can be expected, that no stability-related problems will be associated with the deformation zones shorter than 3,000 m. The deformation zones shorter than 3,000 m will consequently be allocated a safety margin (SM) of 5 m.

It should be noted that the respect distances and margins for construction applied in design step D1 are preliminary and will be evaluated by SKB within the framework of future safety assessment. Accordingly the respect distances may be revised later based on the results of further investigations and/or the safety assessments.

The following assumptions were made when calculating the potential to accommodate the repository in accordance with Equation 3-1:

- K = 25% for all depths /SKB 2004a/.
- N = 4,500 and 6,000 canisters.
- A_S = 240 m² /SKB 2004a/.

It should be noted that only the assessed preliminary percentage loss of deposition holes is used in the present analysis. Analyses of the loss of deposition holes based on the actual site conditions according to /SKB 2005a/ are presented in Section 4.5.

Regarding the number of canisters, N, the minimum required number of canister positions in the repository is according to current plans for the Swedish nuclear programme determined to 4,500. However, in order to accommodate the uncertainty in geological conditions and tentative future extensions of the nuclear plants operation period, the deposition area should according to SKB be designed for a capacity of 6,000 canisters.

Furthermore the calculation of the area available for deposition, A_T, has been performed in accordance with the following premises:

- The percentage of holes lost (25%) has been assumed to include the loss due to deformation zones with length < 3,000 m. Thus, no reduction in area was made with respect to these deformation zones.
- Deposition tunnels will be located within the “priority site” (see Section 3.2) between the deformation zones ZFMNE1193 and ZFMNE00A2 (see Figure 3-4 and Figure 3-5).
- Deposition tunnels are to be located within rock domain RFM029 (see Section 3.2).
- No part of the facility is located within the border of the nature reserve (see Section 3.2).

For the calculation of the area available, A_T, the following aspects have thus not been considered:

- The area occupied by main tunnels.
- The distance between the first deposition hole and the main tunnel.
- The location of the central area, which is located below the residential area.
- Uncertainties in how the area between the deformation zones could be utilised for deposition due to the geometry of the available area.

The calculations of the area available, A_T, are assumed to be performed within an accuracy of ±10%. This implies an uncertainty in the potential, P, of about ±0.15. The potential should therefore be 1.15 or greater if the repository is to be accommodated within the studied area.

3.3.3 Results

Deformation zones with preliminary respect distances and margins for construction and which are located within the “priority site“ are presented in Figure 3-4 and Figure 3-5. More detailed maps are shown in Appendix A.

For calculating the potential to accommodate the repository the “priority site” has been divided into three sub-areas. A schematic illustration of the defined sub areas A_T 1, A_T 2 and A_T 3 is presented in Figure 3-6 and Figure 3-7 . A more detailed illustration of the basis for the calculations is presented in Appendix B. Table 3-1 indicates the area available, A_T, and the potential of the site, P, for depths of 400 m and 500 m.

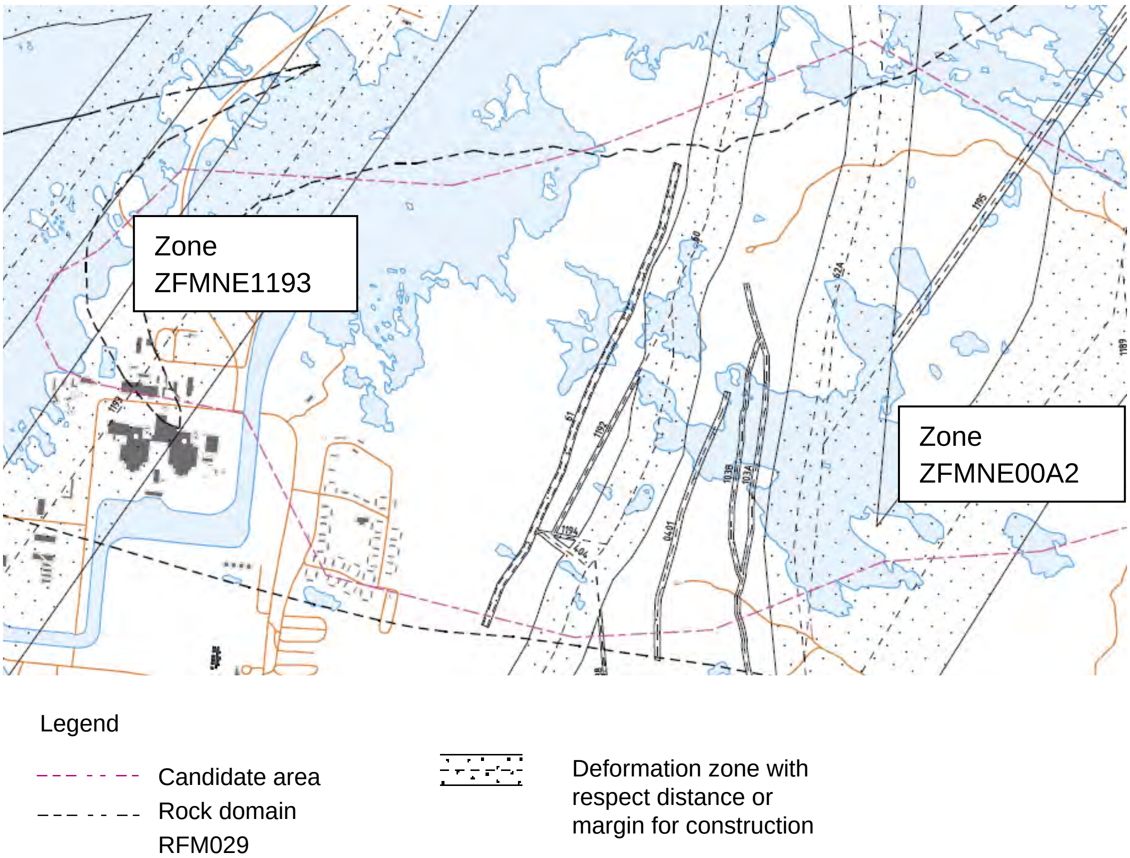


Figure 3-4. *Deformation zones with preliminary respect distances or margin for construction within the “priority site”, depth 500 m.*

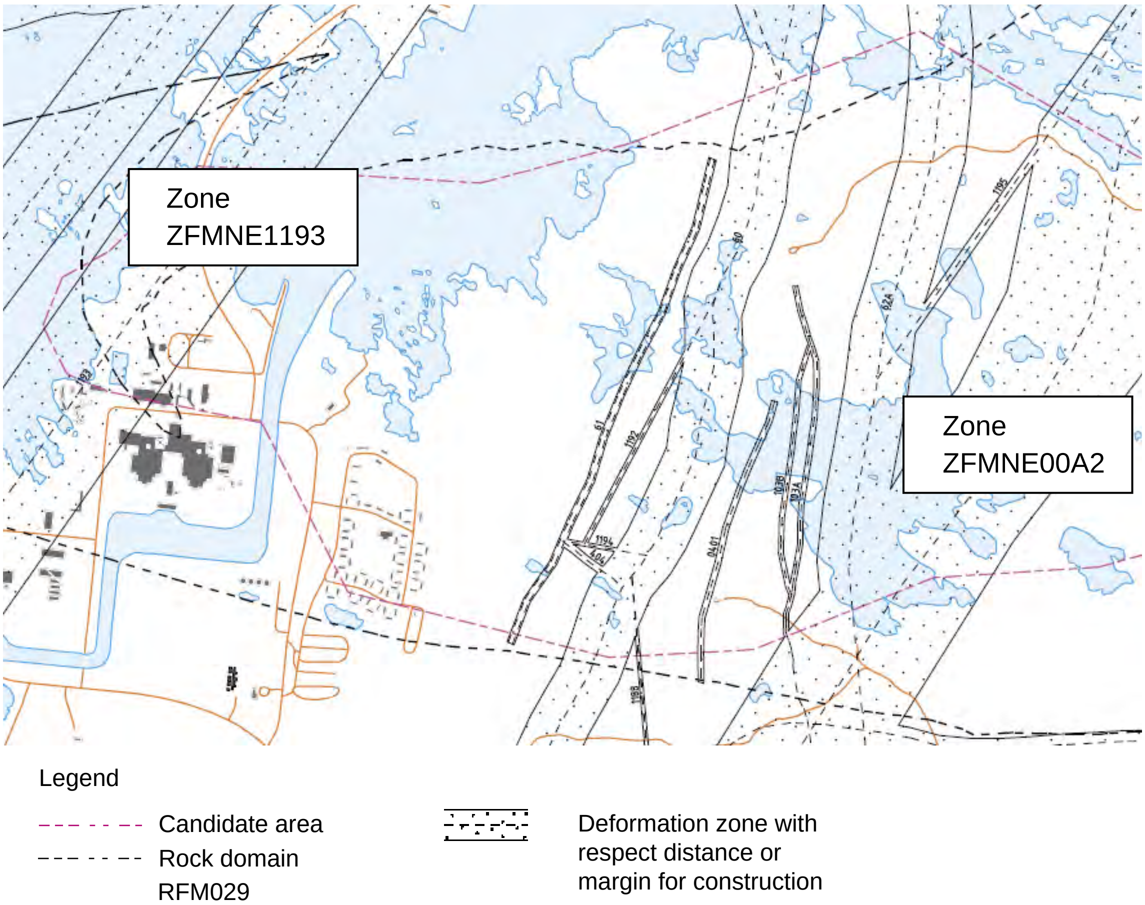


Figure 3-5. *Deformation zones with preliminary respect distances or margin for construction within the "priority site", depth 400 m.*

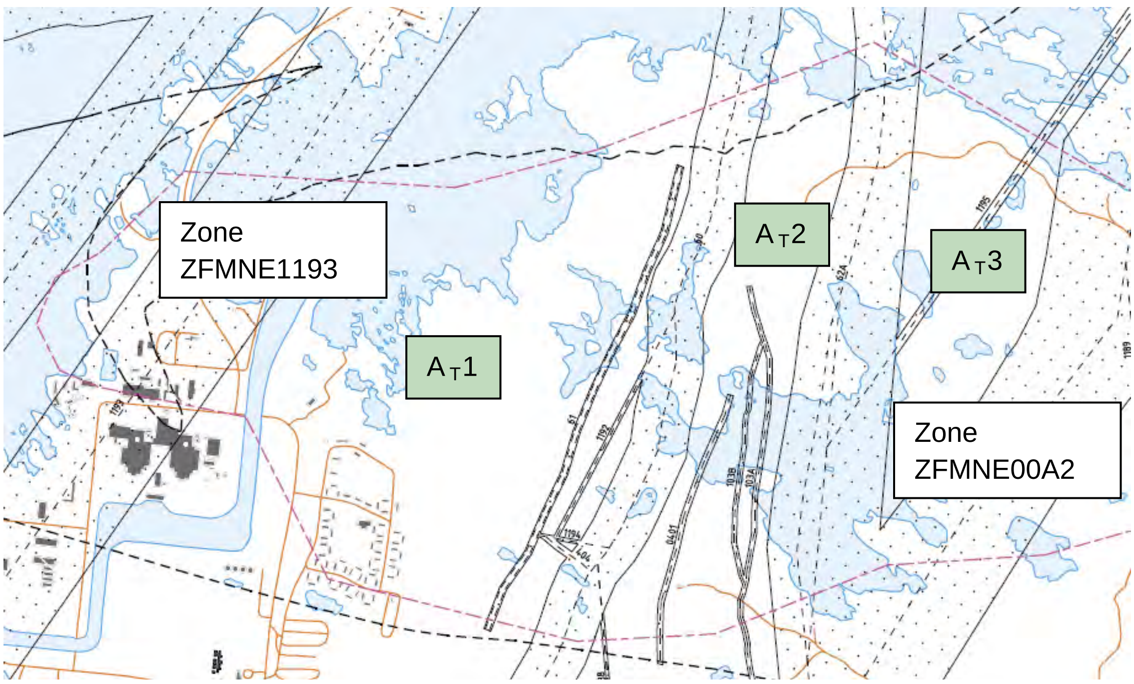


Figure 3-6. *A rough illustration of the definitions for the sub areas A_T1, A_T2 and A_T3, depth 500 m.*

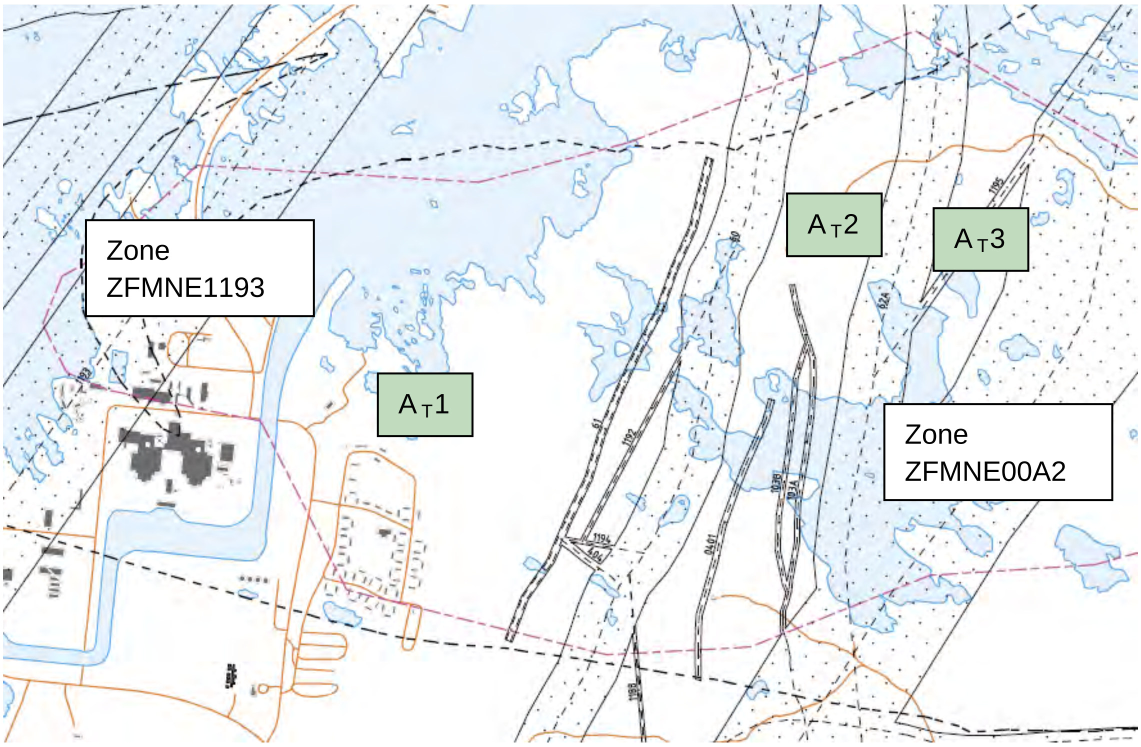


Figure 3-7. *A rough illustration of the definitions for the sub-areas A_T1, A_T2 and A_T3, depth 400 m.*

Table 3-1. Area available, A_T, and the potential of the "priority site", P, for depths of 400 m and 500 m.

Depth (m)	Area available, A_T	N	A_T (m^2)	P
400	A_T 1	4,500	2,195,000	1.52
		6,000	2,195,000	1.14
	A_T 1 + A_T 2	4,500	2,885,000	2.00
		6,000	2,885,000	1.50
	A_T 1 + A_T 2 + A_T 3	4,500	3,095,000	2.15
		6,000	3,095,000	1.61
500	A_T 1	4,500	1,985,000	1.38
		6,000	1,985,000	1.03
	A_T 1 + A_T 2	4,500	2,660,000	1.85
		6,000	2,660,000	1.39
	A_T 1 + A_T 2 + A_T 3	4,500	3,200,000	2.22
		6,000	3,200,000	1.67

3.3.4 Conclusions

From the calculations of the area available for deposition, A_T, and the potential, P, it is concluded that the repository may be accommodated within the “priority site” at depths of 400 m or 500 m. For the subsequent design it was therefore decided that both depths, 400 m and 500 m, should be studied further. Uncertainties in the calculated potential, P, are related to the fact that no reduction of the area was made with regard to for example main tunnels and the central area. As stated in Section 3.1 the assessment of the potential is thus only preliminary.

In order to optimise utilisation of the bedrock between the deformation zones, the transport tunnels could if necessary be located outside rock domain RFM029.

The percentage of deposition holes lost will probably be different at different depths primarily due to increased rock stresses at greater depth.

Based on the results of the design task described in this chapter it was decided to continue the design process and concentrate the work on the identified “priority site”.

4 Design of deposition areas

4.1 Introduction

Based on the preliminary assessment of the potential of the priority site to accommodate the repository, SKB decided to proceed with the design of deposition areas, which include main tunnels, deposition tunnels and deposition holes.

The design work was carried out with focus on the following issues in accordance with /SKB 2004a/:

- Design of deposition tunnels, deposition holes and main tunnels with respect to space for equipments and planned deposition activities, stability and location of temporary plug (see Section 4.2).
- Distance between deposition tunnels and deposition holes with respect to maximum permissible temperature on the canister surface (see Section 4.3).
- Suitable orientation for deposition tunnels with respect to water seepage and stability in deposition tunnels and deposition holes (see Section 4.4).
- Proportion of the deposition holes that might be lost with respect to the minimum permissible distance to stochastically determined fractures, excessive water inflow and instability of deposition holes (see Section 4.5).
- Suitable depth of range for construction of the deep repository (see Section 4.6).

Design step D1 also included design of other underground excavations, covering tunnels and caverns in the central area as well as ramp, shafts and transport tunnels. This design, which was executed by SKB, included some modifications of the design presented in Layout E /SKB 2002/. The other underground excavations are not per definition parts of the deposition area, but for convenience the design of these underground openings is presented in this chapter as well (see Section 4.7).

4.2 Design of deposition tunnels, deposition holes and main tunnels

The deposition areas include deposition tunnels, deposition holes and main tunnels (see Figure 1-1 and 1-2).

According to /SKB 2004a/ the deposition tunnels are to be designed in design step D1, taking the following issues into account:

- Space required for the equipment and installations required for ventilation, transport of rock spoil, rock investigations, preparation and cleaning of deposition holes, deposition of buffer and canisters, backfilling and temporary plugging.
- Possibility of canister retrieval.
- Minimum distance required between deposition holes and the main tunnels with regard to:
  - stress state around the deposition holes due to stress redistribution around the main tunnel,

  – position of the concrete plug considering potential fracturing in the rock mass due to unidirectional water pressure on the concrete plug.

- Minimum distance required between deposition holes and end of tunnel.
- Stability in tunnels.

The design of deposition holes shall take into account:

- Space required for deposition of buffer and canisters.
- Possibility of canister retrieval.

The design of main tunnels shall take into account:

- Space required for the equipment and installations involved for ventilation, transport of rock spoil, rock investigations, preparation and cleaning of deposition holes, deposition of buffer and canisters, backfilling and temporary plugging.
- Stability in tunnels.

In design step D1 the requirements for space regarding equipment and installations in deposition tunnels and main tunnels, the space required for deposition of buffer and canisters in deposition holes and finally the possibility of canister retrieval are considered to be fulfilled if the theoretical rock contours conform to the cross-sectional dimensions and forms of the deposition tunnels, deposition holes and main tunnels presented in the facility description, Layout E /SKB 2002/.

As a prerequisite for the design step D1, it was postulated that the minimum required distance between deposition holes and main tunnels should be 20 m (see Figure 4-1).

The minimum permissible distance between the periphery of the deposition hole and end of tunnel shall in design step D1, see Figure 4-2, be 8 m.

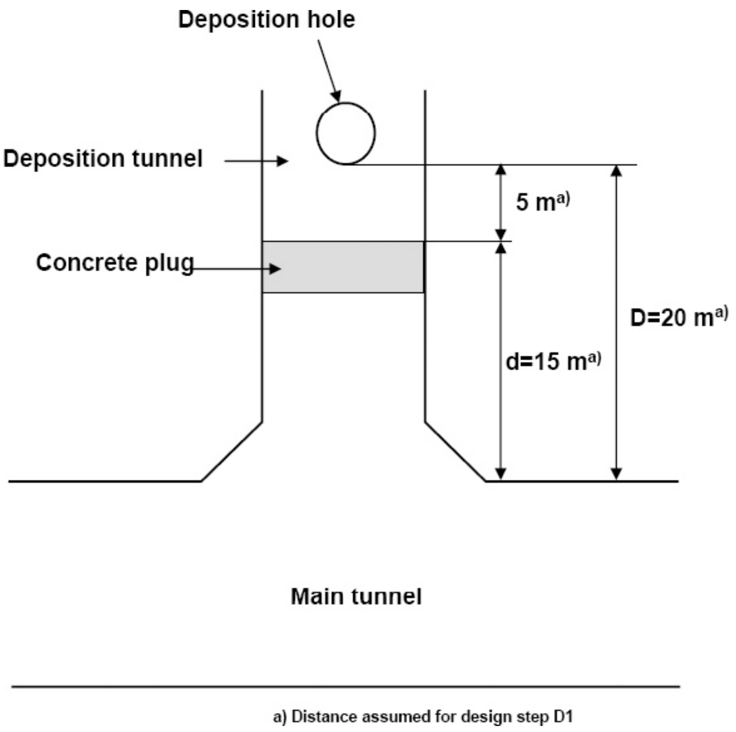


Figure 4-1. *Schematic plan view of main tunnel, deposition tunnel and deposition holes (from /SKB 2004a/).*

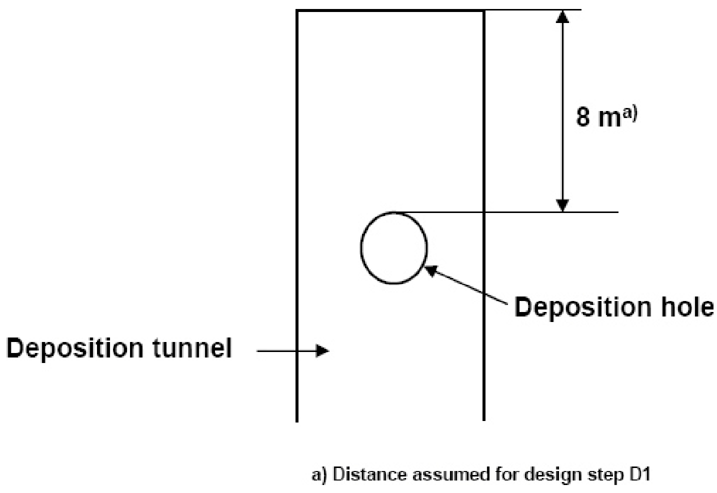


Figure 4-2. *Distance between deposition holes and end of tunnel (from /SKB 2004a/).*

The stability requirements for the tunnels shall be considered to be met in design step D1 if the shape and cross-sectional dimensions of the tunnels according to the facility specification Layout E /SKB 2002/ are applied and necessary rock support is installed.

Figure 4-3 illustrates the cross-section of a deposition tunnel with a deposition hole and Figure 4-4 illustrates the cross-section of a main tunnel. It should be noted that the cross-section of the deposition tunnel is revised compared to the cross-section presented in Layout E /SKB 2002/. In Figure 4-5 the intersection of a main tunnel and a deposition tunnel is illustrated.

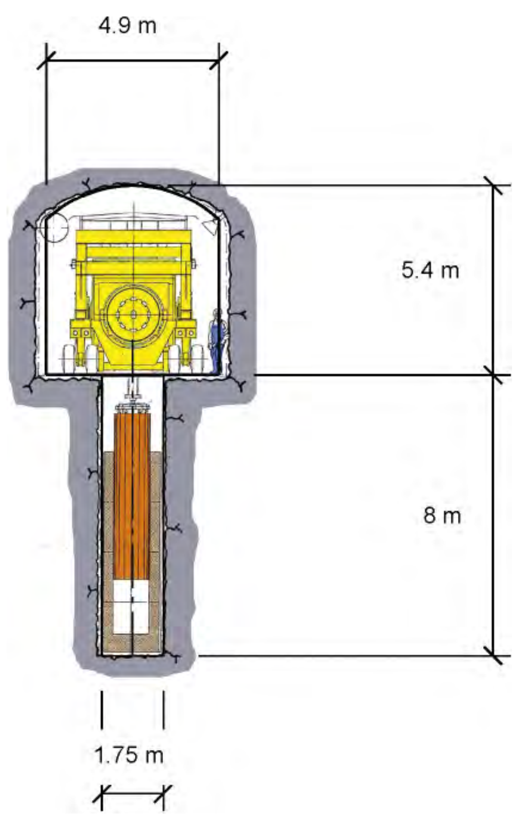


Figure 4-3. *Cross-section of a deposition tunnel and a deposition hole together with the equipment for installation of canisters (given by SKB).*

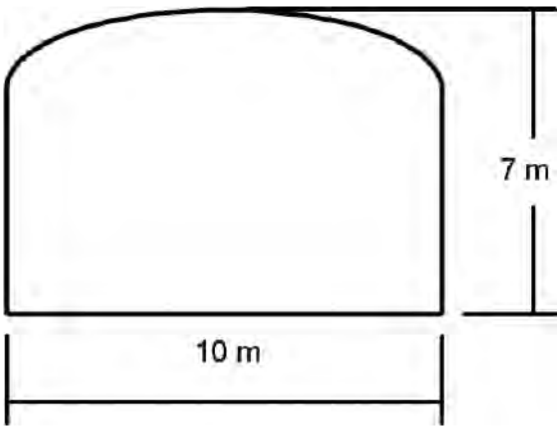


Figure 4-4. *Cross-section of a main tunnel (according to /SKB 2002/).*

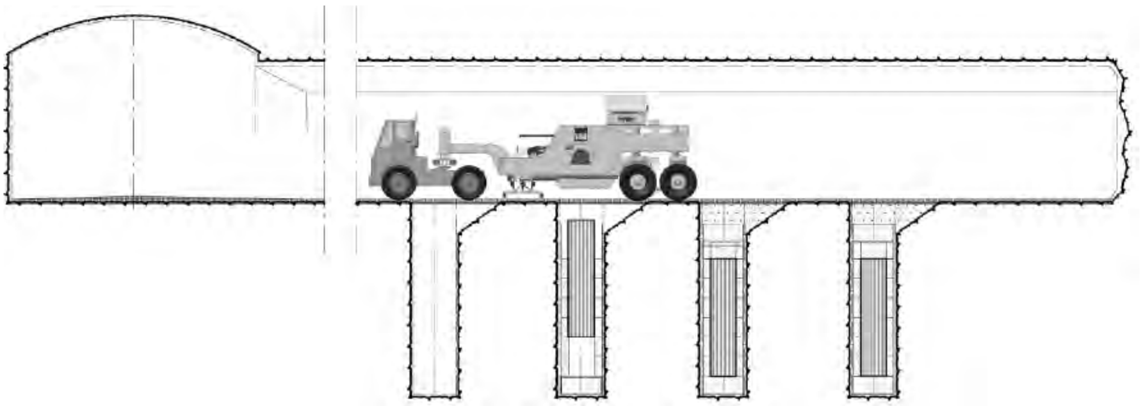


Figure 4-5. *Illustration of the intersection of a main tunnel and a deposition tunnel together with the equipment for installation of canisters (given by SKB).*

4.3 Distance between deposition tunnels and between deposition holes

4.3.1 Introduction

In design step D1 the minimum distance between deposition tunnels and between deposition holes was determined with respect to the highest permissible temperature on the canister surface.

In determining the minimum distance between deposition tunnels and between deposition holes, the following issues should be taken into account:

- Thermal properties of the rock mass.
- Initial temperature at repository depth.
- Heat power of the canisters.
- Buffer and its thermal properties.

According to /SKB 2004a/ a distance between deposition tunnels of 40 m was to be assumed. This distance is in accordance with Layout E /SKB 2002/.

The distance between deposition holes was determined based on the following premises, which are given in /SKB 2004a/:

- The verification shall be carried out by application of the graph in Figure 4-6. This graph is based on an initial canister heat output of 1,700 W/canister, a thermal conductivity in the buffer of 1.0 W/mK, an initial rock temperature of 15°C, and a heat capacity in the rock of 2.08 MJ/m^{3}K.
- The highest permissible temperature on the canister surface is set to 100°C according to /SKB 2004a/. However, taking into account the initial air gap between buffer and canister together with uncertainties in input data (thermal conductivity of the rock, heat capacity of the rock and thermal conductivity of the buffer) a reduced temperature of 80°C shall be used. It may be possible to allow for 10°C higher temperature when evaluating the canister spacing (see Section 2.3.2). However, SKB decided not to utilise this opportunity, and consequently not to revise the study at this late stage.
- When applying Figure 4-6, the maximum permissible temperature on the canister surface (80°C) shall be adjusted linearly with respect to the initial temperature of the rock. This means that if the initial temperature in the rock differs from the initial temperature, on which Figure 4-6 is based (15°C), the maximum permissible temperature in the graph is parallel-shifted accordingly. For example, if the temperature of the bed rock is 13°C (instead of 15°C) at repository level, the maximum permissible temperature is 82°C.
- Mean values for the thermal conductivity and initial temperature of the rock shall be used at actual depth.
- In order to analyse the sensitivity due to variation in the mean value of the thermal conductivity of the rock, the minimum distance between deposition holes was also analyzed when the mean value deviates ±5% from the mean values given by /SKB 2005a/.

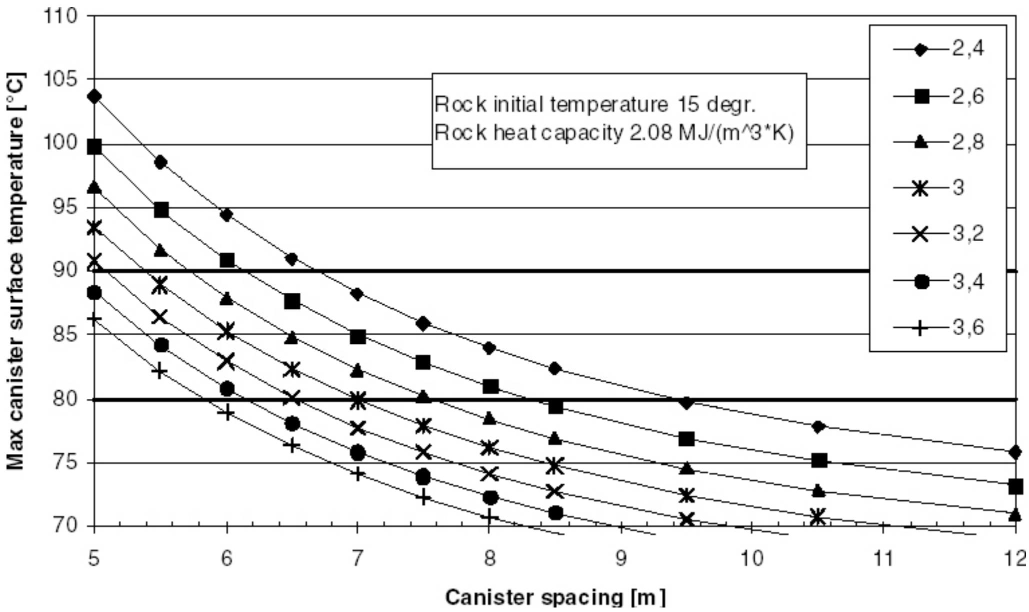


Figure 4-6. *Maximum temperature on canister surface as a function of the distance between deposition holes and the thermal conductivity (W/mK) of the rock (from /SKB 2004a/).*

4.3.2 Input data and assumptions

For the verification of the distance between the deposition holes, input data regarding the thermal conductivity and the initial temperature of the rock mass was given in /SKB 2005a/.

1. Initial temperature of rock mass

The following mean values of the temperature at studied depths are given in /SKB 2005a/.

- Initial temperature, depth 400 m: 10.6°C.
- Initial temperature, depth 500 m: 11.7°C.

2. Thermal conductivity of the rock mass

According to /SKB 2004a/ the mean value of the thermal conductivity should be used when calculating the distance between deposition holes. The anisotropy described in /SKB 2005a/ should thus not be considered. In /SKB 2005a/ the mean value of the thermal conductivity is 3.55 W/m·K for rock domain RFM029. This value of the thermal conductivity is valid in a "canister scale" and for a temperature of 20°C.

Since the temperature around the canister rises due to heat generated within the canister, the temperature will exceed 20°C, and consequently a reduced value of thermal conductivity should be used in the analyses. In /SKB 2005a/, Table 7-2, a –10% reduction per 100°C is given. There is, however, no guidelines in /SKB 2004a/ of the temperature for which the thermal conductivity should be calculated. It was therefore decided that the value of 3.55 W/m·K should be used for the further analyses.

The following values of thermal conductivity were thus used for analysis of the distance between the deposition holes:

- Mean value: 3.55 W/m·K.
- Mean value –5%: approx. 3.4 W/m·K.
- Mean value +5%: approx. 3.7 W/m·K.

4.3.3 Execution

Since Figure 4-6 is valid for an initial temperature of 15°C, the maximum permissible temperature on the canister surface shall be increased by the same number of degrees as the initial temperature is below 15°C /SKB 2004a/. This will give the following maximum permissible temperatures on the canister surface:

- Depth 400 m: approx. 84.5°C.
- Depth 500 m: approx. 83.5°C.

4.3.4 Result

The minimum distance between the deposition holes is presented in Table 4-1. For values of thermal conductivity higher than 3.6 W/m·K the curves in Figure 4-6 have been extrapolated based on the difference between the curves for 3.4 W/m·K and 3.6 W/m·K.

Table 4-1. Distance between deposition holes.

Depth	Thermal conductivity 3.4 W/(m·K)	Thermal conductivity 3.55 W/(m·K) (mean value)	Thermal conductivity 3.7 W/(m·K)
400 m	5.5 m	5.3 m	5.1 m
500 m	5.6 m	5.4 m	5.3 m

4.3.5 Conclusions and discussion

The distance between deposition tunnels is set to 40 m in accordance with the premises stated in /SKB 2004a/.

The result of the analysis of the distance between deposition holes indicates the following:

- The difference in the distance between deposition holes is small for the depths 400 m and 500 m.
- The distance between the deposition holes should be in the range of 5.1–5.6 m for both depths.
- The distance between deposition holes will probably be some decimetres longer, if the thermal conductivity is assessed with respect to the rise in temperature in the rock mass due to the heat generated within the canister (assuming a temperature of 60°C in the rock mass).
- It may be possible to allow for 10°C higher temperature when evaluating the canister spacing (see Section 2.3.2). However, SKB decided not to utilise this opportunity, and consequently not to revise the study at this late stage.

With respect to the early stage in the design process and uncertainties in the input data, SKB decided to base the subsequent design in step D1 on a spacing of 6 m between canisters according to Layout E /SKB 2002/ (see Section 2.3.2).

The distance between deposition holes is a factor which should also be considered in terms of rock mechanical effects. A short distance between deposition holes together with high rock stresses will lead to higher stresses around the deposition holes, which may lead to an increase risk for loss of deposition holes. Loss of deposition holes will be analysed in detail in Section 4.5.

4.4 Orientation of deposition tunnels

4.4.1 Introduction

According to /SKB 2004a/ the orientation of deposition tunnels is to be determined in design step D1 with the objective of minimising the

1. quantity of water leakage into deposition tunnels and deposition holes,
2. risk of spalling in deposition tunnels,
3. volume of potentially unstable wedges in deposition tunnels and deposition holes.

According to /SKB 2004a/ in design step D1 the orientation of the deposition tunnels should be chosen primarily so that the requirements in points 1–3 above are met in the sequence mentioned. If the tunnel orientation chosen in accordance with point 1 is judged to lead to unmanageable spalling problems according to point 2 and/or a large volume of potentially unstable wedges according to point 3, a compromise should be sought so that such problems are manageable from a stability point of view. A small volume of potentially unstable wedges in deposition holes should be given priority over a small volume of potentially unstable wedges in deposition tunnels. Similarly, low water seepage into deposition tunnels should be given priority over minimum water seepage into deposition holes.

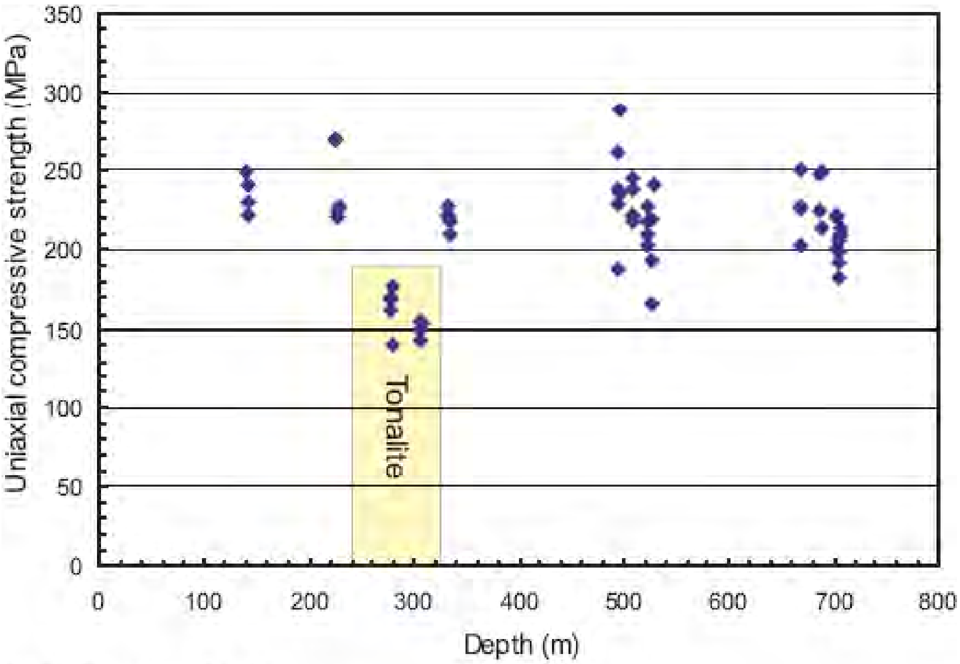
According to /SKB 2004a/ analyses of water leakage into deposition tunnels and deposition holes (according to point 1) should be executed by both analytical and numerical methods. No anisotropy data was presented in /SKB 2005a/ and according to the flow characteristics reported in /SKB 2005a/, the rock matrix is also practically free of flowing channels. The inflow of water to deposition tunnels and deposition holes were thus expected to be very small and not influenced by the orientation of the tunnels. No calculations were thus justified in this design step.

The choice of tunnel orientation with regard to the risk of spalling in deposition tunnels (according to point 2) was determined by stress analyses, which are presented in /Martin 2005/. The main deviation from /SKB 2004a/ was that the analysis was performed for one circular deposition tunnel assuming a uniform stress distribution. Consequently the influence of the actual geometry of the deposition tunnel and connecting main tunnel was not included. This modification was justified by new findings regarding the spalling process achieved from recent research work, for example at Äspö Hard Rock Laboratory. The stress analyses are described briefly in the following. Further details on the analyses are provided in /Martin 2005/. In addition numerical analyses were executed, using a three dimensional model, in order to study the influence of the horse shoe shape of the tunnels.

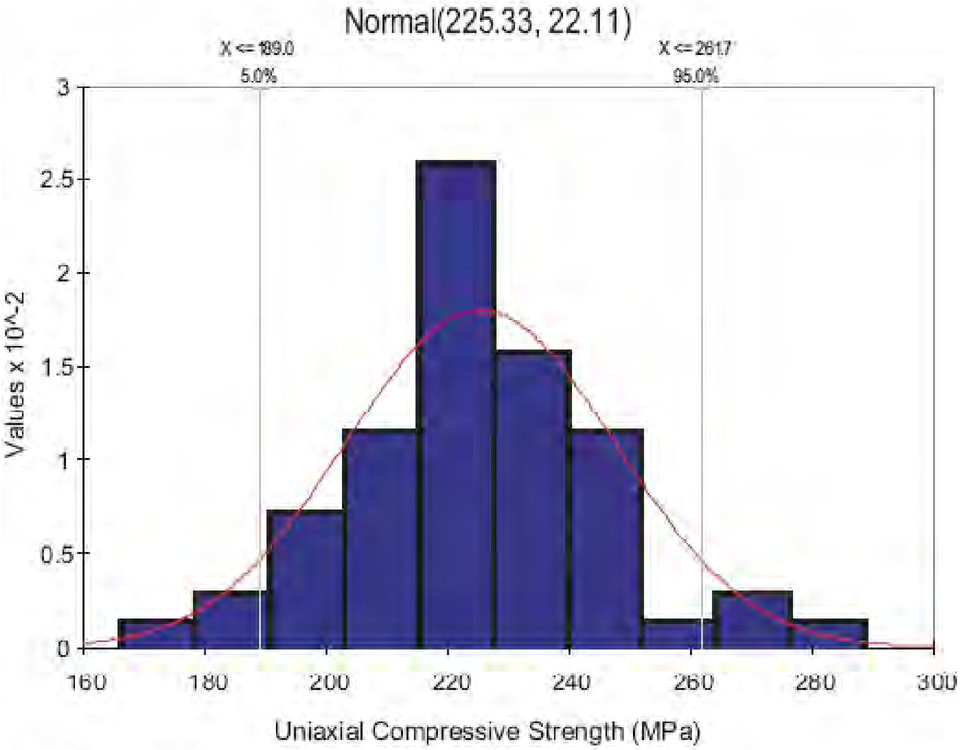
The selection of tunnel orientation considering the volume of potentially unstable wedges in deposition tunnels and deposition holes (according to point 3) was based on "preliminary data" according to the site description model version 1.1 /SKB 2004c/. The reason for this was that analyses based on data from the site description model version 1.1 indicated that orientation of the deposition tunnels will not be influenced by wedge stability analyses /Martin 2005/. According to /Martin 2005/ any potentially unstable wedges could be adequately handled by standard rock support systems. No further analyses based on information in /SKB 2005a/ were thus executed regarding potentially unstable wedges in design step D1.

4.4.2 Input data and assumptions

In situ stresses and the results of uniaxial compression tests according to /SKB 2005a/ were used for analyses of the risk of spalling in deposition tunnels. Table 4-2 indicates the in situ stresses and Figure 4-7 (a) and (b) illustrates the input data for uniaxial compressive strength (UCS). No correlation between stress and strength are assumed for the simulations, i.e., the highest stress can be associated with the lowest strength. According to /Martin 2005/ this may not be the case in reality, as the highest in-situ stresses are often found in the most competent rock mass and the lowest stresses in highly fractured rock masses. This level of information is currently not available, and for these analyses this issue is not addressed /Martin 2005/.



(a) UCS versus depth



(b) UCS histogram

Figure 4-7. *Uniaxial compressive strength (UCS) versus depth from ground surface (top figure) and normal distribution fit to all strength data. Tonalite has been excluded in the normal distribution (from /Martin 2005/).*

Table 4-2. In situ stresses used for spalling analyses. The gradient and variability are from /SKB 2005a/ and are valid for depths between 350 and 650 m. It should be noted that σ_{Hmax} and σ_{hmin} correspond to the stresses denoted σ_H and σ_h in /SKB 2005a/. z is the depth from the ground surface in metres (table from /Martin 2005/).

	σ_{Hmax}	σ_{hmin}	σ_v
Gradient (MPa/m)	35+0.02×z	19+0.025×z	0.0265×z
Variability	±10%	±20%	±0.0005×z
Depth (m)	(MPa)	(MPa)	(MPa)
350	42	28	9.3
450	44	30	11.9
550	46	32	14.6
650	48	35	17.2

In-situ stresses, given as "preliminary data", were used in numerical stress analysis that was executed to study the influence of the shape of the tunnels. The difference between the in-situ stresses, given as "preliminary data" and the data presented in /SKB 2005a/, is such that the minor horizontal stress is higher in /SKB 2005a/ (see Table 2-1 in Chapter 2).

The rock mass spalling strength has to be defined in order to determine whether spalling will occur. According to /Martin 2005/ two major experiments have been carried out since 1990 in order to develop a methodology for assessing the rock mass spalling strength of crystalline rocks: (1) AECL's Mine-by Experiment, and (2) Äspö Pillar Stability Experiment (see /Andersson and Eng 2005/). The rock mass spalling strength, i.e. the tangential stress required to initiate spalling on the boundary of an opening in crystalline rock is based on the findings from these experiments. The rock mass spalling strength (σ_{sm}) indicated in Table 4-3 was based on these findings and used to evaluate the potential for spalling from the stress analyses.

The spalling criteria based on the rock mass spalling strength (σ_{sm}) are indicated in Table 4-3. The ratio between rock mass spalling strength (σ_{sm}) and mean laboratory uniaxial compressive strength (UCS_m) provides a method for evaluating the rock mass spalling strength when only the laboratory uniaxial compressive strength is known.

It should be noted that according to /Martin 2005/, spalling is generally defined as the formation of stress-induced slabs on the boundary of an underground excavation (see Figure 4-8). It initiates in the region of maximum tangential stresses and normally results in a V-shaped notch at the boundary of the opening. For circular underground openings ranging in diameter from 1 to 5 m the slabs falling out can vary in thickness from a few millimetres to a few centimetres, depending on the actual stress magnitudes.

Table 4-3. Spalling criteria (from /Martin 2005/).

	σ_{sm} (MPa)	$\sigma_{sm}/UCSm$
Coarse & medium grained crystalline rocks	120 ± 5	0.57 ± 0.02
$UCSm$= Mean laboratory uniaxial compressive strength σ_{sm} = Rock mass spalling strength $\pm$ = Minimum, maximum		

Figure 4-8. *Example of spalling observed around a 1.8 m diameter borehole in the Äspö Pillar Stability Experiment (from /Martin 2005/).*

4.4.3 Execution

Risk of spalling

The risk of spalling in deposition tunnels was assessed using a traditional factor of safety approach. If the magnitude of the maximum tangential elastic stresses ($\sigma_{\theta\theta}$) on the boundary of an underground opening reaches the rock mass spalling strength (σ_{sm}), the factor of safety is expressed as:

Factor of Safety = $\sigma_{sm} / \sigma_{\theta\theta}$ Equation 4-1

It should be noted that the mean uniaxial compressive strength (UCS_m) is used to establish the rock mass spalling strength (σ_{sm}) according to the criteria given in Table 4-3.

The maximum tangential elastic stress was calculated assuming a circular opening. On the boundary of a circular opening in a continuous, homogeneous, isotropic and linearly elastic rock, these stresses can be expressed using the Kirsch equation for plane strain:

$\sigma_{\theta\theta} = 3\sigma_{max} - \sigma_{min}$ Equation 4-2

where σ_{max} and σ_{min} are respectively the maximum and minimum far-field principal stresses in the plane of analysis.

The factor of safety and probability of spalling were determined using the software @Risk, which consists of a series of macros for EXCEL that conducts Monte Carlo simulations (10,000 simulations were used). The output of factors of safety was expressed as a probability distribution. The probability of spalling corresponds to the part of the simulations resulting in a factor of safety less than 1.0. An illustration of the calculation flow chart is presented in Figure 4-9.

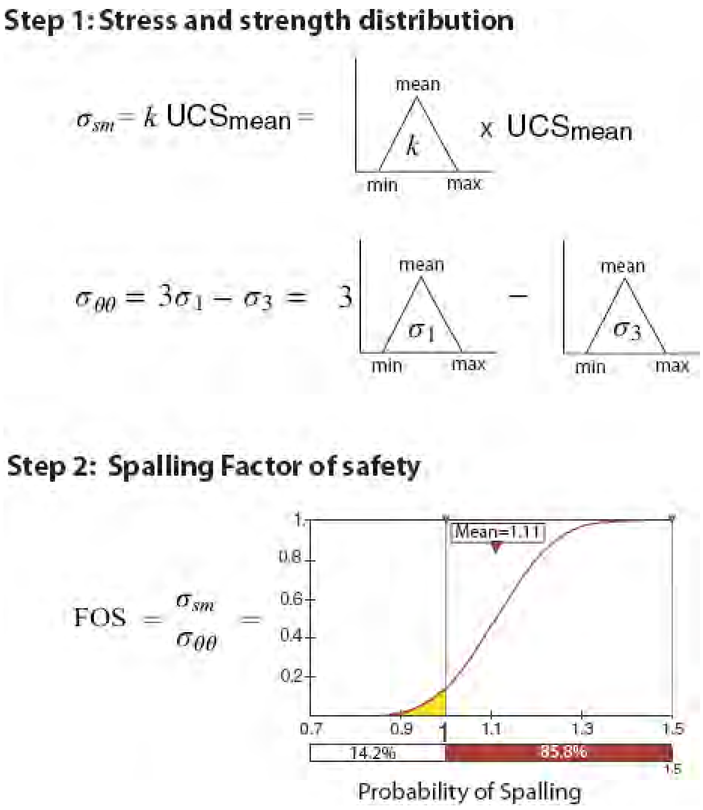


Figure 4-9. *Illustration of the calculation flow chart used to establish spalling factor of safety (FOS), the probability of spalling. The factor k (σ_{sm}/UCS_m) is given in Table 4-3 and σ_1 and σ_3 correspond to the maximum and minimum far-field stresses in the plane of analysis (from /Martin 2005/).*

The stresses were evaluated parallel (0 degrees), at 45 degrees and perpendicular (90 degrees) to the maximum horizontal stress.

4.4.4 Results

Regarding the risk of spalling in deposition tunnels, Figure 4-10 shows the calculated mean value of the factor of safety (mean Factor of Safety) and probability of spalling for different depths and orientations of the deposition tunnel relative to the maximum horizontal stress.

From Figure 4-10 it can be seen that the probability of spalling is very low if the tunnels are oriented parallel or at 45° to the maximum horizontal stress. On the other hand if the tunnels are perpendicular (90°) to the maximum stress there will be a 5% probability of spalling at 400 m depth and a 15% probability at 500 m depth.

In order to illustrate the effect of non-circular geometry, the results of numerical stress analyses obtained using the software Examine 3D are shown in Figure 4-11 and Figure 4-12. It should be mentioned that these calculations were made for 450 m depth and based on “preliminary data” with respect to the in situ stress (see Table 2-1 in Chapter 2). For comparison the maximum tangential stress calculated using Equation 4-2 is 120 MPa for tunnels perpendicular to the maximum horizontal stress and the rock mass spalling strength (σ_{sm}) is 120±5 MPa according to /Martin 2005/.

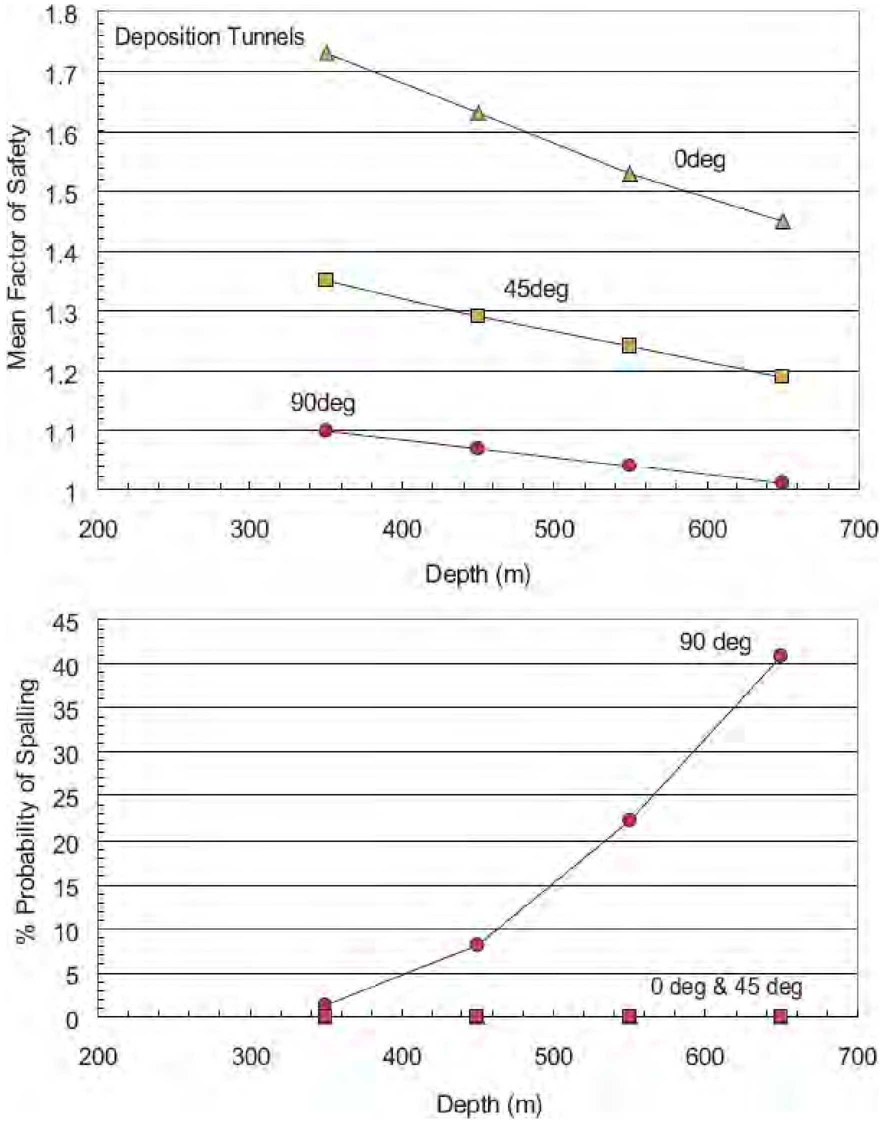


Figure 4-10. *Factor of safety and probability of spalling for deposition tunnels at repository levels 350–650 m. The notation of 0 degrees, 45 degrees and 90 degrees refers to the orientation of the deposition tunnel relative to the maximum horizontal stress (from /Martin 2005/).*

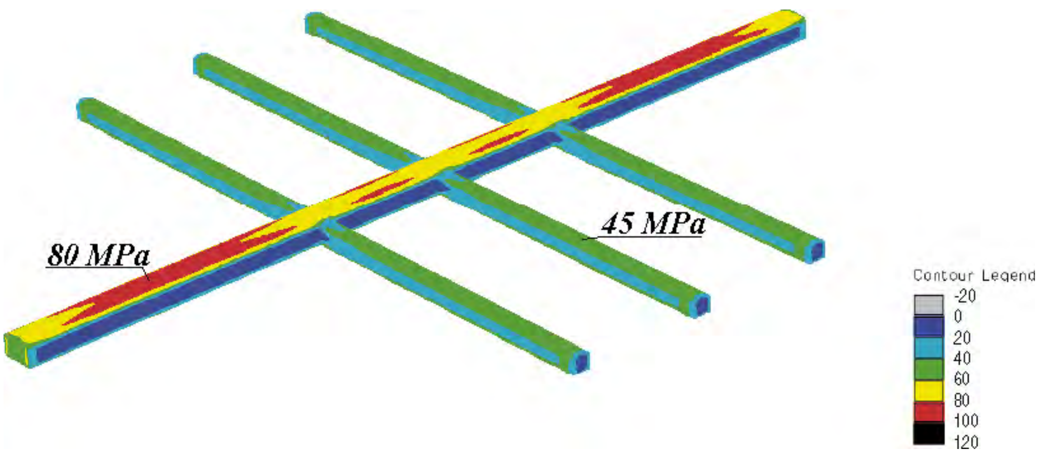


Figure 4-11. *Calculated stresses around main tunnel and deposition tunnels with deposition tunnels parallel to the maximum horizontal stress (the values indicated are maximum stresses in the central part of deposition tunnels and in main tunnels), 450 m depth.*

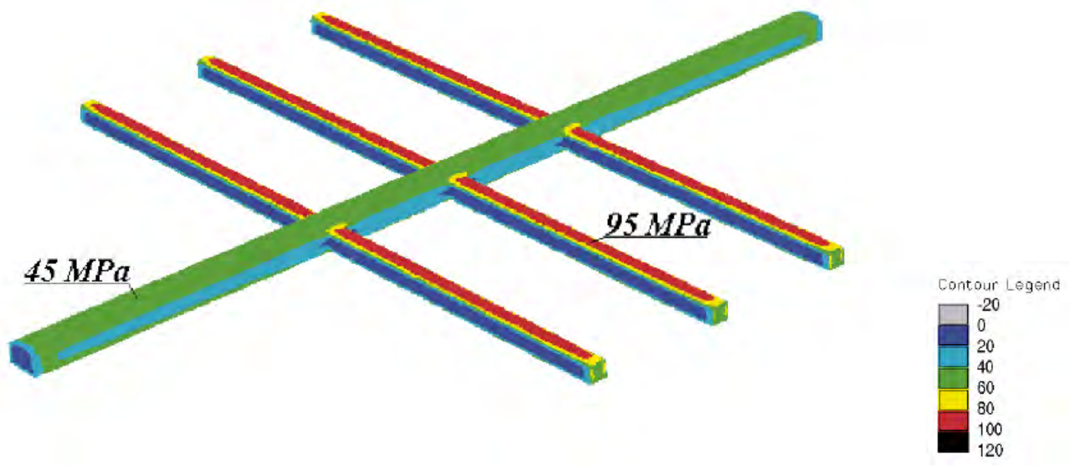


Figure 4-12. *Calculated stresses around main tunnel and deposition tunnels with deposition tunnels oriented 90° to maximum horizontal stress (the values indicated are maximum stresses in the central part of deposition tunnels and in main tunnels), 450 m depth.*

4.4.5 Conclusions

According to /SKB 2005a/ the same hydraulic conductivity shall be used for all tunnel orientations when calculating the seepage. The conclusion is that the orientation of the deposition tunnels can be determined without considering the water inflow to the tunnels. The amount of inflow into deposition tunnels is presented in Section 7.4.

The orientation of the deposition tunnels with regard to the risk of spalling should be parallel to the largest horizontal stress. It should be noted that, if necessary, the deposition tunnels can be oriented at an angle up to at least 45° relative to the maximum horizontal stress without any increased probability of spalling.

It should also be noted that if the deposition tunnels are parallel to the maximum horizontal stress, spalling may arise in the main tunnels, which will be oriented approximately perpendicular (70°–80°) to the maximum horizontal stress. Assuming circular tunnels, there will be a 5% probability of spalling at 400 m depth and a 15% probability at 500 m. From supplementary numerical stress calculations (see Figure 4-11 and Figure 4-12) it can however be concluded that when considering the horse shoe shape of the main tunnels, such probability of spalling is probably an overestimation of the actual risk of spalling. In forthcoming design steps numerical calculations should thus be performed in order to evaluate the risk of spalling in tunnels. In these calculations the variability of in situ stresses should also be taken into account.

The orientation of the deposition tunnels is not influenced by potentially unstable wedges and any potentially unstable wedges could be adequately handled by standard rock support systems (see /Martin 2005/).

The overall recommendation regarding the orientation of the deposition tunnels is that they should be oriented parallel or only slightly inclined to the maximum horizontal principal stress. This recommendation is based on the risk of increased loss of holes due to spalling (detailed analyses of the risk of spalling in deposition holes are presented in Section 4.5). The inflow of water is not considered to influence the orientation of the deposition tunnels.

4.5 Loss of deposition holes

4.5.1 Introduction

According to /SKB 2004a/, estimation of the loss of deposition holes in design step D1 should take account of:

1. minimum permissible distance between deposition holes and stochastically determined fractures/fracture zones with radius r > 100 m,
2. rate of water leakage into deposition holes,
3. outfall of wedges in deposition holes,
4. risk of spalling in deposition holes.

According to /SKB 2004a/ the estimation of loss of deposition holes should also be based on the chosen tunnel orientations indicated in Section 4.4, the distance between deposition tunnels and the distance between deposition holes according to Section 4.3 as well as on the geometries of deposition tunnels and deposition holes (theoretical rock contour) according to Section 4.2.

The analysis of loss of holes taking into account the distance between deposition holes and stochastically determined fractures/fracture zones (according to point 1) was executed by SKB by means of an analytical method for estimating canister/fracture intersections (see /Hedin 2005/). This method calculates the probability of a randomly placed canister being intersected by a fracture exceeding a specified radius, given the distributions of fracture radius and fracture orientations. It should be noted that these analyses were made with some deviations from /SKB 2004a/ (see Table 2-2 in Chapter 2).

According to /SKB 2004a/, analyses of the rate of water leakage into deposition holes (according to point 2) should be executed using numerical methods based on DFN-data (hydro DFN-models) and a limit value regarding the inflow of 10 l/min into a single deposition hole. These analyses should also include a sensitivity analysis with the limit value set to 1 l/min. The hydro-DFN models presented in /SKB 2005a/ show that the rock matrix is practically free of flowing channels. It is therefore unlikely that the water leakage into any single deposition hole will exceed the limit value 10 l/min. This conclusion is also supported by results from numerical analysis, presented in Section 7.4.2, which indicates that an open repository will have a total leakage less than 240 l/min. No calculations were thus justified, and the loss of holes due to water leakage into deposition holes was therefore set to zero in this design step.

The loss of deposition holes due to wedge outfall (according to point 3) was not analysed with input data from /SKB 2005a/, since the probability of having unstable wedges was considered to be insignificant (see /Martin 2005/). The loss of holes due to wedge breakout was therefore set to zero in this design step.

The loss of deposition holes due to the risk of spalling in deposition tunnels (according to point 4) was determined by stress analyses, which are presented in /Martin 2005/. The main deviation from /SKB 2004a/ was that the analysis was executed for a single deposition hole assuming a uniform stress distribution and consequently the influence of the deposition tunnel and adjacent deposition holes was not included. This modification was justified by the fact that new findings regarding the spalling process had been achieved from recent research work. The stress analyses are described briefly in the following sections. Further details on the analyses are provided in /Martin 2005/. In addition a supplementary analysis was executed based on the data given in /SKB 2005a/. This analysis used a three-dimensional model in order to analyse the influence of a deposition tunnel and adjacent deposition holes.

4.5.2 Input data and assumptions

Loss of deposition holes due to stochastically determined fractures

The analysis of loss of deposition holes due to stochastically determined fractures was based on the DFN model given in /SKB 2005a/ (see Table 4-4). In Table 4-4 x_r (in /Hedin 2005/ denoted r_0) is the smallest fracture radius considered in the model, and k_r (in /Hedin 2005/ denoted k) is a dimensionless model parameter. Each fracture set is characterised by specified values of these parameters (see also /La Pointe et al. 2005/). Detailed descriptions of the implication of parameters are reported in /Hedin 2005/.

It should be noted that the input data are not influenced by the depth, and that only fractures with a radius, r, in the range of 50–600 m were considered in the analyses executed in accordance with the methodology stated in /Hedin 2005/.

Table 4-4. Fracture size parameters for fracture sets used for the calculation of loss of deposition holes due to stochastically determined fractures (data from Table 5-34 /SKB 2005a/).

Set	Size model	k_r (median)	x_r (median)
NS	Power law	2.88	0.28
NE	Power law	3.02	0.25
NW	Power law	2.81	0.14
EW	Power law	2.95	0.15
SubH	Power law	2.92	0.25

Loss of deposition holes due to the risk of spalling

Regarding the magnitude of the stresses and the strength of the rock mass, the same input-data was used as in the analyses presented in Section 4.4. The in situ stresses together with the assumed variations are presented in Table 4-2 and the input data for the uniaxial compressive strength (UCS) are presented in Figure 4-7. Variations in the stress magnitude and the strength were assumed to be triangular distributed and uncorrelated (see /Martin 2005/). The criteria for spalling are based on the rock mass spalling strength (σ_{sm}) (see Table 4-3). More details regarding the input-data are given in Section 4.4.2.

Regarding the analyses based on “preliminary data” the difference in stresses compared to /SKB 2005a/ is that the minor horizontal stress is higher in /SKB 2005a/ (see Table 2-1 in Chapter 2). The analyses were executed for two different orientations of the major horizontal stress, parallel with the deposition tunnel and 20° to the deposition tunnel. Variations in the stress magnitude and stress orientation were assumed to be normally distributed and uncorrelated.

For the supplementary analysis in situ stresses from /SKB 2005a/, corresponding to the depth 450 m according to Table 4-2, were used as input data for these calculations. This analysis was executed for two different orientations of the major horizontal stress, parallel with the deposition tunnel and 20° to the deposition tunnel. However, no variations regarding the magnitude and orientation of the in-situ stresses were considered in the supplementary analysis.

The diameter of the deposition hole was set at 1.8 m in both analyses.

4.5.3 Execution

Loss of deposition holes due to stochastically determined fractures

The assessment of loss of holes due to stochastically determined fractures/fracture zones is based on an analytical method presented in /Hedin 2005/. The method is described in /Hedin 2005/ as follows.

"The fracture population is described as several fracture sets, each with specified distributions of sizes and orientations. The distributions of sizes and orientations within a set are assumed to be uncorrelated, an important prerequisite for the method. Furthermore, all fractures are assumed to be infinitesimally thin, circular discs.

The calculation model essentially consists of two factors, of which one is related to the fracture size distribution and the other to the distribution of fracture orientations.

The fracture intensity and fracture size distribution will correspond to an average fracture area per unit volume of host rock, a P_{32}-value, expressed in units of m^2/m^3. Part of the P_{32}-value will be associated with fractures that exceed a certain size and this critical part, a, is determined for a power-law size distribution. For a given rock volume, V m^3, the total critical fracture area is thus equal to a·V m^2.

The cylindrical canisters are to be deposited with their axes oriented vertically. If a fracture of critical size is horizontal, then canister centre-points should not be positioned within half the canister height on either side of the fracture in order to avoid intersection, giving a total canister intersection zone width around a horizontal fracture equal to the canister height. The corresponding width for a vertical fracture equals the canister diameter. For a distribution of fracture orientations, a mean intersection zone width, <L> can be calculated.

Given the mean total critical fracture area, a, and the mean intersection zone width, <L>, a mean value of the volume of rock within which canisters would intersect fractures of critical size is calculated as a·V·<L>. The fraction of the total volume to be avoided, ε, is thus a·<L>. The quantity ε can thus be interpreted as the probability that a randomly emplaced canister will be intersected by too large a fracture."

In the analyses based on "preliminary data", the DFN model was generated in a model volume having the dimensions 2,300 m × 2,300 m × 2,000 m. Into this model volume, a 300 m long deposition tunnel was located horizontally in five different orientations compared to the major horizontal stress. The criteria for determining the point, at which a deposition hole is lost, were in these analyses based on the assumptions according to /SKB 2004a/ (see Table 2-2 in Chapter 2). The numerical calculations were executed using the software FracWorksXP for the generation of DFN simulations, and specially developed applications for calculating the loss of deposition holes.

Loss of deposition holes due to the risk of spalling

The execution of the analysis was based on the same principles as presented in Section 4.4.3.

It should be noted that the analysis assumes a uniform stress distribution along the deposition hole. In reality the deposition holes are 8 m long and connected to a deposition tunnel, hence the magnitudes of stress along the deposition hole will not be uniform.

Both the calcaulations based on "preliminary data" and the supplementary calculations based on input data from /SKB 2005a/ were performed in order to analyse the influence of a deposition tunnel and adjacent deposition holes. The geometrical model included one deposition tunnel and four deposition holes (Figure 4-13). The software Examine 3D was used for these calculations.

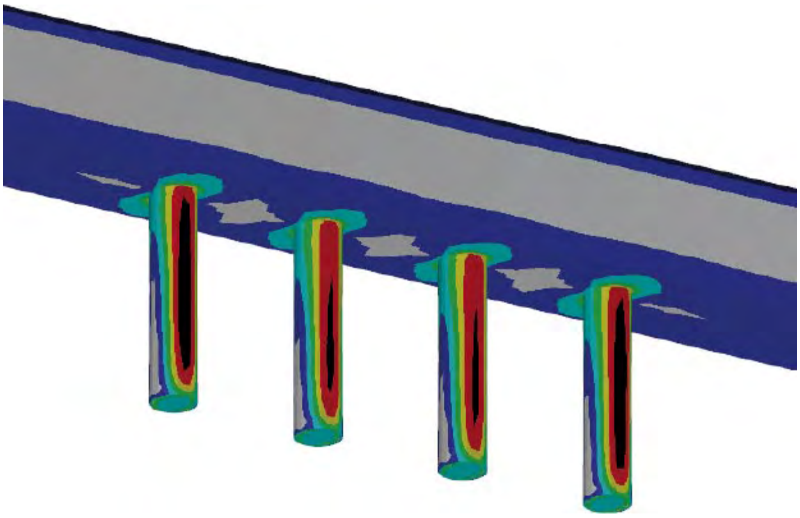


Figure 4-13. *Model used in the analyses based on "preliminary data"and the supplementary analysis based on data from /SKB 2005a/ (the different colours represent intervals of calculated stress magnitude).*

Regarding the analyses based on "preliminary data", input data for the in situ stresses and stress orientations were derived by Monte Carlo simulation (100 simulations). The loss of deposition holes was then derived as the number of calculations resulting in a maximum stress higher than the rock mass spalling strength (σ_{sm}) divided by the total number of calculations (100). The supplementary analysis based on input-data from /SKB 2005a/ was executed for two different orientations of the major horizontal stress (parallel to the deposition tunnel and 20° to the deposition tunnel). However, the supplementary analysis did not consider possible variations in input-data and thus only mean values were used.

4.5.4 Results

Loss of deposition holes due to stochastically determined fractures

The analysis based on input data from /SKB 2005a/ and the analytical method described in /Hedin 2005/, which assessed a loss of deposition holes due to intersections with fractures having a radius, r, in the range of 50–600 m, resulted in a loss of deposition holes of approximately 9%. This can be compared with the results of the analyses based on "preliminary data", which indicated a loss of holes of about 4% independent of the depth.

Loss of deposition holes due to the risk of spalling

Regarding the risk of spalling in deposition holes, the probability of spalling is close to zero for both the depths 400 m and 500 m, according to the results reported in /Martin 2005/ (see Figure 4-14). Consequently the loss of holes due to the risk of spalling is close to zero. This can be compared with the results of the analyses based on "preliminary data", which indicated a loss of holes of about 7% at 400 m depth and about 10% at 500 m depth (with the deposition tunnel parallel to the orientation of the maximum horizontal stress). When comparing the results from the different analyses, it should again be noted that different geometrical models and calculation methods were used in the analyses.

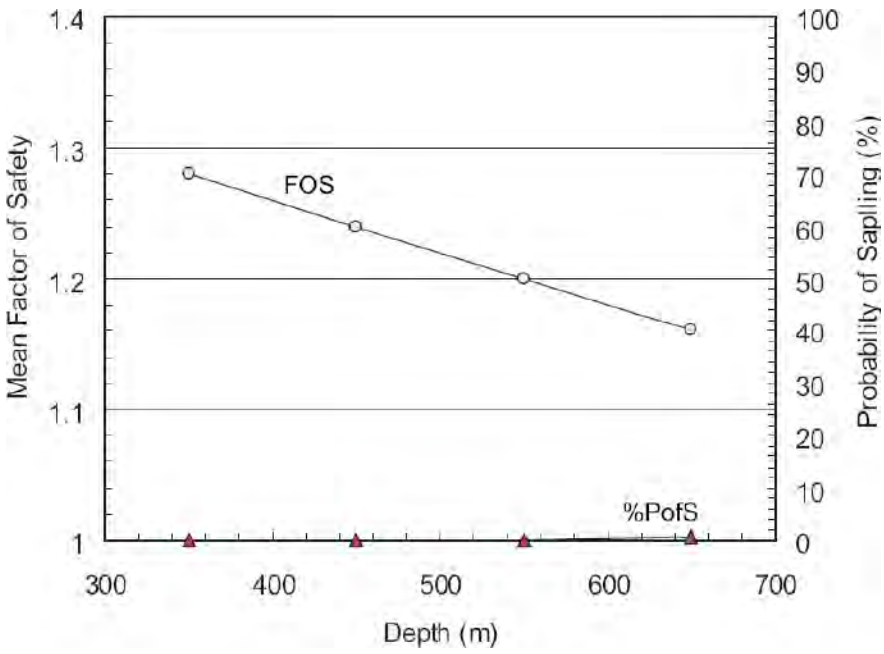


Figure 4-14. *Mean factor of safety and probability of spalling for deposition holes at repository levels 350–650 m (from /Martin 2005/).*

The results of the supplementary calculations, which were based on the input-data given in /SKB 2005a/, are presented in Figure 4-15 and Figure 4-16. For comparison, the corresponding magnitude of the maximum tangential elastic stress ($\sigma_{\theta\theta}$) on the boundary of an underground opening will be 102 MPa according to Equation 4-2. Based on the supplementary calculations it can be concluded that when the deposition tunnel is parallel to the major horizontal stress, the tangential stress calculated by Equation 4-2 is slightly higher than the stresses calculated by Examine 3D in the relevant section of the deposition hole (z > 1.5). However, when the deposition tunnel is not parallel to the maximum horizontal stress, the maximum tangential stress calculated with Equation 4-2 may be lower than the stresses calculated with Examine 3D in the relevant section of the deposition hole (z > 1.5).

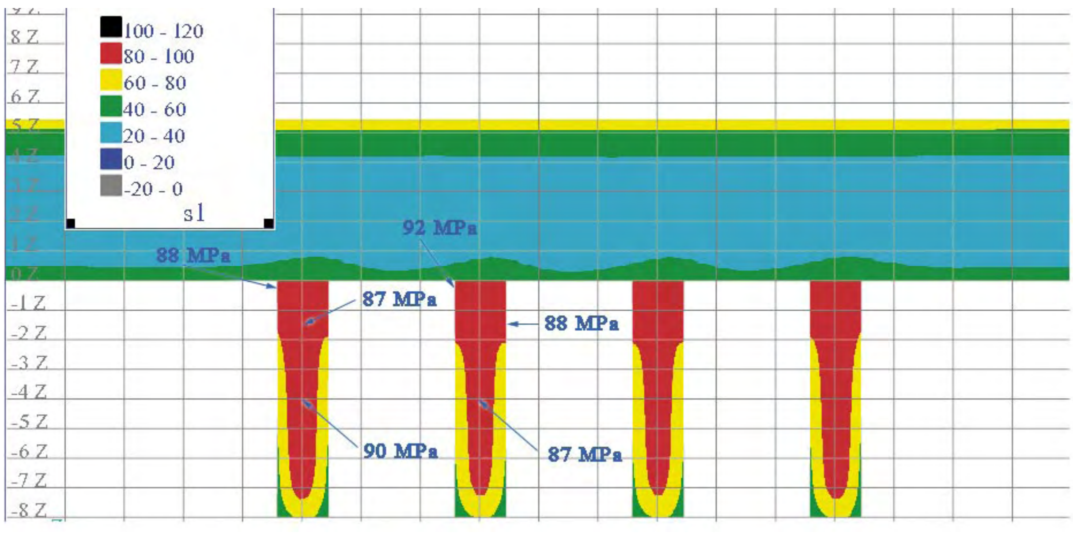


Figure 4-15. *Maximum stresses on the boundary of a deposition hole (s1). Deposition tunnel parallel to maximum horizontal stress, depth 450 m. In-situ stresses according to /SKB 2005a/ (see Table 4-2). The stress given at one specific level (z) corresponds to the maximum stress at that level.*

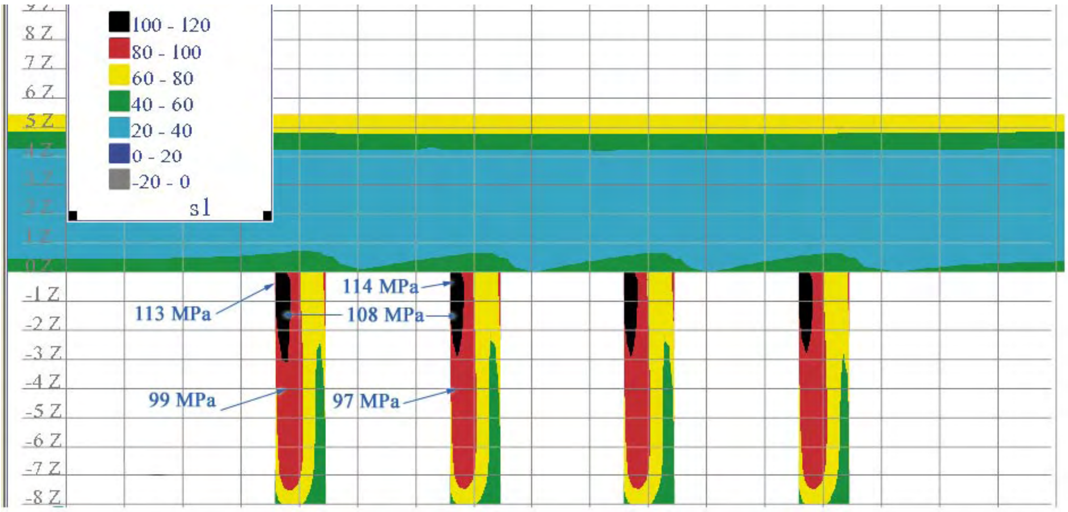


Figure 4-16. *Maximum stresses on the boundary of a deposition hole (s1). Deposition tunnel at 20° to maximum horizontal stress, depth 450 m. In-situ stresses according to /SKB 2005a/ (see Table 4-2). The stresses given at one specific level (z) correspond to the maximum stress at that level.*

It can also be concluded that the stresses will vary along the deposition holes and that higher stresses are encountered in the outer deposition holes compared to the other holes. It should be noted that the top of the canister in the deposition hole is located 1.5 m below the floor of the deposition tunnel.

Table 4-5 presents a summary of the loss of deposition holes due to different factors. For comparison Table 4-5 also includes the loss of deposition holes based on "preliminary data".

Table 4-5. Summary of loss of deposition holes.

Issue	Loss of deposition holes based on /SKB 2005a/ (%)	Loss of holes based on "preliminary data" (%)
Minimum distance between deposition holes and stochastically determined fracture/fracture zones	9*	4**
Water leakage into deposition holes	0	0
Wedge breakout in deposition holes	0	0
Risk of spalling in deposition holes (depth 400 m/500 m)	0	7 (400 m) 10 (500 m)
Summary	9	11 (400 m) 14 (500 m)

* The loss of deposition holes represent the part of deposition holes intersected by fractures having a radius 50 m < r < 600 m.

** Loss of deposition holes according to numerical analyses based on "preliminary data" and criterion according to /SKB 2004a/.

4.5.5 Conclusions

It may be concluded from the analyses of loss of deposition holes that:

- The loss of deposition holes (approximately 9%), which was derived from analyses based on input data from /SKB 2005a/, is due to the possible intersections between deposition holes and stochastically determined fractures/fracture zones.
- The loss of deposition holes due to the risk of spalling was higher when using the "preliminary data" compared to the loss derived from analyses based on input from /SKB 2005a/. On the other hand the loss of deposition holes due to stochastically determined fractures/fracture zones was lower when using the "preliminary data".
- The analyses based on "preliminary data" resulted in a higher total loss of deposition holes compared to the loss derived from analyses based on data from /SKB 2005a/.
- The calculated loss of deposition holes is in principle the same for both 400 m and 500 m depth.

The following conclusions are drawn with regard to the methodology for the analyses:

- The impact of partly different approaches for the analyses presented in the previous sections is shown in Table 4-5. The results clearly indicate that not only data but also assumptions and calculation methods have an influence on the estimated loss of deposition holes. For forthcoming analyses in later design steps it is important that these issues are understood and evaluated.
- The supplementary three dimensional analysis indicates less risk of spalling compared to the results given in /Martin 2005/ when the deposition tunnel is aligned parallel with the major horizontal stress. The analysis also indicates that an alignment of the deposition tunnel of up 20° to the major horizontal stress will not significantly increase the risk of spalling in the deposition holes. In forthcoming design steps it is however recommended that three-dimensional stress analyses should be carried out for those cases where the probability of spalling is significant.
- In the supplementary analysis it is also shown that the outer deposition hole may be subject to the highest stress in the relevant section of the deposition hole. With respect to this issue, it can be pointed out that when a deposition hole is drilled it will accordingly be an outer hole.

Taking into account the analyses of loss of deposition holes it can be concluded that the deposition tunnels should be aligned at an orientation parallel to the maximum horizontal principal stress. The loss of deposition holes may increase if the angle between the deposition tunnel and the maximum horizontal principal stress increases. However, it is assumed that a small angle between the deposition tunnel and the maximum horizontal stress would be possible if necessary, without any significant increase in loss of holes.

4.6 Repository depth

4.6.1 Introduction

The repository should according to /SKB 2004a/ be located within a depth range of 400–700 m below ground surface. The repository depth should be chosen so that:

1. The requirement on the number of canisters to be deposited is met.
2. As favourable conditions as possible are obtained with respect to stability.
3. An efficient and flexible facility is obtained.

4.6.2 Input data and assumptions

The choice of repository depth was based on the analyses presented in Chapter 3 and Sections 4.2–4.4 of Chapter 4. Based on the findings presented in Section 3.2.5, all these analyses were executed for repository depths of 400 m and 500 m.

4.6.3 Execution

The choice of repository depth has been based on weighing the design results together in accordance with /SKB 2004a/, with regard to:

- respect distance to deterministically determined deformation zones,
- design of deposition tunnels, deposition holes and main tunnels,
- distance between deposition tunnels and between deposition holes,
- orientation of deposition tunnels, and
- loss of deposition holes.

4.6.4 Results

The conclusions from the analyses presented in Chapter 3 and Sections 4.2–4.5 of Chapter 4 can be summarized as follows:

- Regarding the respect distance to deterministically determined deformation zones, it was in Chapter 3 concluded that the potential to accommodate the repository is about the same for both 400 m and 500 m depth.
- The design of tunnels and deposition hole is not influenced by the two studied depths (see Section 4.2).
- The distance between deposition tunnels was set to 40 m and the distance between deposition holes was concluded to be 6 m for both 400 m and 500 m depth (see Section 4.3).
- It is preferable to orientate the deposition tunnels parallel with the maximum horizontal stress for both 400 m and 500 m depth. No major difference in the amount of water leaking into the deposition tunnels or risk of spalling in the deposition tunnels is expected for the depths 400 m and 500 m (see Section 4.4).
- The loss of holes will approximately be the same for both 400 m and 500 m depth (see Section 4.5).

From the conclusions presented above and the present knowledge of the site conditions, it is thus obvious that the depth only has a marginal effect on the results of the executed analyses.

4.6.5 Conclusions

Based on findings presented in Sections 4.2 to 4.5 of Chapter 4 it is concluded that the depth 400 m should be chosen as the main alternative for the repository depth, since none of the results obtained so far, justifies a deeper located repository. From the stress analyses presented in Section 4.4 it can be concluded that the probability of spalling will be higher for 500 m depth than for 400 m depth, and thus that a shallow depth should be preferred. A shallow depth is also preferable since this will require shorter access tunnels and shafts. A smaller excavated rock volume will also reduce the construction costs as well as the transportation requirements and the environmental impacts in general.

In order to illustrate the difference between alternative depths a layout has also been proposed for a depth of 500 m.

4.7 Design of other underground excavations

4.7.1 Introduction

Other underground excavations at the repository, excluding the parts of the facility in the deposition areas, include tunnels and rock caverns in the central area, shafts, ramp and transport tunnels (see Figure 1-2). The design of these excavations was undertaken in accordance with /SKB 2004a/ considering:

1. The required space for the activities to be pursued.
2. Stability.

According to /SKB 2004a/ the requirements stated in point 1 will be met if the design of tunnels and caverns in the central area, shafts, ramp and transport tunnels is carried out in accordance with the facility description Layout E /SKB 2002/ with respect to:

- layout of central area,
- dimensions and cross-section profile (theoretical rock contour) of rock caverns and tunnels in the central area,
- length of rock caverns,
- distance between rock caverns,
- dimensions and cross-section profile (theoretical rock contour) of shafts, ramp and transport tunnels.

The requirements stated in point 2 will be met if the shape and cross-sections of other underground excavations are designed in accordance with the facility description Layout E /SKB 2002/, and if rock support is installed in accordance with the conclusions in Chapter 9.

The length of transport tunnels was determined in conjunction with and based on the proposed design of the deposition areas as set out in Chapter 5.

The location of the central area, ramp, shafts and transport tunnels in the repository together with the tunnels in the deposition area are illustrated in Chapter 5.

More detailed design of the configuration of other underground excavations will be carried out in later design steps.

4.7.2 Central area

For the operation of the repository and deposition of canisters a so-called central area will be constructed at the level of the repository. In addition to the transport tunnels the central area includes rock caverns for the following systems and facilities of the repository:

- Elevators for transport of personnel.
- Ventilation.
- Handling of excavated rock.

- Power supply.
- Water handling (e.g. for drainage, fire hydrants, vehicle washing).
- Workshops and storage facilities.
- Handling of canisters with spent fuel.
- Handling of bentonite for the buffer.

It should be noted that some modifications of the central area have been made in relation to Layout E /SKB 2002/. The main modifications include the addition of a silo and a tunnel for handling of excavated rock at the rock mass station, and repositioning of some of the rock caverns. All rock caverns in the central area are about 10–15 m wide and 10–15 m high.

The design and location of tunnels and rock caverns in the central area was undertaken by SKB.

Figure 4-17 indicates the central area including the ramp based on the assumption that the surface facility is located at the so-called residential area. The ramp will be modified if the surface facility is to be divided into two locations, i.e. at the residential area and close to the SFR facility (see Section 4.7.3 in Chapter 4). The caverns in the central area should be oriented parallel to the maximum horizontal stress due to the risk of spalling.

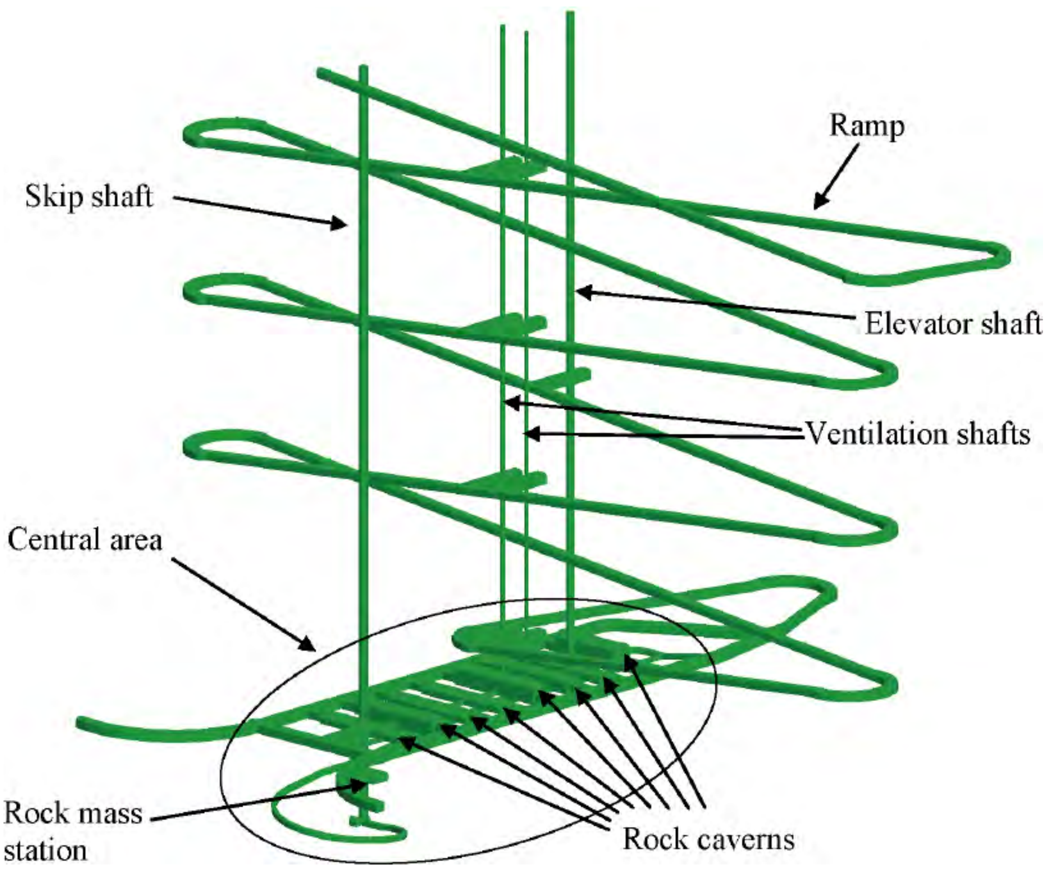


Figure 4-17. *Central area and ramp, assuming the surface part of the repsoitory are located at the residential area (layouts of ramp, central area and shafts are given by SKB).*

4.7.3 Ramp

The main purpose of the ramp is for transport of all material and canisters between the ground surface and the underground repository during construction and operation of the repository. If the surface facilities are to be located partly at the residential area and partly at the SFR facility, the ramp will be modified in order to connect to the surface close to the SFR facility. A proposed location of the ramp relative to the central area is indicated in Figure 4-17, assuming that the surface facilities are located at the so-called residential area. The main parts of the ramp should be oriented parallel to the maximum horizontal stress due to the risk of spalling.

The ramp cross-section is illustrated in Figure 4-18. It should be noted that the ramp cross-section has been modified in relation to the cross-section shown in Layout E /SKB 2002/.

4.7.4 Shafts

All shafts were designed by SKB. Different types of shaft will be constructed for the operation of the deep repository. Assuming that the surface part of the repository is located at the residential area, the following shafts will be connected to the central area:

- Skip shaft for the transportation of excavated rock materials, diameter 5 m.
- Elevator shaft for transportation of personnel, diameter 5.4 m.
- Air intake shaft, diameter 3.5 m.
- Air exhaust shaft, diameter 2.5 m.

The skip shaft will not be constructed if the surface facility is located partly at the residential area and partly at the SFR facility. In this case excavated rock material will be transported via the ramp.

For exhaust air at remote parts of the deposition area, it was assumed that two ventilation shafts of 3 m diameter will be required.

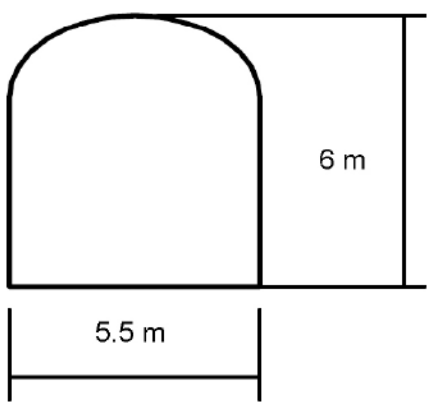


Figure 4-18. *Cross-section of the ramp (given by SKB).*

4.7.5 Transport tunnels

Transport tunnels will connect the central area and different deposition areas together and will be used during construction of the repository and deposition of canisters. The cross-section of the transport tunnels is shown in Figure 4-19.

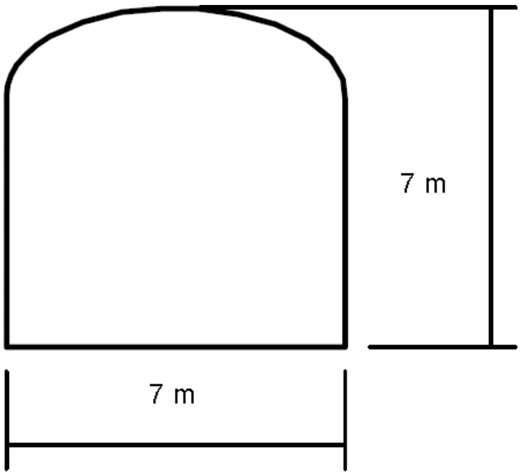


Figure 4-19. *Cross-section of transport tunnels (according to /SKB 2002/).*

5 Layout studies

5.1 Introduction

5.1.1 General

Based on the design work described in Chapters 3–4, alternative layouts for the hard rock facility were prepared. The layouts were designed before the Preliminary site description version 1.2 /SKB 2005a/ was completed, and were thus based on premises derived from "preliminary data" (see Table 4-5). Since the layouts will be slightly conservative with respect to the spatial distribution of the repository, it was considered reasonable not to revise the layouts (see Table 2-2 in Section 2.3.1).

5.1.2 Premises for the layouts

According to /SKB 2004a/ the premises for the layouts are:

1. 6,000 canisters should be accommodated in the repository.
2. 200–400 canisters should be deposited during an initial operation phase.
3. The minimum and maximum length of a deposition tunnel is 100 m and 300 m respectively.
4. The minimum distance between the deposition areas for initial and regular operations should be 80 m.
5. The minimum distance between deposition tunnels and other parts of the facility should be 50 m.
6. Maximum inclination in the access ramp (tunnel) is 1:10.
7. Maximum inclination in other tunnels is 1:100.
8. The layout shall permit construction of the facility to proceed in parallel with investigations, deposition, backfilling and temporary plugging.
9. Emergency evacuation should be possible in two directions in the main and transport tunnels.
10. It should be assumed that 100 canisters per year are to be deposited during the initial operation, and 200 canisters per year during regular operation.

The requirements set out in points 1, 2 and 4–7 above have been strictly applied in the preparation of the layouts. The requirement stated in point 2 was followed by the clarification that the deposition area for the initial operation does not require to be developed as a separate part of the repository as indicated in Layout E /SKB 2002/. In order to commence the initial deposition operation, transport tunnels must be designed so that emergency evacuation is possible in either direction. The area for initial operation should be situated close to the central area.

The requirement regarding the minimum length of a deposition tunnel (point 3) has been applied with some exceptions, resulting in some tunnels being slightly shorter than 100 m.

Regarding point 8 the objective during the layout work was to achieve a suitable subdivision of the deposition area into logical units of similar size in order to facilitate construction of the repository to proceed in parallel with investigation, deposition, backfilling and temporary plugging (concerns both the initial and regular operation). A more detailed description of these activities are given in /SKB 2002/.

In order to fulfil the requirements of point 9, the layout has been prepared making emergency evacuation possible in either direction, and consequently no dead-ends for the transport tunnels or main tunnels are allowed.

The requirement under point 10 has not been considered in this design step. In forthcoming design steps the layout will be optimised with respect to various aspects, such as the rate of deposition.

Furthermore, logistical and functionality issues were considered regarding, for example, transport distances. These issues were however not studied in detail.

Table 5-1 summarises the premises for the layout of the repository and the related results from the design tasks as reported in Chapters 3 and 4.

Table 5-1. Premises for the layout of the repository (resulting from design work presented in Chapters 3 and 4).

Premises	Results from the design work	Section in report
Location of the repository	The repository should be located within the defined "priority site", northwest of deformation zone ZFMNE00A2, southeast of the inlet canal for cooling water at the nuclear power plant, and not too far out from the shore line. Transport tunnels could be located outside rock domain RFM029.	3.1
Respect distance	Deformation zones longer than 3 km: • 100 m from the centre of the zone. Deformation zones shorter than 3 km: • Deposition tunnels may pass these zones. No deposition holes are allowed within a distance from centre of zone equal to half the width of the zone plus 5 m.	3.2
Distance between periphery of first deposition hole and main tunnel	20 m	4.2
Distance between periphery of deposition hole and end of deposition tunnel	8 m	4.2
Design of deposition tunnels, deposition holes, and main tunnels	As per Layout E with some modifications	4.2
Distance between deposition holes and between deposition tunnels.	40 m between deposition tunnels and 6 m between deposition holes	4.3
Orientation of deposition tunnels	Deposition tunnels will be orientated parallel to maximum horizontal stress, NW/SE (145°)*	4.4
Loss of deposition holes	Loss of deposition holes: 400 m – 11%** 500 m – 14%**	4.5
Depth of repository	400 m and 500 m (upper roof level)	4.6
Design of other underground excavations	As per Layout E /SKB 2002/ with some modifications	4.7

*In /SKB 2005a/ the orientation of the maximum horizontal stress is 140°.

**Based on input data from /SKB 2005a/ the loss of deposition holes will be about 9% independent of the depth (see Section 4.5).

In order to achieve an effective layout from a construction and operational point of view with limited environmental impact the following aspects were also considered in the layout work:

- In order to facilitate separation of construction and deposition, the deposition areas on each side of the ventilation shaft should approximately be equal in size. This aspect was considered when determining the location of the ventilation shafts.
- In order to reduce the time and costs of construction as well as the environmental impact, the goal was to achieve a layout with short length of main tunnels and long deposition tunnels instead of many short deposition tunnels. With this approach the number of concrete plugs will be reduced.
- Few intersections between tunnels and deformation zones in order to reduce the amount of grouting and rock support.

Furthermore, the design work was executed based on the following additional premises, which are derived from the findings presented in Chapter 3:

- The boundaries of rock domain RFM029 and the nature reserve should be taken into account.
- Transport tunnels may be located outside rock domain RFM029.
- In order to minimise the difference in elevation between the central area and deposition tunnels, a maximum elevation difference of approximately 10 m was assumed between the various parts of the facility in the deposition area. As a consequence a number of pump sumps must be constructed to handle drainage water.
- In order to limit the need for investigations of the area located below the sea, the extent of the deposition area below the sea should be limited.
- As no geological investigations so far have been carried out below the power plant, and that the related legal conditions are not fully clarified, it has been assumed that no deposition tunnels should be located north-west of the inlet canal for cooling water.

The layouts were prepared using the software Microstation version 08.05.00.64.

5.2 Execution

5.2.1 Possible layouts

Based on the premises outlined in Section 5.1.2 above, the design work was carried out in order to illustrate some possible alternative layouts. Together with SKB it was decided that the layouts for the depths 400 m and 500 m should include the two main alternatives regarding the location of the surface facilities of the deep repository (see Chapter 3). The proposed possible layouts can be summarised as follows:

Layout 1 (base layout): 400 m depth. The surface facility is located within the so called residential area (central area with skip and a spiral access ramp). This layout was considered as the base layout, and thus constitutes the basis for the sensitivity analysis presented in Section 5.3 below. For the base layout the following illustrations have been prepared:

- plan of the deposition areas and central area, including the ramp between the ground surface and the repository depth (Figure 5-1),
- plan of the layout together with an overview of the Forsmark area (Figure 5-2),
- profile (Figure 5-3),
- the layout in three dimensions (Figure 5-4),
- a possible layout of the final repository, including both surface- and underground parts of the repository (Figure 5-5).

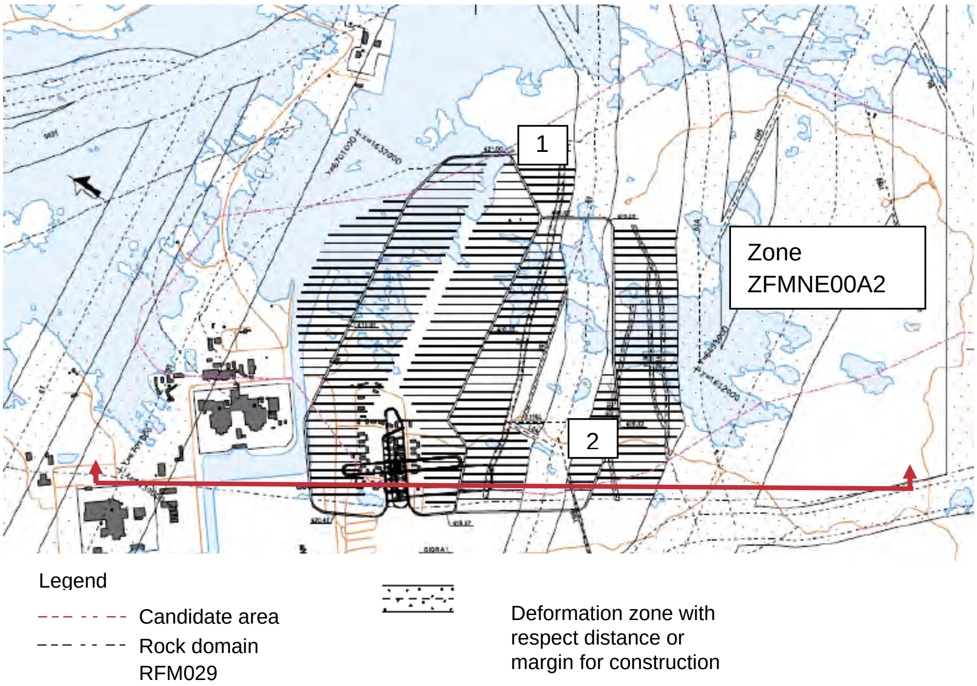


Figure 5-1. *Layout 1 (base layout), depth 400 m (red line shows the approximate location of the profile illustrated in Figure 5-3). The markings 1 and 2 show possible areas for the shafts for exhaust air.*

Figure 5-2. *Illustration of Layout 1 and the Forsmark area.*

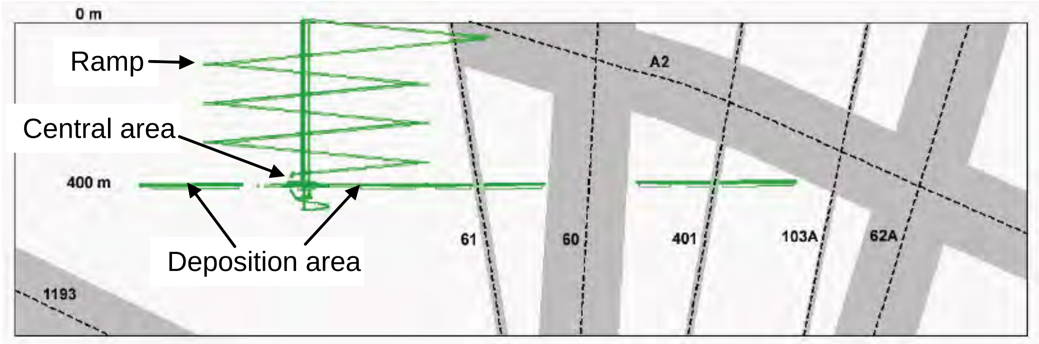


__Figure 5-3.__ Profile for Layout 1 (marked with green) and deterministic deformation zones (marked with grey) including respect distances (zones ≥ 3 km) and "margin for construction" (zones < 3 km). Notations of the zones correspond to the last numbers in the full name of the zone. The profile is according to Figure 5-1.

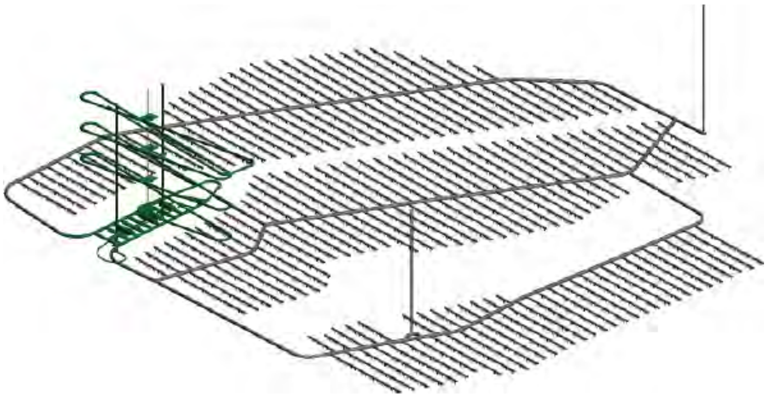


__Figure 5-4.__ 3D illustration of Layout 1.

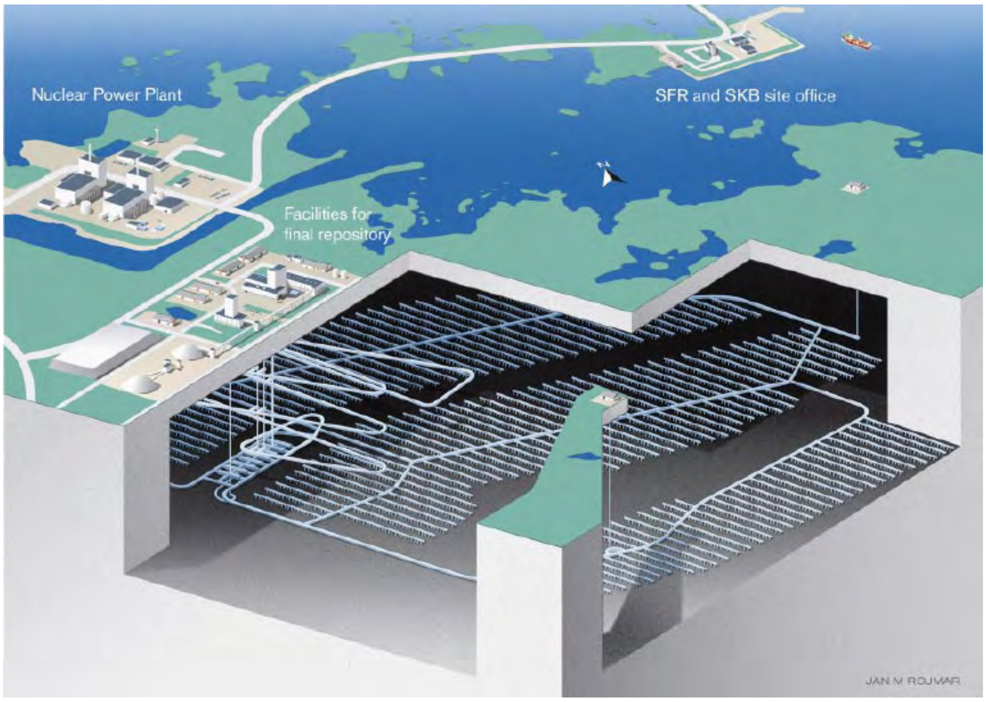


__Figure 5-5.__ Illustration of a possible layout of facilities for the final repository. The layout of the underground part of the facility is according to Layout 1 (from SKB).

Layout 2: 400 m depth. One part of the surface facility is located within the so called residential area, and the other part is located close to the SFR-facility (central area without skip and with a modified ramp to the SFR-facility). A plan of the deposition areas and central area, including the ramp between the ground surface and the repository depth, is illustrated in Figure 5-6.

Layout 3: 500 m depth. In this layout the surface facility is located within the so called residential area (central area with skip and a spiral access ramp). A plan of the deposition areas and central area, including the ramp between the ground surface and the repository depth, is illustrated in Figure 5-7.

Layout 4: 500 m depth. One part of the surface part is located within the so called residential area and the other part is located close to the SFR-facility (central area without skip and with a modified ramp to the SFR-facility). A plan of the deposition areas and central area, including the ramp between the ground surface and the repository depth, is illustrated in Figure 5-8.

The proposed alternative layouts with respect to deposition area, central area, ramp and shafts are further described in the following.

Larger plan of the layouts are also shown in Appendix C.

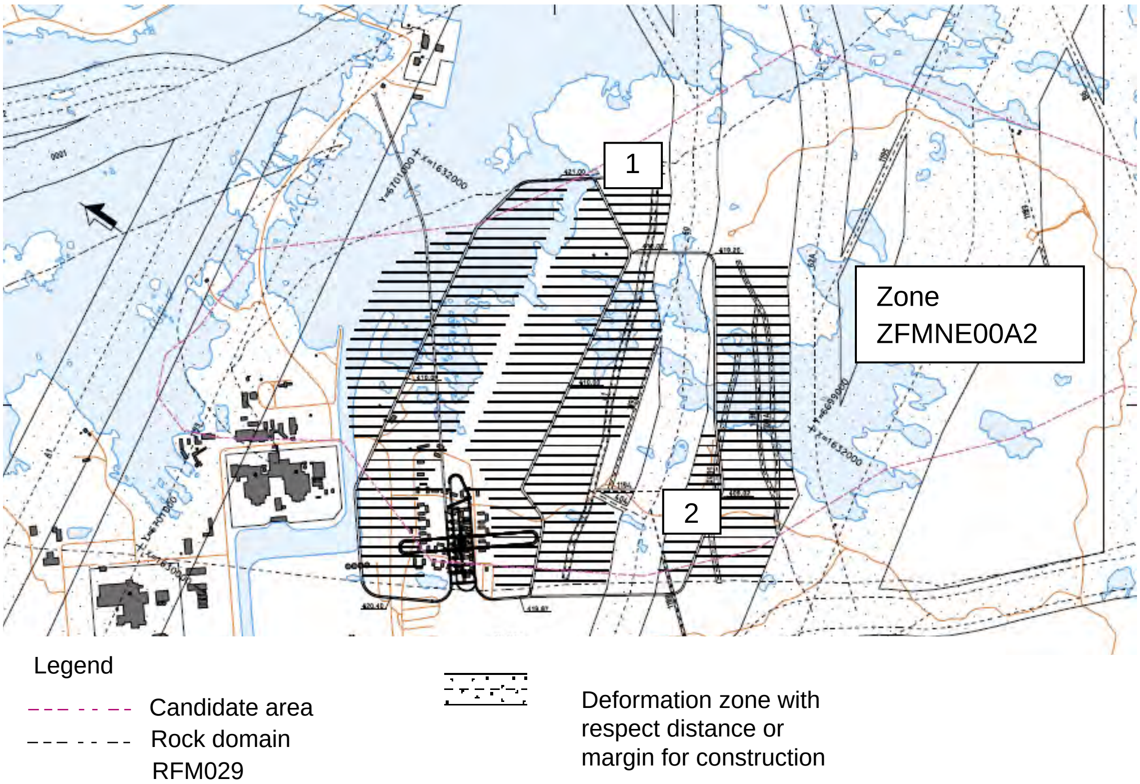
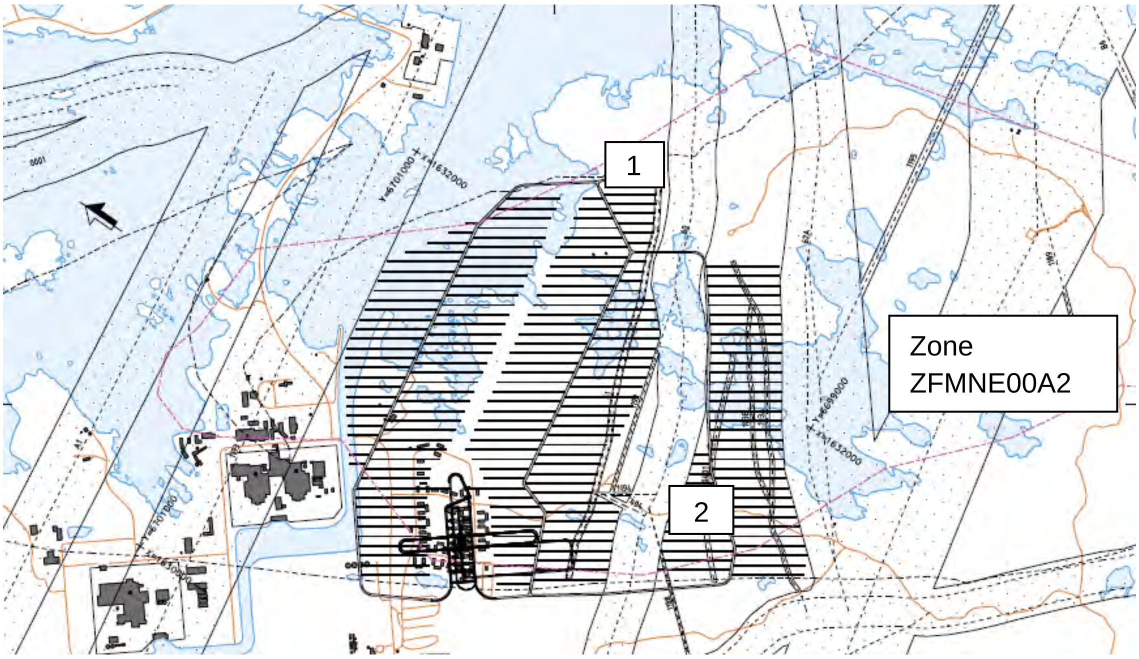


Figure 5-6. Layout 2, depth 400 m. The markings 1 and 2 show possible areas for the shafts for exhaust air.



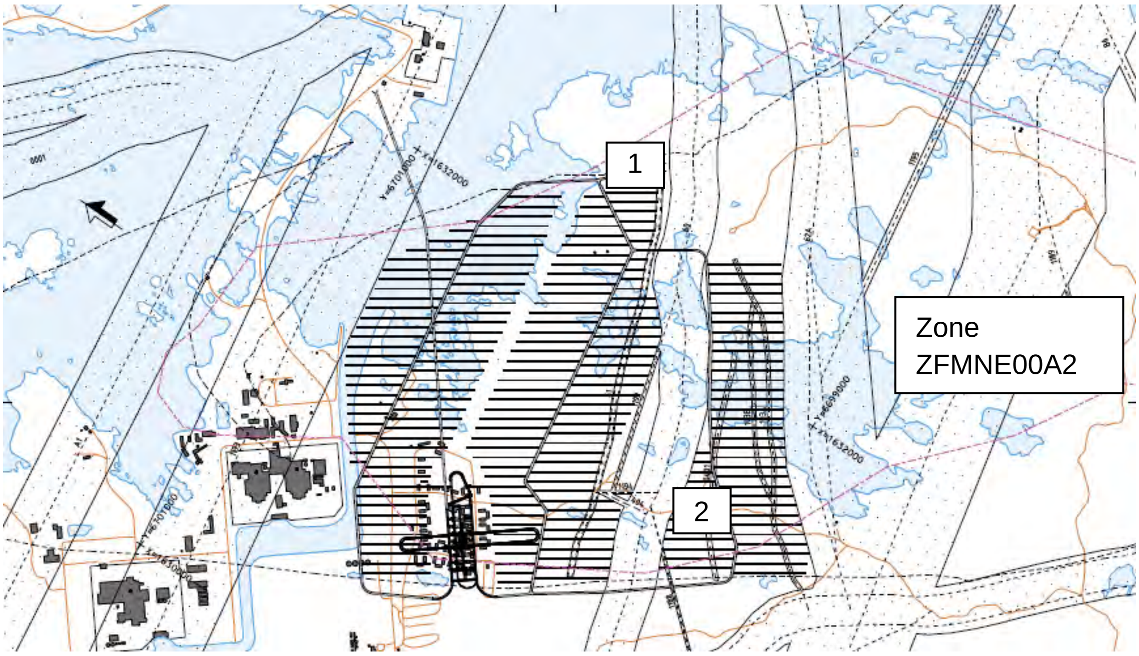
Legend

Candidate area

Rock domain RFM029

Deformation zone with respect distance or margin for construction

Figure 5-7. *Layout 3, depth 500 m. The markings 1 and 2 show possible areas for the shafts for exhaust air.*



Legend

Candidate area

Rock domain RFM029

Deformation zone with respect distance or margin for construction

Figure 5-8. *Layout 4, depth 500 m. The markings 1 and 2 show possible areas for the shafts for exhaust air.*

5.2.2 Key data for proposed deposition areas

Figure 5-9 indicates the principles by which the deposition area is divided and named. Key data for the deposition area are presented in Table 5-2, Table 5-3, Table 5-4 and Table 5-5. It should be noted that tunnels from which deposition tunnels are connected are denoted main tunnels and all other tunnels in the deposition area are denoted transport tunnels. The deposition of canisters for the initial operation is considered to be located to Dep 1 and Dep 2 as per Figure 5-9.

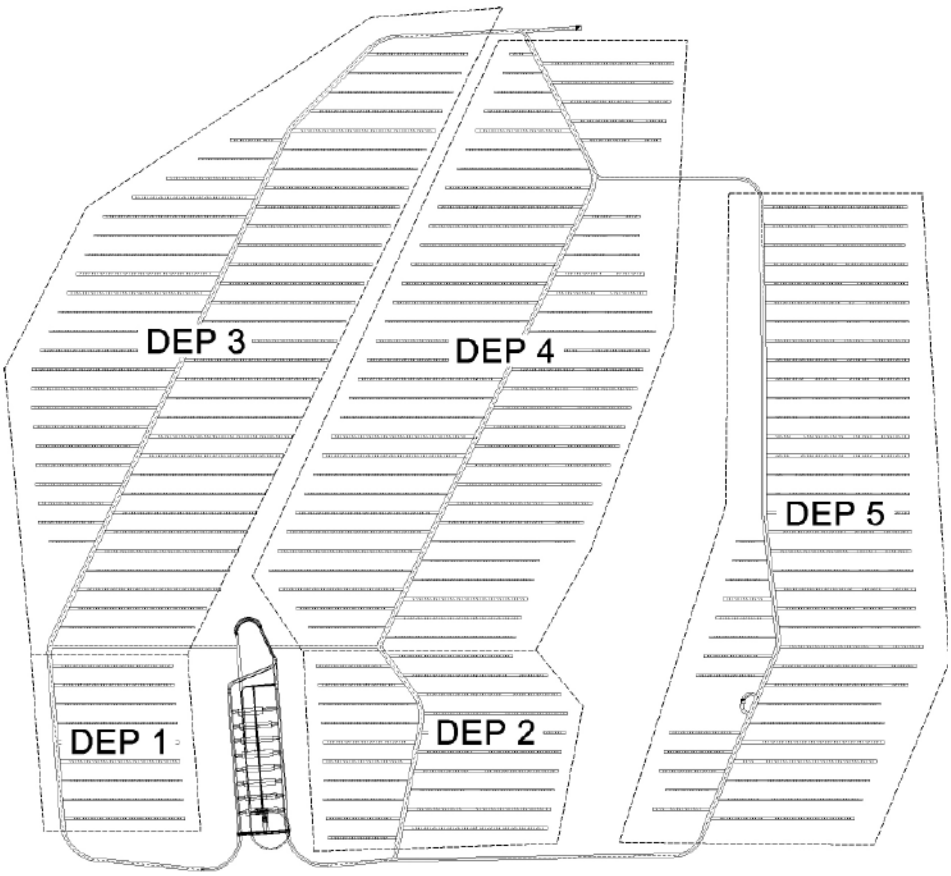


Figure 5-9. *Principal definitions of different parts of the layouts.*

Table 5-2. Summary of key data for the deposition area, Layout 1 and 2.

Key data	Value
Total length of main tunnels (m)	5,030
Total length of transport tunnels (m)	2,676
Number of deposition tunnels	187
Total length of deposition tunnels (m)	47,503
Number of canister positions excluding loss of deposition holes	6,824
Number of canister positions, allowing for 9% loss of deposition holes	6,210 (6,073*)

* Number of canister positions allowing for 11% loss of deposition holes (according to analyses based on “preliminary data”).

Table 5-3. Summary of key data for different parts of the deposition area, Layout 1 and 2.

Key data	Dep 1	Dep 2	Dep 3	Dep 4	Dep 5
Number of deposition tunnels	9	20	53	59	46
Length of deposition tunnels (m)	2,266	4,717	14,231	14,973	11,316
Number of canister positions excluding loss of holes	342	669	2,162	2,153	1,498
Number of canister positions, allowing for 9% loss of holes	311	609	1,968	1,959	1,363

Table 5-4. Summary of key data for the deposition area, Layout 3 and 4.

Key data	Value
Total length of main tunnels (m)	4,923
Total length of transport tunnels (m)	2,869
Number of deposition tunnels	190
Total length of deposition tunnels (m)	48,618
Number of canister positions excluding loss of deposition holes	7,009
Number of canister positions, allowing for 9% loss of deposition holes	6,378 (6,027*)

* Number of canister positions allowing for 14% loss of deposition holes (according to analyses based on "preliminary data").

Table 5-5. Summary of key data for different parts of the deposition area, Layout 3 and 4.

Key data	Dep 1	Dep 2	Dep 3	Dep 4	Dep 5
Number of deposition tunnels	9	20	56	60	45
Total length of deposition tunnels (m)	2,554	4,741	14,422	14,817	12,085
Number of canister positions excluding loss of holes	390	675	2,183	2,163	1,598
Number of canister positions, allowing for 9% loss of holes	355	614	1,987	1,968	1,454

5.2.3 Location of the central area

In all layout alternatives the central area was located below the residential area (see Chapter 3).

5.2.4 Location of shafts in the deposition area

It was assumed that there will be a need for two shafts for exhaust air from the deposition area. An accessibility map showing areas marked for key biotypes and ecological value was used as a basis for the positioning of the ventilation shafts. Nature values and site accessibility maps are more in detail described in /Kyläkorpi 2004/.

In design step D1 the positions of the ventilation shafts were considered as possible positions, which may be revised in subsequent design steps. The number of shafts required and their locations will be studied further in separate investigations.

It should be noted that one of the ventilation shafts was located adjacent to an existing road and that the location of the second shaft requires a new road to be constructed.

5.2.5 Excavated volume

The volume excavated is based on theoretical cross-sections prepared for various parts of the facility according to Layout E /SKB 2002/, with some modifications given by SKB. Input data used for the calculation of excavated volume are summarised in Table 5-6.

In Table 5-7 and Table 5-8 the excavated volume for the possible layouts 1–4 is shown. The excavated volume does not include deposition holes.

The calculated volume is presented as theoretical cubic metres, rounded up to the next 1,000 m^3.

Table 5-6. Data used for calculation of the volume excavated.

Data		
Cross-section of main tunnels, 10×7 m (m^2)	70	
Cross-section of transport tunnels, 7×7 m (m^2)	49	
Cross-section of deposition tunnels, 5.4×4.9 m (m^2)[1]	25	
Cross-section of access ramp, 6×5.5 (m^2)[1]	33	
Diameter of skip shaft (m)	5	
Diameter of elevator shaft (m)	5.5	
Diameter of ventilation shaft (air intake) (m)	3.5	
Diameter of ventilation shaft (exhaust air) in central area (m)	2.5	
Diameter of ventilation shaft (exhaust air) in deposition area (m)	3	
Number of ventilation shafts (exhaust air) in deposition area	2	
	Layout 1 and 2, 400 m	**Layout 3 and 4, 500 m**
Total length of main tunnels (m)	5,030	4,923
Total length of transport tunnels (m)	2,676	2,869
Total length of deposition tunnels (m)	47,503	48,618
Length of access ramp (m)	4,000	5,000
Length of elevator and ventilation shafts (m)	400	500
Length of skip shaft, Layout 1 and Layout 3 (m)	450	550

[1] Modified tunnel cross-sections compared to Layout E /SKB 2002/. The tunnel cross-sections are given by SKB.

Table 5-7. Excavated volume (m^3), 400 m depth. The tunnel type refers to Layout E /SKB 2002/.

Part of Facility	Volume excavated, Layout 1 (m^3)	Volume excavated, Layout 2 (m^3)
Main tunnel (tunnel type A)	353,000	353,000
Transport tunnel (tunnel type B)	132,000	132,000
Deposition tunnel (tunnel type D[1])	1,188,000	1,188,000
Access ramp (tunnel type E)	132,000	132,000
Central area[2] including rock silo	140,000	140,000
Shafts	30,000	21,000
Total volume	1,975,000	1,966,000

[1] Tunnel type D according to Layout E /SKB 2002/ has been modified by SKB.

[2] Volume given by SKB.

Table 5-8. Excavated volume (m³), 500 m depth. The tunnel type refers to Layout E /SKB 2002/.

Part of Facility	Volume excavated, Layout 3 (m³)	Volume excavated, Layout 4 (m³)
Main tunnel (tunnel type A)	345,000	345,000
Transport tunnel (tunnel type B)	141,000	141,000
Deposition tunnel (tunnel type D[1])	1,216,000	1,216,000
Access ramp (tunnel type E)	165,000	165,000
Central area[2] including rock silo	140,000	140,000
Shafts	37,000	27,000
Total volume	2,044,000	2,034,000

[1] Tunnel type D according Layout E /SKB 2002/ has been modified by SKB.

[2] Volume given by SKB.

5.3 Sensitivity analysis

5.3.1 Introduction

The possible layouts were designed based on the premises presented in Section 5.1.2. These premises are more or less associated with uncertainties. In order to study the effect of changed premises a sensitivity analysis was therefore executed.

5.3.2 Input data and assumptions

The sensitivity analysis is based on the designated base layout, Layout 1, which is located at 400 m depth. For this layout the calculation of parameters A_u, V and U was based on the utilized deposition area as indicated in Figure 5-10. These parameters are defined as:

A_u = enclosed deposition area utilised (m²),

V = volume of rock excavated (m³) (theoretical volume),

U = utilisation ratio defined as $6{,}000/N_T$, where N_T is the number of canister positions available after taking the loss of deposition holes into account.

The values of A_u, V and U are presented in Table 5-9. The volume indicated in Table 5-9 includes main tunnels and deposition tunnels only.

The number of available canister positions, N_T, is slightly different (approximately 40 positions) from the number based on "preliminary data" given in Table 5-2. This difference is due to the fact that no deposition tunnels were allowed to be shorter than 100 m in the sensitivity analysis whereas in the layout a few tunnels are shorter than 100 m. The difference in tunnel lengths will also produce a small difference in volume (approximately 3,000 m³) compared to the calculated volume given for main tunnels and deposition tunnels in Table 5-7.

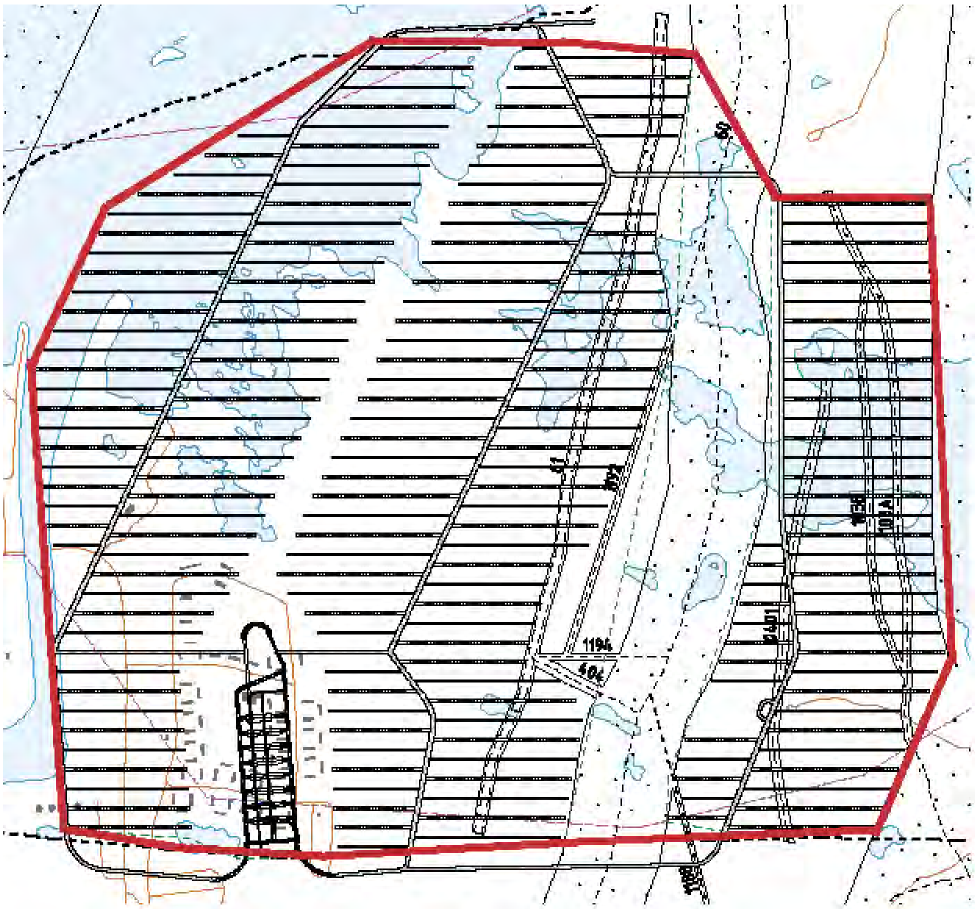


***Figure 5-10.** Borders (delimited by red lines) for calculation of the enclosed utilized deposition area, A_u, for Layout 1 (base layout) in the sensitivity analysis.*

Table 5-9. A_u, V and U for base layout.

A_u	2,730,000 m²
V	1,543,350 m³
U	0.98 (N_T = 6,111)

Prerequisites for the sensitivity analysis were chosen in consultation between SKB and the design team. In the following these prerequisites are described together with an dentification number:

1a. In the sensitivity analysis the effect of deformation zone ZFMNE0060 being shorter than 3 km was analysed, which implies that the zone will not be given respect distance. When the zone is shorter than 3 km, deposition tunnels may pass the zones, but no deposition holes are allowed within a distance from the zone centre equal to half the zone width (including uncertainties) plus a safety distance (SM) of 5 m (see Chapter 3.2).

1b. The number of deposition holes lost due to passages through deformation zones shorter than 3 km in the north-west part of the layout was assumed to be equal to the corresponding number of lost holes in the south-east part of the "priority site".

2. The dip of deformation zones is varied based on the uncertainty values given in /SKB 2005a/, cf Table 5-10 below. The strike of the zones was assumed to be fixed.

Table 5-10. Variation in dip of deformation zones.

Deformation zone	Dip	Horizontal movement at repository depth used in sensitivity analysis [m]
ZFMNE0060	87° ±10°	±70
ZFMNE062A	73° ±10°	±80
ZFMNE00A2	24° ±10°	±300

3. The effect of an increased safety distance (SM) from 5 m (base layout) to 10 m for deformation zones shorter than 3 km was analysed (see 1a above).

4a. The effect of a decreased distance between deposition holes from 6 m (base layout) to 5.5 m was analysed.

4b. The effect of a decreased distance between deposition tunnels from 40 m (base layout) to 37 m was analysed.

5. The base layout assumed a loss of deposition holes equal to 11% (based on "preliminary data"). In order to analyse the effect of uncertainty in the loss of deposition holes the sensitivity analysis was performed for deposition hole losses of 1% and 21%.

6. The maximum length of deposition tunnels was changed from 300 m (base layout) to 600 m.

Referring to prerequisite 2 above, Table 5-10 shows the used variation in dip of the different deformation zones in the sensitivity analysis. A positive movement of a deformation zone implies a dip of the zone to the south-east.

It should be noted that according to /SKB 2005a/ the dip of the deformation zones ZFMNE00A2 and ZFMNE0060 are associated with an uncertainty of –10° and not ±10° as assumed in Table 5-10. The motive for using other values than stated in /SKB 2005a/ was that it was considered valuable to evaluate the sensitiveness due to different variations.

5.3.3 Execution

The premises for the sensitivity analysis were that:

- The premises for the layout given in Section 5.2 are valid.
- One parameter is changed at a time.
- The location of the main tunnels is not changed.

The effect of each change of parameter was analysed in accordance with /SKB 2004a/ with respect to:

- A_u = enclosed deposition area utilised (m²).
- V = volume of rock excavated (m³) (theoretical volume).
- U = utilisation ratio defined as 6,000/N_T, where N_T is the number of canister positions available after taking the loss of holes into account.

Limitations in tunnel distance (length) from deformation zones were taken into account when estimating the length and number of positions available for deposition in deposition tunnels. The procedure for estimating the length of the deposition tunnels due to the influence from the deformation zones is exemplified in Figure 5-11.

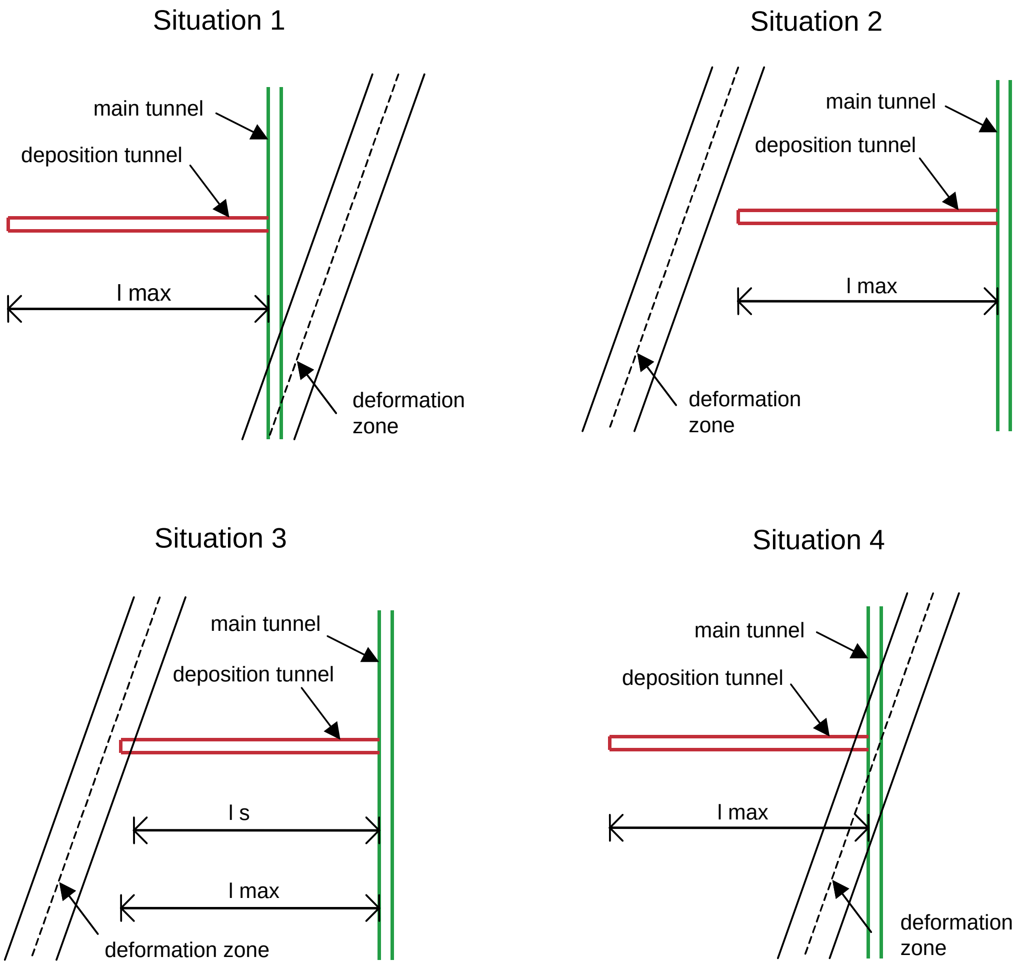


Figure 5-11. *Procedure for estimating the limitations in length and number of positions available for deposition of canisters depending on position of deposition tunnel relative to fracture zones (the parameters lmax and ls are described in the text below).*

Situation 1

Fracture zone (including safety distance) does not influence the deposition tunnel. The length of the deposition tunnel is taken as the maximum deposition tunnel length.

Situation 2

Fracture zone (including safety distance) does not influence the deposition tunnel. The length of deposition tunnel is taken as the maximum deposition tunnel length.

Situation 3

Fracture zone (or safety distance) overlaps the deposition tunnel. If the distance from the deposition tunnel opening to the zone including respect distance or margin for construction exceeds the minimum deposition tunnel length, the length of the deposition tunnel is taken as ls. If the distance from the deposition tunnel opening is less than minimum deposition tunnel length (100 m), the length of deposition tunnel is taken as zero.

Situation 4

Fracture zone (including safety distance) overlaps the deposition tunnel opening. In this situation the length of deposition tunnel is taken as zero.

When the position of a fracture zone changes, deposition tunnels included in the layout illustrated in Figure 5-10 may become longer, shorter or even be eliminated. However, it should be noted that the possibility of having new deposition tunnels at locations where the fracture zone (including safety distance) overlaps the deposition tunnel opening in Figure 5-10 was also included in the analysis.

5.3.4 Results

The results of the sensitivity analysis are summarised in Table 5-11.

It should be noted that due to the definition of the utilisation ratio, U, a value < 1 indicates that capacity for more canisters are available, while a value > 1 indicates that insufficient space is available to accommodate 6,000 canisters in the "priority site" (see Figure 5-10). However, a value of U > 1 does not mean that the repository cannot be accommodated within the site in Forsmark. The reason for this is that the sensitivity analysis only considered the enclosed area utilised for deposition, A_u, and that there is additional area available in rock domain RFM029. In Section 10.2 the possibility of the site to accommodate the repository is analysed more in detail. In this analysis the potential additional area of the site is also discussed further.

Table 5-11. Summary of results of sensitivity analysis. Values in parenthesis indicate the percentage change compared with the base layout.

Analysis	A_u (m²)	V (m³)	U	N_T
1a	2,730,000 (0)	1,703,350 (10.4)	0.88 (–10.6)	6,836 (11.9)
1b	2,730,000 (0)	1,543,350 (0)	1.04 (6.4)	5,745 (–6.0)
2				
ZFMNE0060 +70 m	2,730,000 (0)	1,395,550 (–9.6)	1.10 (12.2)	5,446 (–10.9)
ZFMNE0060 –70 m	2,730,000 (0)	1,540,700 (–0.2)	0.98 (0)	6,112 (0)
ZFMNE0062 +80 m	2,731,188 (0)	1,544,450 (0.1)	0.98 (–0.1)	6,118 (0.1)
ZFMNE0062 –80 m	2,640,633 (–3.3)	1,494,250 (–3.2)	1.02 (4.3)	5,860 (–4.1)
ZFMNE00A2 +300 m	2,730,000 (0)	1,543,350 (0)	0.98 (0)	6,111 (0)
ZFMNE00A2 –300 m	2,443,426 (–10.5)	1,382,150 (–10.4)	1.13 (14.9)	5,317 (–13.0)
3	2,730,000 (0)	1,543,350 (0)	1.03 (5.3)	5,803 (–5.0)
4a	2,730,000 (0)	1,543,350 (0)	0.90 (–7.9)	6,637 (8.6)
4b	2,730,000 (0)	1,638,650 (6.0)	0.91 (–7.4)	6,600 (8.0)
5				
Loss of holes 1%	2,730,000 (0)	1,543,350 (0)	0.88 (–10.1)	6,797 (11.2)
Loss of holes 21%	2,730,000 (0)	1,543,350 (0)	1.11 (12.7)	5,424 (–11.2)
6	See text below			

As per prerequisite 6 (maximum length of deposition tunnels) the following comments should be made. With a maximum length of the deposition tunnels of 600 m instead of 300 m, the conditions for the layout will be different, affecting the number and location of the main tunnels. Consequently, in order to evaluate the effect of longer deposition tunnels it is necessary to design a new base layout. It was however decided that a new base layout should not be designed in design step D1 and that further analyses of the consequence of longer deposition tunnels should not be executed. With main tunnels located as indicated in the base layout some deposition tunnels may be longer but the effect is minor. A new layout based on deposition tunnels with a maximum length of 600 m would probably give shorter main tunnels and less concrete plugs, which would positively influence the cost and environmental impact.

A summary of the results of a supplementary analysis, where the utilisation ration, U, is set to 1, is provided in Table 5-12. This indicates the area and volume, given that 6,000 canisters are deposited (U=1).

A reduction of tunnels in the event of over-capacity in the area was achieved by eliminating tunnels at the greatest distance from the access ramp.

Table 5-12. Summary of results of sensitivity analysis, given that U=1.

Analysis	A_u (m^2)	V (m^3)	U	N_T
1a	2,635,649	1,526,400	1.00	6,000
1b			–	–
2				
ZFMNE0060 +70 m[1]			–	–
ZFMNE0060 –70 m[1]	2,658,593	1,519,775	1.00	6,000
ZFMNE0062 +80 m	2,694,468	1,521,500	1.00	6,000
ZFMNE0062 –80 m			–	–
ZFMNE00A2 +300 m[1]	2,696,760	1,522,575	1.00	6,000
ZFMNE00A2 –300 m			–	–
3			–	–
4a	2,532,480	1,419,900	1.00	6,000
4b[2]	2,730 ,000 (unchanged)	1 ,521,691	1.00	6,000
5				
Loss of holes 1%	2,485,880	1,390,775	1.00	6,000
Loss of holes 21%			–	–
6	–	–	–	–

[1] Tunnels shorter than100 m will be accepted for deposition of canisters.

[2] The volume is based on the assumption that the length of main tunnels is not changed compared with the base layout.

5.4 Conclusions

From the layout studies it can be concluded that the differences in location, layout and excavated volume are small between the four proposed alternatives (Layouts 1–4).

Conclusions from the sensitivity analysis are summarised below:

- For the proposed layout (base layout), the dip of the deformation zone ZFMNE00A2 should be investigated further since only a minor increase in dip will have a significant effect on the number of canister positions available. In addition the area available between zones ZFMNE0062A and ZFMNE00A2 will be eliminated. It should be noted that it would be possible to modify the layout with respect to steeper dip of ZFMNE00A2 by using the area between the zones ZFMNE0060 and ZFMNE0062A (the location and orientation of the deformation zones are illustrated in Section 3.2).
- The effect on the number of positions available, N_T, will be approximately the same if the distance between the deposition holes is shorter or if the distance between the deposition tunnels is shorter.
- The possibility of decreasing the distance between deposition holes or deposition tunnels depends on the thermal properties of the rock mass and the mechanical effects due to the high rock stresses. This issue should be evaluated further in the future. With regard to the thermal effects, analyses presented in Section 4.3 indicate that a shorter distance than 6 m between the deposition holes should be possible.

Some aspects requiring consideration when evaluating the conclusions above are that:

- The effect of variations in the different parameters according to the sensitivity analysis will be different if the number of main tunnels and their location can be adjusted with respect to new conditions. In the light of this possibility, the result of the sensitivity analysis is probably somewhat conservative.
- Single variations of the different parameters will affect the number of canister positions available only to a minor extent. It must however be noted that only one parameter is changed at a time. If several parameters are changed at the same time the result will be different. The effect of changing the parameters at the same time is evaluated in the technical risk assessment (see Chapter 10).

Unknown or not yet understood factors which influence the layout, for example not identified deformation zones, may influence the result of the sensitivity analysis and thus the possibility of accommodating a repository within the studied site.

6 Identification of passages through deformation zones

6.1 Introduction

Based on the proposed layouts in Chapter 5, this chapter comprises an identification and classification of passages through deformation zones.

The purpose of identifying and classifying passages through deformation zones was to establish a basis for determining construction requirements with respect to excavation, grouting and rock support in proposed tunnels. Based on the number and the total length of passages with associated rock quality a comparison between different layouts has been made.

According to /SKB 2004a/ the deformation zones passed by different parts of the facility should be classified on the basis of rock quality. In the present report the rock quality is characterised in terms of the Q-value according to /Barton 2002/. The rock mechanical and hydraulic properties of the zones are also described as a basis for the assumption of rock support and grouting requirements.

6.2 Input data and assumptions

In /SKB 2005a/ (Table 6-4 in /SKB 2005a/) Q-values are given for deformation zones. These Q-values are based on logging of rock cores from boreholes KFM01A, KFM02A, KFM03A and KFM04A, and apply to low rock stresses and dry conditions. These Q-values consequently represent an SRF value of 1.0 and a joint water reduction factor J_w of 1.0. The Q-values range most frequently between approximately 10 and 50. The variation is however considered to be large. The Q-values indicated in /SKB 2005a/ are in the same order as those estimated based on “preliminary data”.

An example of a rock mass including two fracture zones is given in Figure 6-1. The most frequent Q-values for the fracture zones according to /Barton 2003/ are 17–38, corresponding to “good rock” in the Q system.

In Table 6-1 the rock mechanical properties for the deformation zones with reference to /SKB 2005a/ are given. Rock mechanical properties of the rock mass between the zones are also given in Table 6-1 for comparison. The rock mechanical properties given in /SKB 2005a/ are the same as those given as “preliminary data”.

With regard to hydraulic properties, there were some differences between the data presented in /SKB 2005a/ and estimations based on “preliminary data”. Transmissivity values in “preliminary data” were based on the site description version 1.1 /SKB 2004c/, flow loggings in bore holes KFM01A and KFM04A (see /Rouhiainen and Pöllänen 2003/ and /Rouhiainen and Pöllenen 2004a–d/), preliminary results of hydraulic measurements in boreholes and KFM05A and KFM06A. An assessment of the transmissivity for different deformation zones was then executed and finally, assuming a hydraulic width of 10 m, hydraulic conductivity values were calculated. In Table 6-2 the estimation of hydraulic conductivity values, K, based on “preliminary data” is presented.

Figure 6-1. Example of fracture zones identified in borehole KFM01A. The upper zone is classified as Q=17 (most frequent value with SRF=1) and the corresponding value for the lower zone is Q=38 (SRF=0.5) (From /Barton 2003/).

Table 6-1. Rock mechanics properties of the rock mass according to /SKB 2005a/.

Rock mechanic property	Rock mass between deformation zones, Rock Domain 29, RFM029. Mean value/Min–Max	Deformation zones (all stochastic and deterministic zones shorter than 10 km). Mean value/Max–Min
Uniaxial compressive strength (MPa)	122/60–195	112/55–160
Deformation modulus (GPa)	67/40–80	58/25–75
Tensile strength (MPa)	2.0/0.3–5.0	1.2/0.2–2.9

Table 6-2. Assessment of hydraulic conductivity values, K, for deformation zones passed (based on "preliminary data")

Zone	Hydraulic conductivity K m/s
Sub-horizontal near-surface zones (depth 0–200 m)	1×10^{-4}–1×10^{-6}
ZFMNE0061	1×10^{-7}–1×10^{-9}
ZFMNE0060	1×10^{-6}–1×10^{-8}
ZFMNE0401	1×10^{-8}–1×10^{-9}
ZFMNE103A and B	1×10^{-8}–1×10^{-9}
ZFMNE1188	1×10^{-8}–1×10^{-10}

The hydraulic conductivity values, K, in Table 6-2 can be compared with the data in /SKB 2005a/, where hydraulic properties of the deformation zones are given. The transmissivity (trend lines) for deformation zones are given in Figure 8-33 in /SKB 2005a/, and for steeply dipping zones the tranmissivity is about 1×10^{-7} to 1×10^{-8} m²/s at depths of 400–500 m. Assuming a hydraulic width of 10 m these transmissivities correspond to hydraulic conductivities of 1×10^{-8} to 1×10^{-9} m/s.

In /SKB 2005a/ (Table 8-6 in /SKB 2005a/) transmissivities are also provided for some specific deformation zones. For the zones passed by tunnels in the layouts at repository level, values in Tabell 6-3 are given. It should be noted that no transmissivity value for the deformation zone ZFMNE0060 is given in Table 8-6, /SKB 2005a/.

In summary, the hydraulic conductivities based on "preliminary data" are higher than corresponding data from /SKB 2005a/ for all deformation zones except the sub-horizontal near-surface zones. This is estimated to be the case for the zones ZFMNE0060 and ZFMNE0061 in particular.

Tabell 6-3. Transmissivity values and widths for deformation zones according to /SKB 2005a/ and calculated hydraulic conductivities.

Zone	Transmissivity, T (m²/s)	Width (m)	Calculated hydraulic conductivity (m/s)
Sub-horizontal near-surface zones (depth 0–200 m)	1×10^{-3} to 1×10^{-4}	1–10	1×10^{-3} to 1×10^{-5}
ZFMNE0061	$< 1\times10^{-9}$	45	$< 0.2\times10^{-10}$
ZFMNE0401	$< 1\times10^{-9}$	15	$< 0.6\times10^{-10}$
ZFMNE103A and B	1×10^{-8}	158	0.6×10^{-10}
ZFMNE1188	1.4×10^{-8} and 1×10^{-9}	38 and 5 respectively	3.7×10^{-10} and 2×10^{-10}

6.3 Execution

The design task was executed in accordance with /SKB 2004a/, which specifies the scope as follows:

1. Identification of passages through deterministically determined deformation zones to provide access routes between different parts of the hard rock facility.
2. Classification of each passage with respect to rock quality.
3. Estimation of the length L (m) of each passage.
4. Assessment of the difficulties and measures anticipated for each passage with respect to excavation, rock support and grouting based on empirical knowledge.

6.4 Results

6.4.1 Identification of passages through deformation zones

The passages of deterministically determined deformation zones are illustrated in Figure 6-2 and Figure 6-3 for the layouts described in Chapter 5. In Table 6-4 and Table 6-5 the passages through the zones are described with respect to the number of passages and length of passages. The length of the passage (L) is assumed to be equal to the zone width (including uncertainties), which is given in /SKB 2005a/ for each specific zone.

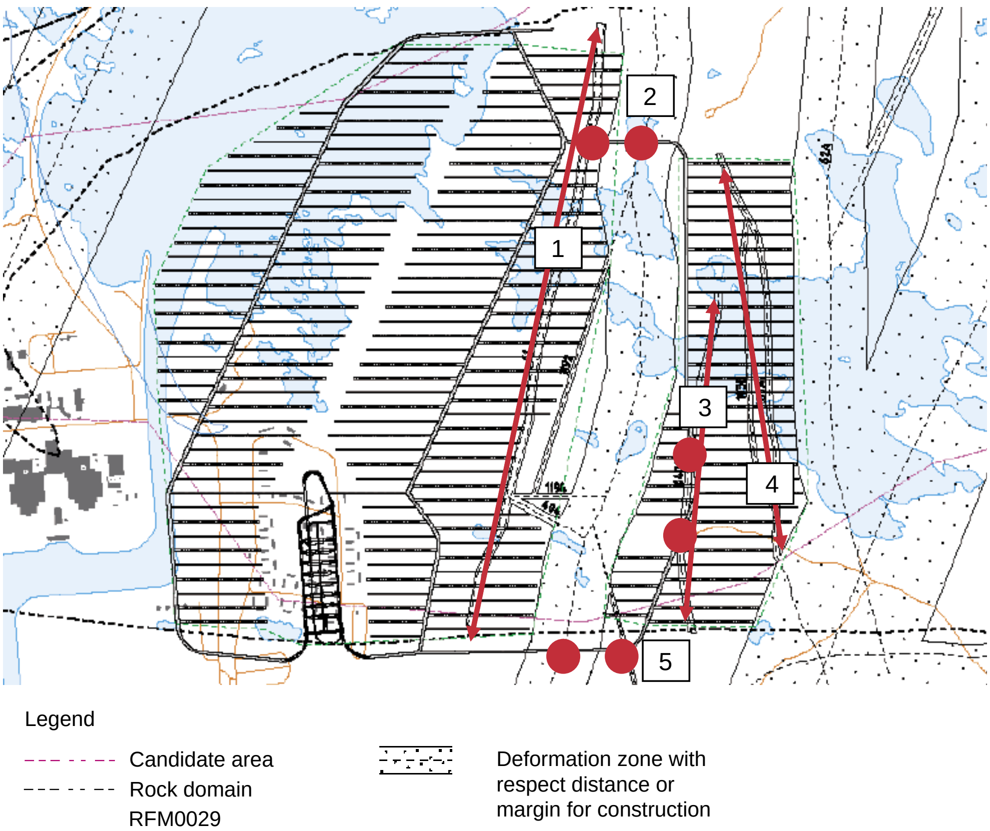


Figure 6-2. Passage of deterministically determined deformation zones, depth 400 m (Layout 1 and 2). Red dots indicate passages with transport tunnels or main tunnels and red arrows indicate passages with deposition tunnels. Numbers refer to the deterministically determined deformation zones given in Table 6-4.

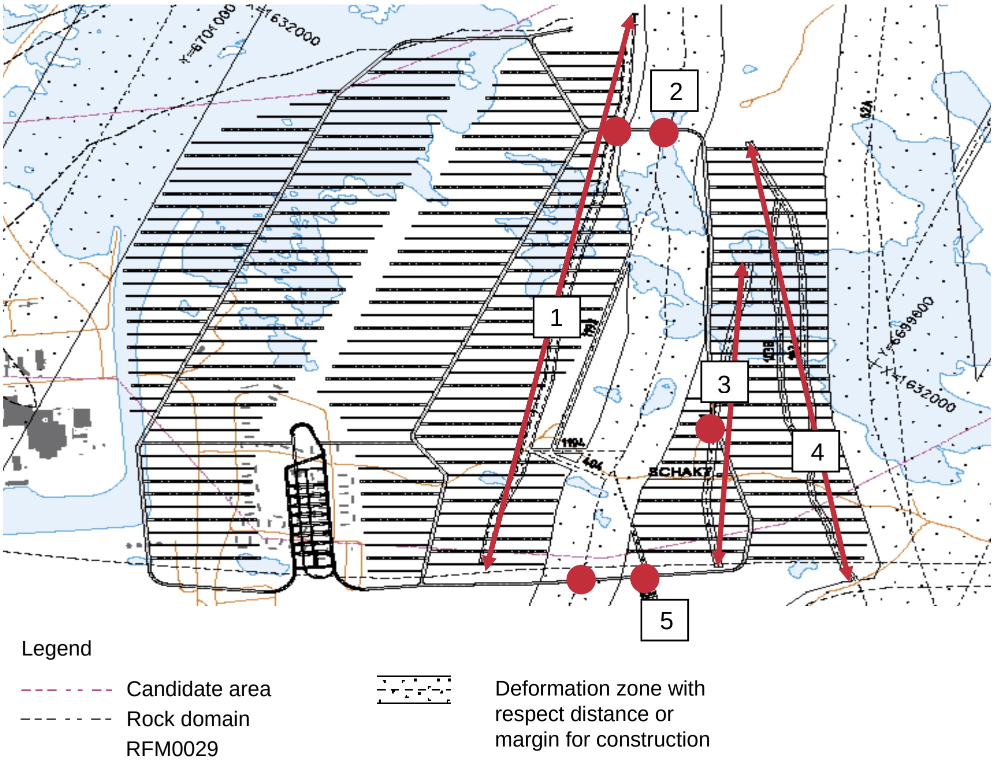


Figure 6-3. *Passage of deterministically determined deformation zones, depth 500 m (Layout 3 and 4). Red dots indicate passages with transport tunnels or main tunnels and red arrows indicate passages with deposition tunnels. Numbers refer to the deterministically determined deformation zones given in Table 6-5.*

Besides from passages through the deterministically determined deformation zones listed in Table 6-4 and Table 6-5, the ramp and shafts in all layouts will pass through sub-horizontal water-bearing zones which are present down to about 200 m depth /SKB 2005a/.

Table 6-4. Identification of passages through deterministic deformation zones, Layout 1 and 2 (400 m depth).

Notation	Zone	Part of Facility	Number of passages N	Length of passage L (m)	Total length of passages ΣL (m)
1	ZFMNE0061	Deposition tunnel	31	15	465
1	ZFMNE0061	Transport tunnel	1	15	15
2	ZFMNE0060	Transport tunnel	2	5–15	10–30
3	ZFMNE0401	Deposition tunnel	23	6–10	138–230
3	ZFMNE0401	Transport tunnel	2	6–10	12–20
4	ZFMNE103A and B	Deposition tunnel	45	6–7	270–315
5	ZFMNE1188	Transport tunnel	1	1–2	1–2
Total number of all passages N_{Tot}			105		
Total length of all passages ΣL_{Tot} (m)					911–1,077

Table 6-5. Identification of passages through deterministic deformation zones, Layout 3 and 4 (500 m depth).

Notation	Zone	Part of Facility	Number of passages N	Length of passage L (m)	Total length of passages ΣL (m)
1	ZFMNE0061	Deposition tunnel	23	15	345
1	ZFMNE0061	Transport tunnel	1	15	15
2	ZFMNE0060	Transport tunnel	2	5–15	10–30
3	ZFMNE0401	Deposition tunnel	22	6–10	132–220
3	ZFMNE0401	Transport tunnel	1	6–10	6–10
4	ZFMNE103A and B	Deposition tunnel	54	6–7	324–378
5	ZFMNE1188	Transport tunnel	1	1–2	1–2
Total number of all passages N_{Tot}			104		
Total length of all passages ΣL_{Tot} (m)					833–1,000

It can be concluded from Table 6-4 and Table 6-5 that the total number and length of all passages (N_{Tot} and ΣL_{Tot}) are of the same order for all layouts. A somewhat shorter total length is obtained for Layouts 3 and 4. When performing a more detailed design in forthcoming design steps the layouts should, if possible, be adjusted with regard to the location of deformation zones. When making such adjustments the number of deposition tunnels, length of main tunnels and the transport requirements must be compared with the increased rock support requirements resulting from passages through deformation zones. A layout with few deposition tunnels and many passages through zones may be preferred to one with many deposition tunnels and few passages.

6.4.2 Classification and description of deformation zones

In this section the resulting classification and description of the deformation zones is presented.

The Q-values presented in /SKB 2005a/, was derived with a stress reduction factor (SRF) of 1.0. However, according to /Barton 2002/ the value of SRF should be set at 2.5 for “weakness zones intersecting excavation”. When using a value of 2.5 the most frequent Q-values will be 4–10, corresponding to “fair rock” according to the Q system (assuming that the joint water reduction number J_w is 1.0). For the classification of the deformation zones passed it is therefore considered reasonable, perhaps somewhat conservative, to ascribe a Q-value of 4–10 (“fair rock”) to the zones. The deformation zone ZFMNE0060 is however assumed to be of somewhat poorer rock quality. In the present stage the only reason for such an assumption is that this is the only zone longer than 3 km, and consequently associated with a respect distance. Since the rock mechanics parameters for the deformation zones do not differ significantly from the surrounding rock, it is reasonable to assume that the rock quality in zone ZFMNE0060 is at worst one Q class lower than the other zones. This means that the Q-value for deformation zone ZFMNE0060 is set to 1–4, corresponding to “poor rock” according to the Q system. In Table 6-6 the classification of the deformation zones by means of the Q-value is presented.

It can be noted that no classification of the sub-horizontal near surface zones has been made as any relevant information regarding the rock quality in these zones are not available. Further investigations must thus be made.

Table 6-6. Classification of deformation zones in terms of rock quality, Q-value.

Zone	Rock quality, Q-value
ZFMNE0060	1–4 (poor)
ZFMNE0061	4–10 (fair)
ZFMNE0401	4–10 (fair)
ZFMNE103A and B	4–10 (fair)
ZFMNE1188	4–10 (fair)

From Table 6-1 it can be concluded that the difference between the rock mechanical properties for the deformation zones and for the rock mass between the zones is small.

In the deformation zones the probability of spalling may be different than in the rock between the deformation zones, since other in-situ stresses and uniaxial compressive strength values for the intact rock may be encountered. Since higher stresses than in the surrounding rock are not likely in the zones due to a more fractured rock, and the strength is estimated to be about the same or less in the zones and in the surrounding rock, the probability of spalling is probably less. If spalling occurs it is expected to be encountered in the transition between the deformation zone and the surrounding rock. In the main tunnels some spalling could be expected, for example at the intersections with deposition tunnels, but the probability for additional spalling problems is assumed to be low. Other stress induced stability problems such as large deformations may occur but, due to the relatively good rock quality encountered in the deformation zones, such stability problems are not judged to be likely.

Regarding the hydraulic properties, Table 6-7 presents the values of hydraulic conductivity, K, used for the zones passed in the layouts.

If input data from /SKB 2005a/ had been used, the major difference in the resulting classification and description of the deformation zones with regard to the hydraulic conductivity would be that lower hydraulic conductivity values had been achieved.

Table 6-7. Hydraulic conductivity values, K, for deformation zones passed (based on "preliminary data").

Zone	Hydraulic conductivity K m/s
Sub-horizontal near-surface zones (depth 0–200 m)	1×10^{-4}–1×10^{-6}
ZFMNE0061	1×10^{-7}–1×10^{-9}
ZFMNE0060	1×10^{-6}–1×10^{-8}
ZFMNE0401	1×10^{-8}–1×10^{-9}
ZFMNE103A and B	1×10^{-8}–1×10^{-9}
ZFMNE1188	1×10^{-8}–1×10^{-10}

6.4.3 Difficulties and measures in passing deformation zones

One advantage with respect to the tunnel excavation is that all passages through deterministically determined deformation zones are perpendicular to the zones. Another advantage is that deposition tunnels and transport tunnels, which are passing through the deformation zones, are oriented parallel to the major principal stress. The sub-horizontal near-surface zones may give rise to problems with stability and water inflow in the access tunnel due to unfavourable orientation of the tunnel relative to the zones. Adequate probe drillings must be carried out during excavation of the ramp in order to investigate the related hydraulic and rock mechanical conditions.

It is assumed that excavation will be performed by conventional drilling and blasting. Smooth blasting will be required to minimise the damaged zone around the tunnel periphery. According to /SKB 2004a/ it should also be assumed that deposition tunnels will be excavated with top heading and bench. The rock caverns will probably also be excavated in this way.

Rock support is assumed to be carried out by conventional methods with rock bolts, shotcrete and steel mesh. Based on the rock mass description and classification in Section 6.4.2 it is assumed that potential instability problems in the deformation zones may involve outfall of wedges or loose rock. There may also be some spalling in the main tunnel, which passes the zone ZFMNE0401, but the probability of spalling is assumed to be low. The stability of the horizontal near-surface zones should be investigated further. A more detailed description of rock support requirements and proposed support systems is described in Chapter 9.

With regard to the grouting requirement it can be assumed that grouted rock will correspond to a hydraulic conductivity in the range of 10^{-7} to 10^{-9} m/s. This assumption is based on the requirement from /SKB 2004a/ and that only cement based grouts are allowed to be used. With this assumption it can be concluded that grouting will be executed mainly in the sub-horizontal zones near the surface during excavation of the access tunnel and shafts. From the analyses based on "preliminary data" grouting will also be required to a certain extent in zone ZFMNE0060 during excavation of the transport tunnels and in the zone ZFMNE0061 during excavating the deposition tunnels. In the other zones, no or only minor grouting need should be expected.

It should be noted that grouting requirements will finally be determined by the maximum allowable water inflow into the deposition tunnels, but no such restrictions have been specified at present. A more detailed description of grouting requirements is provided in Chapter 8.

6.5 Conclusions

No major problems are expected during tunnelling through the deterministically determined deformation zones. However, extensive probe drilling, grouting and special excavation requirements and rock support are expected when passing through the sub-horizontal near-surface zones which can be highly water-bearing.

The need of rock support and grouting is analysed and described in greater detail in Chapters 8 and 9. In view of the expected rock support and grouting requirements it may be worthwhile in later design steps to reduce the number of passages through deformation zones. In attempting to minimise the number of passages, the cost and time required to install rock support and grouting works must be compared with the resulting number of deposition tunnels and length of main tunnels. The optimum layout may not be the one with the fewest passages through deformation zones.

7 Seepage and hydrogeological situation around repository

7.1 Introduction

Seepage into the repository and the hydrogeological situation around the repository with respect to salinity (TDS) as well as lowering of the groundwater table was evaluated in order to define the effect from grouting during construction, and water management during the operational stage. The evaluation has been made both with analytical and numerical tools. The numerical calculations were delivered by SKB to the design team by means of a report by Urban Svensson, Computer-aided Fluid Engineering AB (see /Svensson 2005/).

It should be noted that analytical calculations were executed based on “preliminary data”.

7.2 Input data and assumptions

In the following sections the input data for the analyses are presented. Two depths are considered, 400 m and 500 m depth.

7.2.1 Geology

Dominant structures in the “priority site” are generally vertical or sub-vertical, striking in the NNW-SSE and NE-SW directions. Horizontal or sub-horizontal structures are common in the upper part of the bedrock where they may be filled with sediments (sandy, silty) with very high hydraulic conductivity. According to /SKB 2005a/, five joint sets have been identified in rock domain RFM029:

Designation	Strike
NS	2°
NE	47°
NW	131°
EW	100°
HZ	73°

The NE, HZ and NW sets are the most prominent.

7.2.2 Hydrogeology

Information from the northernmost located boreholes of the candidate area, which form the background for the Preliminary site description version 1.2 /SKB 2005a/, indicate nearly impervious bedrock below approximately 360 m depth, whereas the uppermost 200 m may be highly permeable. This conclusion also applies for the central and southern parts of the candidate area, but here there is a difference at each side of deformation zone A2 (ZFMNE002A). This zone separates a relatively impervious rock below the deformation zone from a more permeable rock above. All boreholes penetrating A2 (KFM01A, KFM02A, KFM04A and KFM05A) indicate very low hydraulic conductivity beneath A2 with virtually no groundwater flow in the rock mass outside deterministically identified deformation zones. A conceptual model of the Forsmark site, which was presented as "preliminary data", is presented in Figure 7-1. For a comparison, in /SKB 2005a/ Volume B is defined for the depth 220–360 m, and Volume C for the depth 100–220 m.

Information on the water-bearing properties are available from boreholes KFM01A–KFM05A /Rouhiainen & Pöllänen 2003, 2004a–d/. The results of hydraulic tests in the boreholes suggest that there are no significant water flows in Volume D. /SKB 2005a/ presents statistical data on the hydraulic conductivity in the target volume as indicated in Table 7-1. The data is within the range presented in /SKB 2005a/ for Volume B, i.e. within the variation defined as upper and lower bounds of the data set.

The data given in /SKB 2005a/ for the target volume below 400 m depth below the deformation zone ZFMNEA2 in rock domain RFM029 suggest that there are no significant water flows in Volume D. No hydraulic data existed above the lower measurement limit in boreholes drilled into the target volume. Consequently, it has not been possible to execute a more elaborated analysis of the water seepage into the deposition areas of the repository. An assessment regarding the hydraulic conductivity in different directions is given in Table 7-1, which has been used as the distribution function for the hydraulic conductivity for modelled 20 m blocks with fractures longer than a certain length (L_{min}). The data given in the table is more representative for the rock mass above 360 m to 400 m depths than for the actual target volume, which is expected to have an even lower hydraulic conductivity. The conductivity data from /SKB 2005a/ representing Volume B is included for comparison. The data set used for the design analyses is close to the values given by the upper bound with a used median conductivity of 5×10^{-11} m/s compared to 2.2×10^{-10} m/s as defined by the upper bound. As a comparison, the lower bound results in a median conductivity of 2.8×10^{-13} m/s.

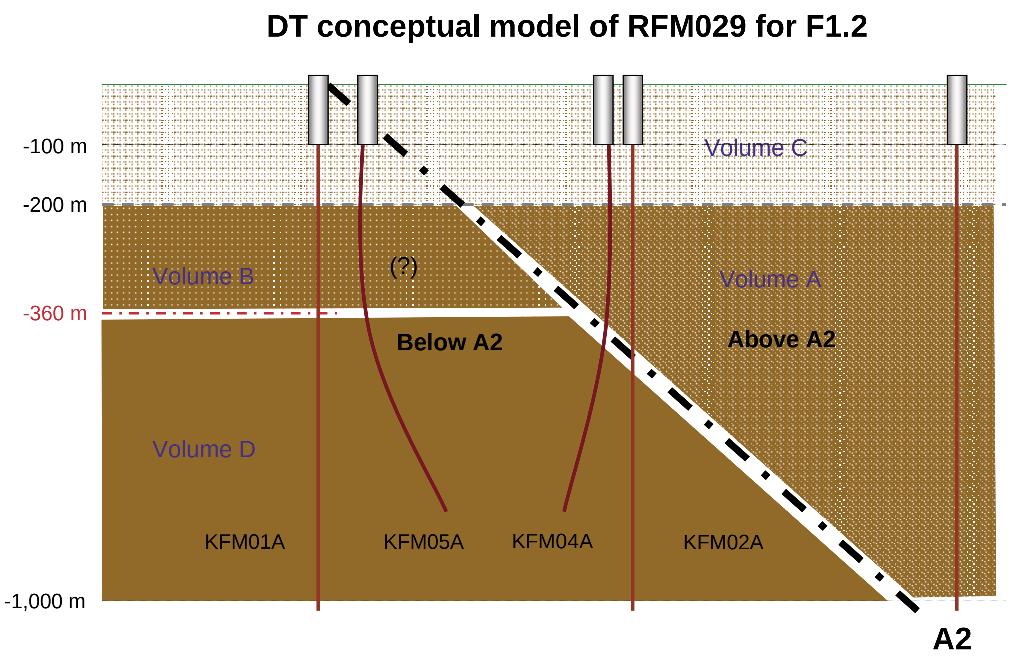


Figure 7-1. *Schematic cross-sections showing a conceptual model of RFM029. "Darcy Tools" was used for numerical simulations of regional groundwater flow and mass (salt) transport in version 1.2 (given as "preliminary data" by SKB).*

Table 7-1. Percentiles for the effective hydraulic conductivity of the dominant fracture sets for 20 m blocks evaluated by DarcyTools, which was given as "preliminary data", together with upper and lower bounds according to /SKB 2005a/. The anisotropy is non-existent according to /SKB 2005a/.

	Profile	Scale	L_{min}	$\log_{10}(K_b)$ (m/s) 10 percentile	25 percentile	50 percentile	75 percentile	90 percentile
"Preliminary data"	NE	20 m	10 m	–12.3	–11.7	–10.3	–9.6	–9.4
	NW	20 m	10 m	–12.3	–11.6	–10.8	–9.6	–9.3
/SKB 2005a/	Upper bound	20 m	5.64 m	–10.17	–9.89	–9.65	–9.14	–9.03
	Lower bound	20 m	5.64 m	–15.58	–14.15	–12.55	–10.94	–9.52

The 90-percentiles are 4×10^{-10} m/s, 9.3×10^{-10} m/s and 3×10^{-10} m/s for the design, upper bound and lower bound distributions respectively.

A back-calculation based on data from flow loggings (PFL-data) presented in /SKB 2005a/ suggests a 20 m block conductivity of 3.63×10^{-11} m/s, a value which is very close to the values used in the analyses. Taking uncertainties into account it appears that the design calculations are based on reasonable assumptions and within the range of the data given in /SKB 2005a/.

For the analytical and numerical calculations it was assumed that $K_X = K_Y = K_Z$, which is in accordance with the data presented in /SKB 2005a/.

The numerical calculations with DarcyTools v 3.0 used input data from /SKB 2005a/.

7.2.3 Sealing levels

It was decided together with SKB that the following two sealing levels (grouting levels according to /SKB 2004a/) should be used as the basis for the analytical calculations:

- Sealing level 1: $K_t = 1\times10^{-7}$ m/s.
- Sealing level 2: $K_t = 1\times10^{-9}$ m/s.

The symbol K_t denotes the hydraulic conductivity of the grouted zone around the underground openings. The numerical calculations with Darcy Tools were made also for ungrouted condition ($K_t = 1\times10^{-7}$ m/s), and for a level with very low hydraulic conductivity ($K_t = 1\times10^{-11}$ m/s).

It should be noted that the sealing levels are given as preliminary values in the design premises /SKB 2004a/ (level 1 and level 2 according to /SKB 2004a/), as no specific requirements on seepage are formulated for design step D1. These values were considered suitable since these sealing levels can be anticipated to be achieved when using cement based grouts. From Table 7-1 it can however be concluded that the probability of having a hydraulic conductivity in the rock mass higher than 1×10^{-9} m/s is very small. Since the analytical calculations did not include the deformation zones the sealing level 1×10^{-7} m/s was not regarded for the general case. However, an analysis of the impact of the shallow water bearing fracture zones, which will intersect the ramp and shafts, was made.

On the other hand, in the numerical analyses, which included the deformation zones, four sealing levels were used. One supplementary sealing level, 1×10^{-11} m/s, was also introduced for these calculations, even if it is not likely that this sealing level could be achieved by grouting with cement based grouts. The sealing levels were taking into account by applying the specified hydraulic conductivity values for all cells in contact with the repository. The cells in contact with the tunnels have all a cell size of 4 m.

A maximum conductivity of 1×10^{-7} m/s implies that all cells with conductivities exceeding this value are reduced to 1×10^{-7} m/s.

7.2.4 Points in time (excavation stages)

The excavation of the deep repository has in the analytical calculations been divided into one early and one late stage of excavation (points in time according to /SKB 2004a/). The late stage corresponds to completion of construction of the whole repository. The different points in time have been simulated by assuming a radius of the repository equal to 50 m and 500 m respectively. In the numerical calculations only a complete open repository is analysed.

7.3 Execution

Analyses of seepage and the hydrogeological situation around the repository were performed as numerical calculations using Darcy Tools together with analytical calculations using Monte Carlo simulations. The analyses included an evaluation of seepage into the repository as well as a prediction of upconing of saline water and draw down of the groundwater table for various sealing levels, i.e. without grouting and with grouting. The procedure outlined in the /SKB 2004a/ was followed for the evaluations, but slightly modified as agreed with SKB. The agreed modifications include:

- Only DarcyTools was used for the numerical calculations.
- No analytical calculations were performed of the draw down of the piezometric water table due to the specific hydraulic conditions at the site, involving a strongly water-bearing upper part of the rock mass in direct contact with the sea, which produces a ixed upper boundary.

7.3.1 Analytical calculations

The analytical calculations executed with an assumed lognormal distribution (Figure 7-2) with a geometric mean of 5×10^{-11} m/s with a 90% percentile at 3.98×10^{-10} m/s, which is in accordance with the results presented in /SKB 2005a/.

The software applications used for the analytical calculations were as indicated in Table 7-2. The calculations were performed as Monte Carlo simulations with 10,000 simulations. A more detailed description of the analytical calculation models is given in Appendix 2 in /SKB 2004a/.

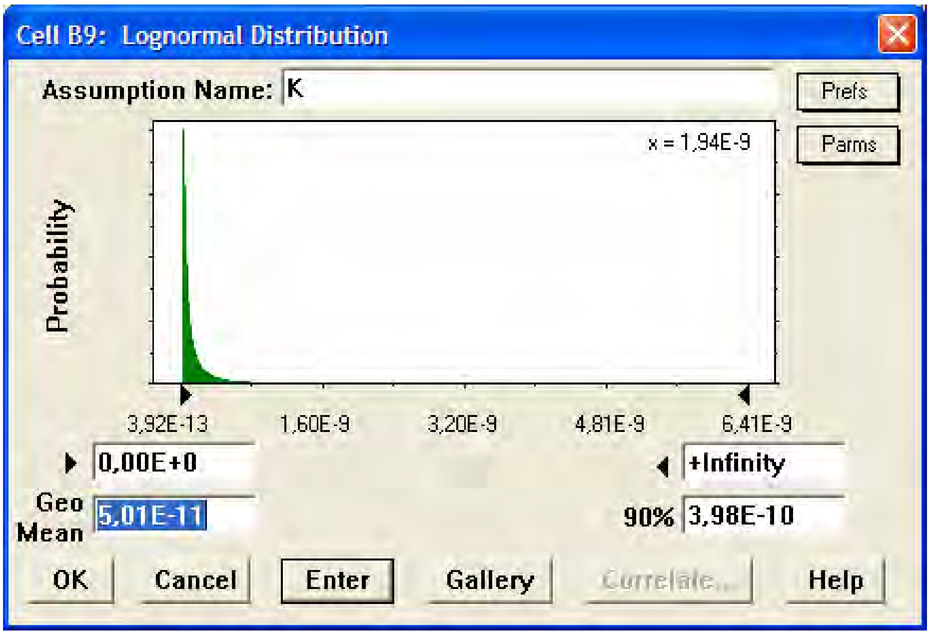


Figure 7-2. *Chosen lognormal distribution of hydraulic conductivity.*

Table 7-2. Software used for the analytical calculations.

Software	Version	Developer	Modules used	Comments
Excel 2003	(11.6113.5703)	Microsoft Corporation	General calculation procedures	Calculation generator for Monte Carlo simulations with Crystal Ball
Crystal Ball	Version 6.0	Decisionering, Inc., Denver	Excel add-in for Monte Carlo simulations	Monte Carlo simulation
Statistica	Version 7	StatSoft, Inc.	Basic and descriptive statistics	Distribution function and general statistics

Seepage into deposition tunnels

The deposition tunnel was simulated as a circular horizontal drain without deposition holes. As an analytical simplification the calculations were performed for a single tunnel without other open tunnels in the vicinity and without grouting. A simplified model for steady-state seepage in accordance with Equation 7-1 was used (see /Alberts and Gustafson 1983/ and Figure 7-3).

$$q_s = \frac{2\pi\, K_b d}{\ln\left[\frac{2d}{r_w}\right] + \xi} \qquad \text{Equation 7-1}$$

where:

q_s = steady-state seepage into deposition tunnel (m^3/s,m)

d = depth of centre of deposition tunnel below groundwater table (m)

r_w = radius of deposition tunnel = $[Atunnel/(\pi)]^{0.5}$ (m)

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

ξ = natural skin factor of deposition tunnel (dimensionless)

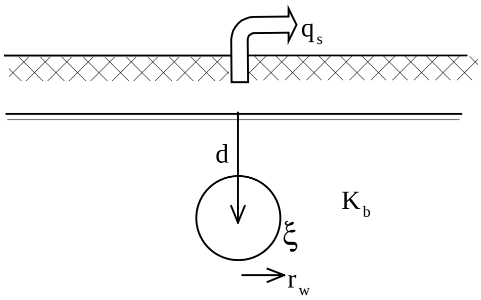


Figure 7-3. *Calculation model used for the analytical calculations of ungrouted deposition tunnels (from /SKB 2004a/).*

Seepage to the repository

Seepage to the repository at different points in time (excavation stages) and for the sealing level 1×10^{-9} m/s were analysed by means of Equation 7-2. In this case the repository is regarded as a large-diameter well with a constant draw down.

$$Q_S = \frac{2\pi T_b \Delta s}{\ln\left[\frac{R_0}{r}\right] + \left[\frac{K_b}{K_t} - 1\right]\ln\left[1 + \frac{m}{r}\right] + \sigma} \qquad \text{Equation 7-2}$$

where

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

K_t = hydraulic conductivity of the grouted rock (m/s)

m = thickness of grouting (m)

Q_s = seepage under steady-state conditions (m^3/s)

r = distance, equal to equivalent radius of the repository, r_w (m)

R_0 = distance to boundary conditions (m)

T_b = representative transmissivity of rock mass (m^2/s)

Δs = draw down, equal to depth of the repository, D (m)

σ = skin factor inside grouting (dimensionless)

Equation 7-2 applies only to a confined aquifer and steady-state conditions. In the calculations the transmissivity was derived by the formula $T_b=K_b(h_0+h_w)/2$, which converts Equation 7-2 from a confined to an unconfined aquifer, where $(h_0+h_w)/2$ is the mean height of the groundwater level.

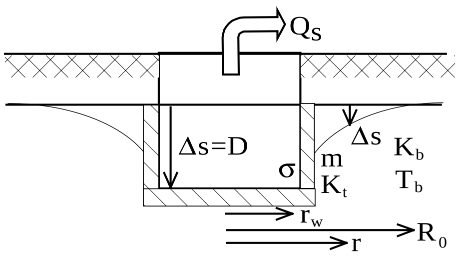


Figure 7-4. *Analytical model for steady-state seepage calculation (from /SKB 2004a/).*

Distance of influence

The influence radius at steady-state in rock was estimated for different points in time and a sealing level of 1×10^{-9} m/s using Equations 7-3, 7-4 and 7-5 for different seepage. The influence radius was also checked by means of a water balance calculation, making sure that the groundwater recharge (m/s) on the lowered groundwater table (m) and any seepage from the boundaries is equivalent to the seepage into the repository (m^3/s). The parameters in the equations are also shown in Figure 7-5.

$$R_0 = r_w e^{\left[\frac{2\pi T_b \Delta s}{Q_s}\right]} \qquad \text{Equation 7-3}$$

$$h_0^2 - h^2 = \frac{Q_s}{\pi K}\ln\left[\frac{R_0}{r}\right] - \frac{W}{2K}\left[R_0^2 - r^2\right] \qquad \text{Equation 7-4}$$

$$Q_s = AW = \pi r_Q^2 W \qquad \text{Equation 7-5}$$

where

A = area (m^2)

h = hydraulic head (m)

h_0 = hydraulic head at outer boundary condition (m)

K = representative hydraulic conductivity for rock and grouting (m/s)

Q_s = seepage under steady-state conditions (m^3/s)

r = distance, equal to equivalent radius of the repository, r_w (m)

r_w = representative radius of repository (m)

R_0 = distance to boundary conditions (m)

T_b = representative transmissivity of rock mass (m^2/s)

W = groundwater recharge in rock (m/s)

Δs = draw down (m)

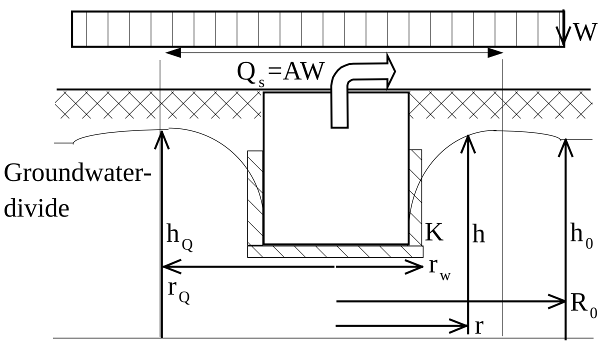


Figure 7-5. *Analytical model to define the distance of influence (from /SKB 2004a/).*

Upconing of saline water

Upconing height and maximum critical seepage at steady-state were calculated approximately using Equation 7-6. Discharge of groundwater from the repository causes saline water at depth to rise. Stable upconing of the saline water interface is judged to be no more than about 0.25–0.60 of the distance between the lowest part of the deep repository and the saline water interface. Greater upconing height entails an increased risk of upconing and a higher salinity of seepage water (TDS). The parameters in the equations are also shown in Figure 7-6.

$$
\begin{aligned}
&h_{tds} = \frac{Q_s}{2\pi\left[\left(\rho_s - \rho_f\right)/\rho_f\right]Kd_s} \\
&Q_{\max} = 2\pi d_s^{\,2} K\left[\left(\rho_s - \rho_f\right)/\rho_f\right] \\
&Q_{design} = Q_{\max} C \\
&h_{kr} = d_s C \\
&\left(C \approx 0.25 - 0.60\right)
\end{aligned}
\qquad \text{Equation 7-6}
$$

where

d_s = distance between bottom of deep repository and saline water interface (m)

h_{kr} = critical upconing height for unstable equilibrium (m)

h_{tds} = upconing height of saline water interface under steady-state conditions (m)

K = representative hydraulic conductivity for rock and grouting (m/s)

Q_s = seepage under steady-state conditions (m³/s)

ρ = density of saline groundwater ρ_s and non-saline groundwater ρ_f (kg/m³)

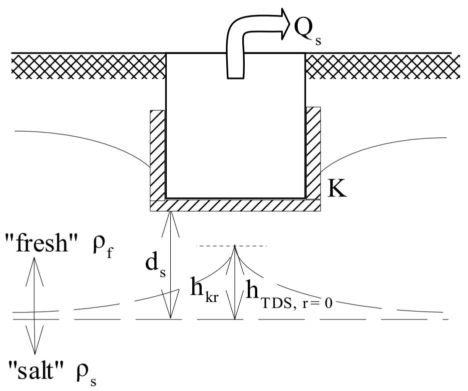


Figure 7-6. *Analytical model for salt water upconing under steady-state conditions (from /SKB 2004a/).*

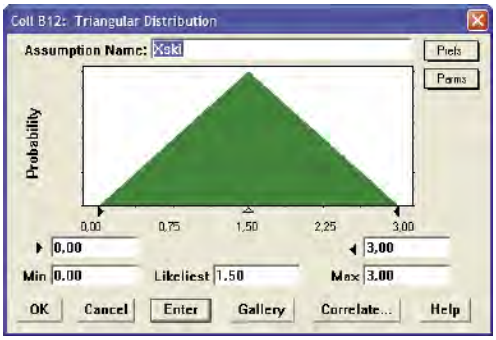
Summary of calculation models and input data

In Table 7-3 a summary of calculation models and input data are shown.

Table 7-3. Summary of calculations models and input data.

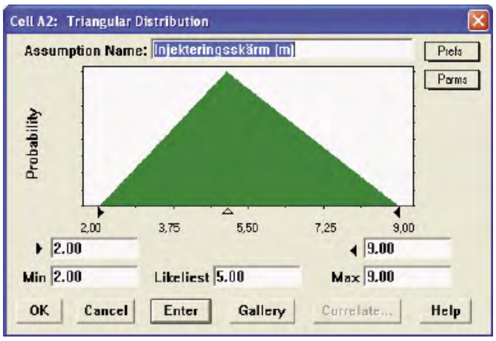
1. Seepage to deposition tunnels

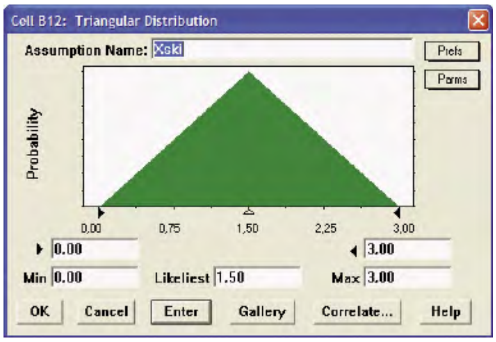
$$q_s = \frac{2\pi\, K_b d}{\ln[\frac{2d}{r_w}] + \xi}$$

Input data	Premise/assumption	Comment
K_b	Lognormal distribution, median 5×10^{-11} m/s and 90%-percentile 4×10^{-10} m/s	
D	400 m and 500 m	Premise for design
r_w	Deposition tunnel 4.9 × 5.4 m, corresponds to a diameter of about 6.1 m (radius 3.05 m)	Premise for design
ξ	Triangular distribution, 0 to 3	

2. Seepage to repository

$$Q_S = \frac{2\pi T_b \Delta s}{\ln\left[\frac{R_0}{r}\right] + \left[\frac{K_b}{K_t} - 1\right]\ln\left[1 + \frac{m}{r}\right] + \sigma}$$

Input data	Premise/assumption	Comment
T_b	$T_b = K_b h_{avg} = K_b(h_0 + h_w)/2$ where h_0 = repository depth	
Δs=D	400 m och 500 m	Premise for design
R_0	500 m when r=50 m; 2,500 m when r=500 m	< 2,500 m according to /SKB 2004a/
$r=r_w$	50 m and 500 m	Assumed values for different points in time. Correspond to the extent of the repository in the initial operation and in the regular operation respectively.
K_b	Lognormal distribution with a median conductivity of 5×10^{-11} m/s and a 90%-percentile of 4×10^{-10} m/s	
K_t	1×10^{-9} m/s	
m	Triangular distribution, 2 to 9 m.	

σ	Triangular distribution, 0 to 3	

3. Distance of influence

$$R_0 = r_w e^{\left[\frac{2\pi T_b \Delta s}{Q_s}\right]}$$

$$h_0^2 - h^2 = \frac{Q_s}{\pi K}\ln\left[\frac{R_0}{r}\right] - \frac{W}{2K}\left[R_0^2 - r^2\right]$$

$$Q_s = AW = \pi r_Q^2 W$$

Input data	Premise/assumption	Comment
R_0	See 2 above, Seepage to repository	
$r=r_w$	See 2 above, Seepage to repository	
T_b	See 2 above, Seepage to repository	
Δs	See 2 above, Seepage to repository	
Q_s	Result from 2, Seepage to repository	
K	$K=K_b$ at repository depth	Representative hydraulic conductivity for rock and grouting. Assumption is most likely conservative.
W		Used for evaluation of result.

4. Upconing of saline water

$$h_{tds} = \frac{Q_s}{2\pi\left[(\rho_s - \rho_f)/\rho_f\right] K d_s}$$

$$Q_{\max} = 2\pi d_s^{\,2} K\left[(\rho_s - \rho_f)/\rho_f\right]$$

$$Q_{design} = Q_{\max} C$$

$$h_{kr} = d_s C$$

$$(C \approx 0{,}25 - 0{,}60)$$

Input data	Premise/assumption	Comment
Q_s	See 2 above, Seepage to repository	Late point in time, r=500
ρ_s	1,060 kg/m^3	
ρ_f	1,005 kg/m^3	
K	$K=K_b$	Representative hydraulic conductivity for rock and grouting. Assumption is most likely conservative.
d_s	800 m to 1,200 m, most likely value is 1,000 m	

7.3.2 Numerical calculations

Numerical calculations were undertaken by means of DarcyTools in order to simulate the following:

- Seepage to the tunnels.
- Upconing of saline water to the repository.
- Extent of influence area.

DarcyTools was adopted in order to meet the objectives above. DarcyTools version V3.0 was employed in the study.

The properties used are the same as used in /SKB 2005a/. The new version V3.0 employs a free-surface algorithm with the main objective of simulating the repository's possible interaction on the near-surface hydrology. As a consequence the near-surface properties require calibration, in particular the hydraulic conductivity. The draw down above the repository also requires some consideration with regard to the conductivity. It should be emphasised that this study has not included any "fine trimming" of the surface hydrology part of the model as this is beyond the scope of the current design. The values adopted are therefore based mainly on earlier experience and some test simulations.

The following equations and algorithms are employed in DarcyTools:

- Conservation of mass, including the effects of a variable density and specific storativity.
- A transport equation for salinity.
- A transport equation, used as a tracer, for precipitation water.
- The Darcy equation, including the gravitational term.
- The subgrid model FRAME, based on the multi-rate diffusion model, is used for both salinity and the precipitation tracer.
- The groundwater table is tracked with a free-surface algorithm which can handle both natural conditions and the draw down due to the repository.
- A tunnel routine puts atmospheric pressure in all computational cells in contact with the repository. All cell walls of these cells have a specified maximum conductivity.

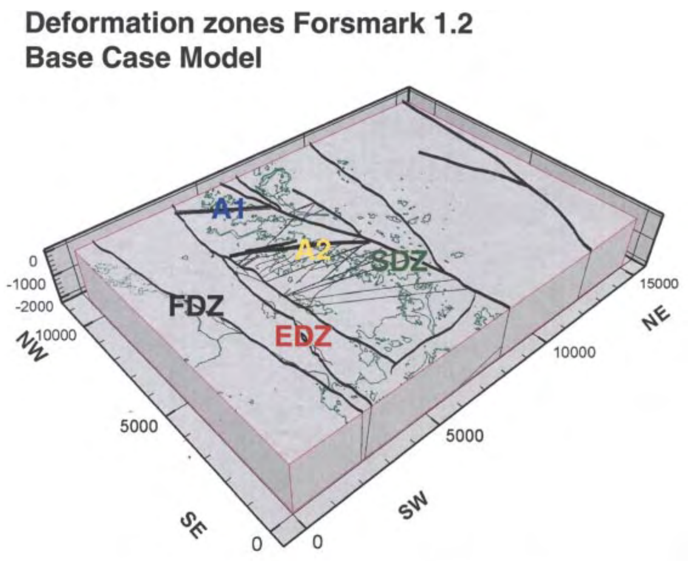
The case modelled is outlined conceptually in Figure 7-7. The key deformation zones are the Singö zone (SDZ), the Eckarfjärden zone (EDZ), and the gently dipping deformation zones A1 and A2. In addition to these zones a set of smaller deformation zones is also represented in the base case model. Between these zones the base case defines continuous porous medium (CPM) blocks, designated CPM1, CPM2 and CPM3. The repository is located in CPM3.

The hydraulic conductivity of the rock blocks were selected as (from /SKB 2005a/):

$K_{min} = 1\times10^{-9}$ m/s (CPM1).

$K_{min} = 5\times10^{-10}$ m/s (CPM2).

$K_{min} = 1\times10^{-11}$ m/s (CPM3).



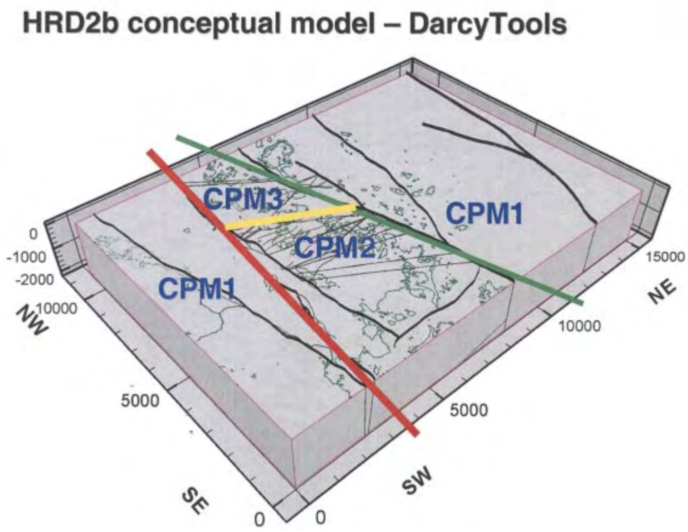


Figure 7-7. *The base case model. Deformation zones (top) and rock blocks (CPM) between the zones (lower), scales are in m (from /Svensson 2005/).*

7.4 Results

7.4.1 Analytical calculations

The analytical calculations are based on the properties given by the HydroDFN in the provided "preliminary data", which, however, is in the range presented in /SKB 2005a/. The simulations focused on the rock at repository depth and the strong contrast in conductivity between the upper part of the rock mass (< 200 m). However, a separate analysis was made regarding the seepage to the ramp and shafts from the shallow sub horizontal fracture zones.

Seepage into deposition tunnels

The results are presented in Table 7-4 for depths 400 m and 500 m. The calculations were performed as Monte Carlo simulations with 10,000 simulations.

Table 7-4. Calculated seepage to ungrouted deposition tunnels at repository depth.

Depth	Probability for seepage less than	Seepage l/s and 100 m	l/min and 100 m
400 m	50%	2.0×10^{-3}	0.1
	90%	1.5×10^{-2}	0.9
	95%	2.8×10^{-2}	1.7
	99%	8.6×10^{-2}	5.2
500 m	50%	2.0×10^{-3}	0.1
	90%	1.9×10^{-2}	1.1
	95%	3.3×10^{-2}	2.0
	99%	1.0×10^{-1}	6.2

Seepage to the repository

Water seepage into the repository is expected to be very low. The majority of the seepage will take place in the access ramp and shaft, which penetrate the more pervious sub horizontal fracture zones at shallow depth.

The inflow from the sub horizontal fracture zones was calculated for the depths 50 m, 100 m and 150 m, with a hydraulic conductivity of the zones ranging from 1×10^{-3} to 1×10^{-5} m/s. The calculations were executed by means of Equation 7-1 and 7-2, and the result is presented in Table 7-5 and 7-6. However, due to the strong hydraulic interference between bore holes (see /SKB 2005a/) it is assumed that the total inflow to the ramp and the shafts in the central area can be assessed as the inflow to the ramp. The reason is that the calculated inflow to the ramp is based on a condition with good hydraulic connection between the underground opening and the surrounding groundwater. Furthermore, an unlimited availability of water for infiltration, which is assumed in the calculations, will probably result in an overestimation of the inflow to the ramp. More accurate analyses may be executed if results from large scale interference tests are available.

Regarding the two shafts for exhaust air in the deposition area, the total inflow from these shafts is assessed as the inflow from one shaft. For the assessment of inflow due to the sub horizontal zones, it has been assumed that the interference presented in /SKB 2005a/ is somewhat reduced due to the grouting in shafts and ramp.

To sum up, when considering the probable interference, the total inflow to the underground openings from the sub horizontal zones is assumed to be the sum of the inflow to the ramp and one shaft.

The inflow of water to a repository located at 500 m depth and sub horizontal fracture zones at 150 m depth is given in Table 7-7. Since the major inflow is associated to the sub horizontal zones only a minor difference in inflow will be expected at 400 m depth. If the sub horizontal zones are located shallower, the inflow will decrease accordingly.

It should be noted that it is probably difficult to achieve a sealing level of 1×10^{-9} m/s when grouting a fractured pervious rock mass. A sealing level of 1×10^{-7} to 1×10^{-8} m/s is probably more likely.

Table 7-5. Inflow to the ramp from a sub-horizontal fracture zone. The q_{50m} – value is defined as the inflow corresponding to a contact length of the zone and ramp equal to a vertical zone with a width of 50 m.

Depth	Probability that calculated inflow is lower	Seepage to ramp			
		Sealing level 1×10^{-7} m/s		Sealing level 1×10^{-9} m/s	
		q_s l/s, m	Q_{50m} l/s	q_s l/s, m	Q_{50m} l/s
50 m	50%	0.03	1.5	0.0003	0.02
	90%	0.05	2.5	0.0005	0.03
	95%	0.05	2.5	0.0006	0.03
	99%	0.07	3.5	0.0008	0.04
100 m	50%	0.06	3.0	0.0007	0.04
	90%	0.09	4.5	0.0010	0.05
	95%	0.11	5.5	0.0012	0.06
	99%	0.15	7.5	0.0016	0.08
150 m	50%	0.09	4.5	0.0010	0.05
	90%	0.14	7.0	0.0015	0.08
	95%	0.16	8.0	0.0018	0.09
	99%	0.22	11	0.0024	0.12

Table 7-6. Inflow to a shaft from a sub-horizontal fracture zone assuming a zone with a thickness corresponding to the median thickness of the data set, i.e. 2.3 m.

Depth	Probability that calculated inflow is lower	Seepage to a shaft	
		Sealing level 1×10^{-7} m/s	Sealing level 1×10^{-9} m/s
		Q l/s	Q l/s
50 m	50%	0.04	0.00
	90%	0.9	0.01
	95%	2.0	0.02
	99%	8.2	0.12
100 m	50%	0.1	0.00
	90%	1.7	0.02
	95%	4.0	0.05
	99%	17	0.24
150 m	50%	0.1	0.00
	90%	2.6	0.03
	95%	6.0	0.07
	99%	25	0.37

Table 7-7. Calculated seepage to a repository at 500 m depth, at different stages of excavation expressed as probabilities of the seepage being less than the figure indicated.

Depth	Probability	Seepage to tunnels at 500 m depth (sealing level 1×10^{-9} m/s)			
		Early stage		Late stage	
		l/s	l/min	l/s	l/min
Repository depth	50%	2.5×10^{-2}	1.5	4.6×10^{-2}	2.8
	90%	0.2	11	0.4	21
	95%	0.3	21	0.7	40
	99%	1.0	59	1.8	109
		Seepage to grouted ramp and shafts			
		Sealing level in sub-horizontal zone, 1×10^{-7} m/s		Sealing level in sub-horizontal zone, 1×10^{-9} m/s	
		l/s	l/min	l/s	l/min
Effect from ramp and shafts (depth of sub horizontal zone is 150 m)	50%	4.6	276	0.05	3.1
	90%	9.6	576	0.11	6.6
	95%	14	840	0.16	9.6
	99%	36	2,160	0.49	29

Distance of influence

The very low seepage will cause the distance of influence to be limited. The analytical calculations give a distance of influence of about 200 m for the early stage and 300 m when the repository reaches its greatest extent. The distance is measured from the outer rim of the repository to the point at which no influence of the hydraulic head may be observed. The result for a repository located at 400 m depth is given in Table 7-8. The results reflect the conditions with ungrouted tunnels at repository depth. The results for the 500 m depth are very similar to the 400 m depth. The calculations were performed as Monte Carlo simulations with 10,000 simulations.

Table 7-8. Calculated distance of influence from a repository at 400 m depth, at different stages of excavation expressed as probabilities of the distance being less than the figure indicated.

Depth	Probability	Distance of influence (m) Early stage	Late stage
400 m	50%	205	331
	90%	252	1,407
	95%	302	1,845
	99%	642	2,540

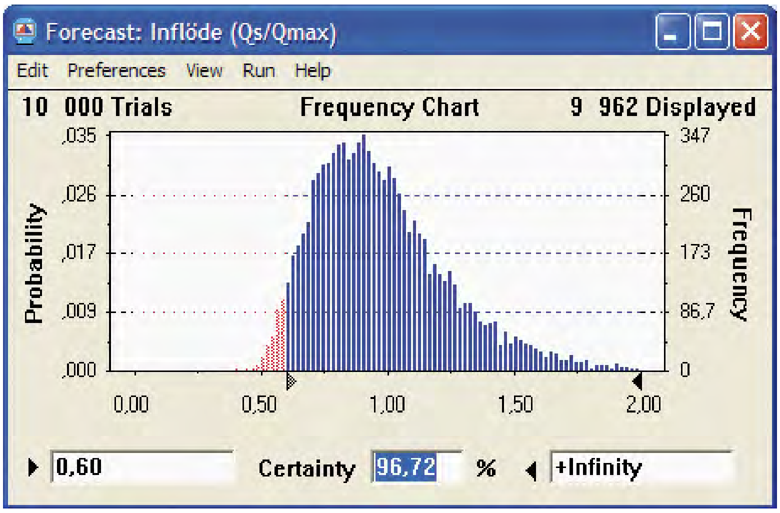


Figure 7-8. *Probability distribution of the risk of increased TDS in the seepage water. Dimensionless scale.t*

Upconing of saline water

The analytical calculations suggest that there is a risk for upconing saline water with an increase of the TDS in the seepage water into the repository. The simulations indicate a 96% probability that saline water will affect the repository. The result of the Monte Carlo simulation is presented in Figure 7-8, expressed as the ratio Q_s / Q_{max} (where Qs = seepage under steady-state conditions and Q_{max} = the density driven flow from below), which should be below 0.6 to avoid an increase of TDS in the seepage water.

7.4.2 Numerical calculations

Seepage to repository

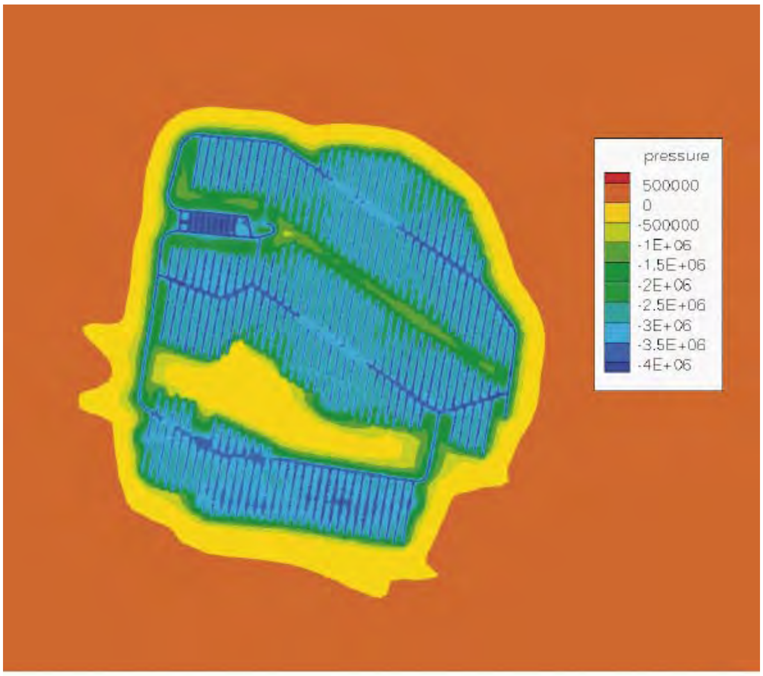
The calculated seepage to the complete repository including ramp and shafts at a late stage of excavation is presented in Table 7-9 for different sealing levels. The repository is located at a depth of 500 m. However, it should be noted that the uppermost 100 m of ramp and shafts are not included. Table 7-9 clearly shows that the total seepage is very small for the adopted model of the fracture system and rock blocks.

Table 7-9. Seepage to the repository as a function of applied sealing levels at a late stage of excavation.

Sealing level (m/s)	Seepage (l/s)	Seepage (l/min)
Ungrouted	4.0	240
1×10^{-7}	4.0	240
1×10^{-9}	1.9	114
1×10^{-11}	0.05	3

Distance of influence

One of the few effects of different sealing levels is illustrated in Figure 7-9. If no grouting is executed, the pressure will be nearly uniform in the repository area, while the in-situ ground water pressure dominates when grouting is significant. In the un-grouted case the distance of influence will be a few hundreds of meters, whereas when grouting is successful (corresponding to a sealing level of 1×10^{-11} m/s) the distance will be approximately 100 m. If a sealing level of 1×10^{-7} m/s or 1×10^{-9} m/s is achieved, which is assumed to be more likely, the distance of influence will consequently be in the range of 100 m to a few hundreds of meters.



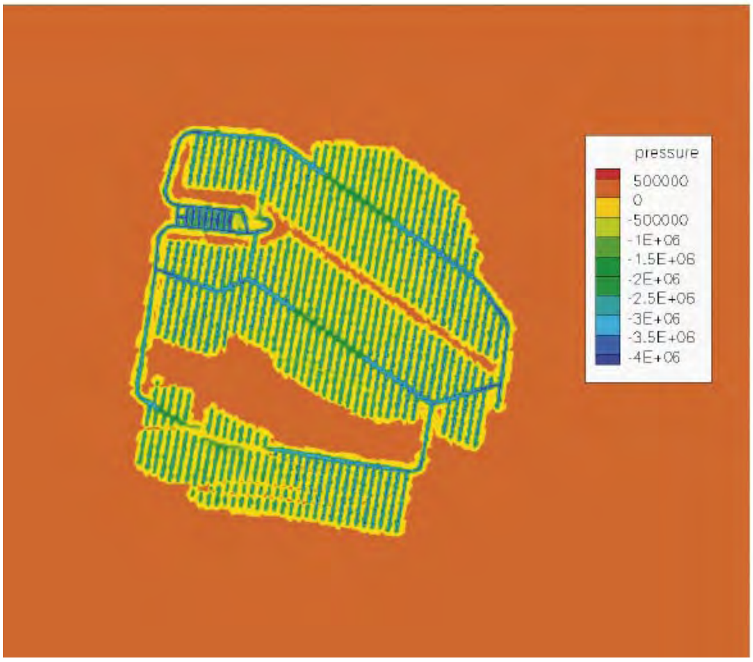


Figure 7-9. *Pressure distribution around the repository, at 415 m below sea level. No grouting applied (top) and maximum hydraulic conductivity specified as 10^{-11} m/s (lower) (from /Svensson 2005/).*

Upconing of saline water

The effect on the salinity field is illustrated in Figure 7-10. Only the case without grouting is indicated, as the effect is small even for this case. In Figure 7-11 the layout at 400 m depth is illustrated together with the coordinate system used in Figure 7-10.

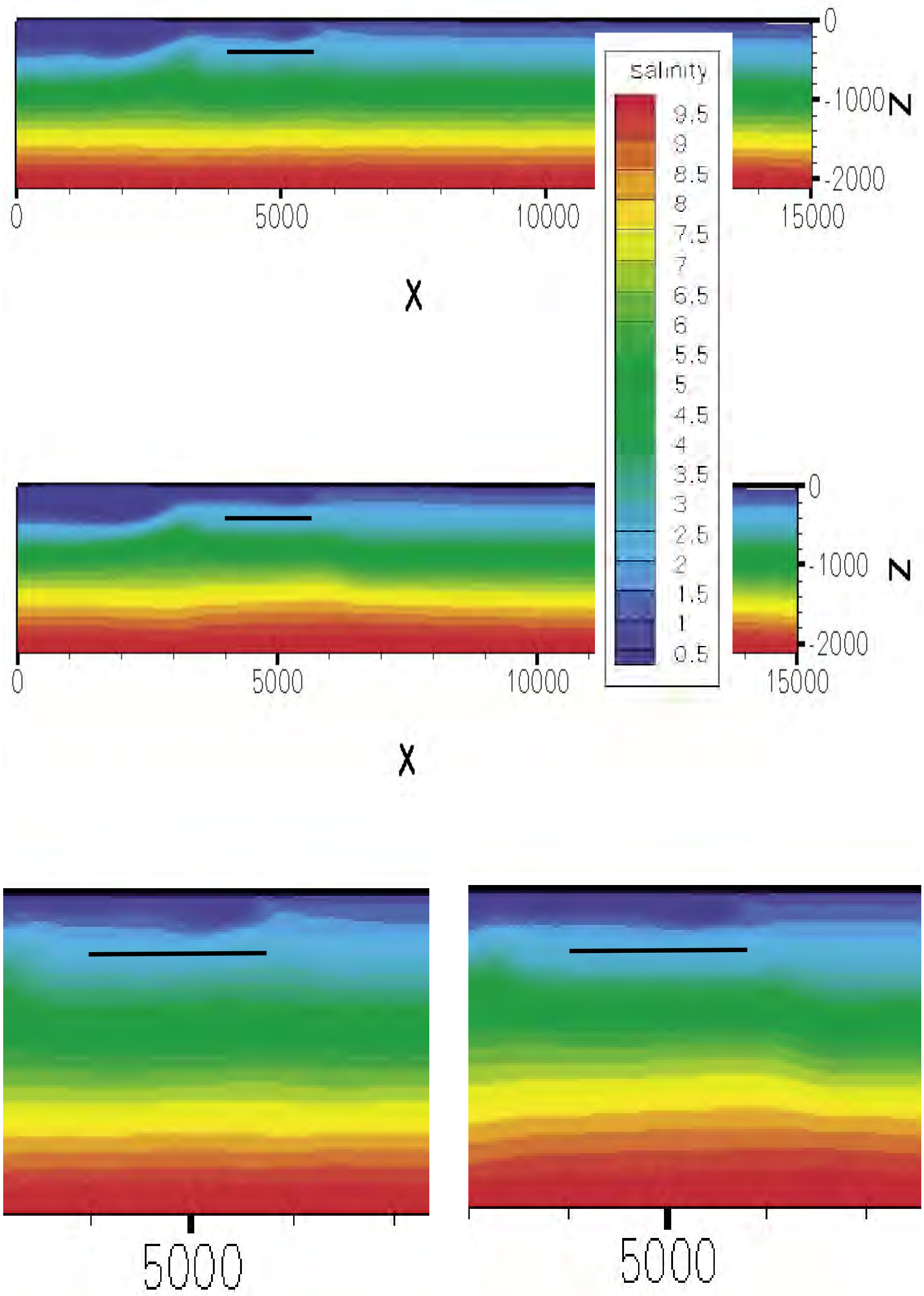


Figure 7-10. *Salinity (in %) distribution in a west-east vertical section through the repository. Natural condition (top, and lower left) and with a repository present, with no grouting applied (middle and lower right). Scales in m. The black line indicates the extension of the repository at 400 m depth. X represents the horizontal distance along the x-axis and N the depth according to definitions given in /Svensson 2005/.*

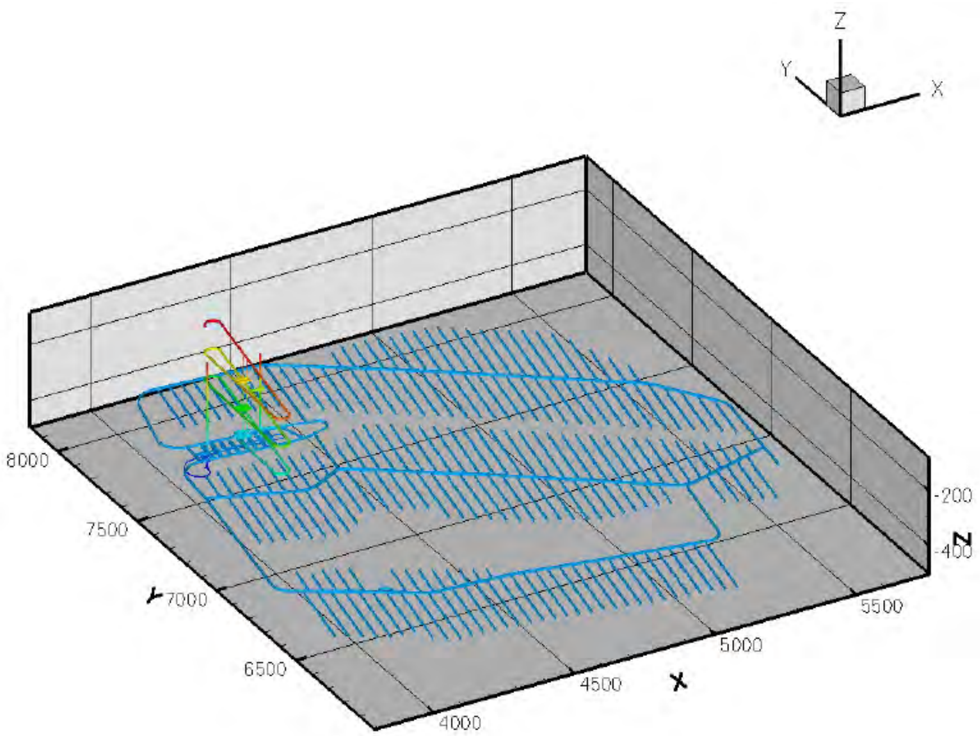


Figure 7-11. Layout of the repository, 400 m depth (Layout 1). The x and y coordinates refer to the local system in the regional model. The y-direction points to North-West (from /Svensson, 2005/).

7.5 Conclusions

According to the numerical calculations the seepage to the repository (below 100 m depth) will be in the range of 114–240 l/min for sealing levels 1×10^{-7}–1×10^{-9} m/s. The analytical calculations resulted in a lower seepage, but these calculations did not include the deterministically determined deformation zones. In addition to the calculated inflow given above, there will also be an inflow from the shallow sub-horizontal fracture zones. This inflow was estimated based on analytical calculations only and resulted in an inflow of 3–276 l/min (50% probability values), depending on the achieved sealing level. The total inflow to the repository is thus expected to be approximately 100–500 l/min.

The analytical calculations do not take into account the large differences in hydraulic conductivity between the shallow, water bearing rock mass and the almost impervious rock mass at repository depth below the deformation zone ZFMNE00A2. Numerical modelling is more capable of modelling the actual variations.

The small quantity of seepage also implies that the distance of influence will be limited. The analytical calculations suggest that there may be an influence at repository depth at a distance of 200 m during the early stages, and 300 m when the repository has been excavated to full extent. According to the numerical calculations an influence on the head distribution may be observed at a distance of approximately 100 m. In addition to studying the impact on the hydraulic head, the numerical modelling also indicated that there will be no influence of groundwater levels at the surface.

No upconing of saline water is discernible in the numerical modelling, whereas the analytical simulations indicate a high risk of saline water migration from below leading to an increase of TDS in the seepage water. The discrepancy between the tools used is probably attributable to the analytical model having been developed to avoid an increase in salinity in coastal wells. The model is based on the density difference between fresh and saline water, which is used as the driving force. For a repository at 500 m depth with the fresh/saline-water interface initially located at a depth of 1,500 m, the model simulates the effect of a well which is pumped with a draw down of 500 m, rendering it highly probable that an increase in salinity will be observed. Long-term monitoring of the seepage water is therefore likely to reveal a slow increase in salinity, although the total TDS will remain extremely low.

The analytical and numerical calculations performed clearly demonstrate that the very tight rock at the repository level will result in favourable conditions for the repository with low seepage rates. It also results in a very limited influence on the hydraulic situation around the repository, and no influence on the groundwater table close to the ground surface.

In conclusion, there are no major differences when using data in /SKB 2005a/ compared to the findings in the design (“preliminary data”).

8 Assessment of rock grouting need

8.1 Introduction

Assessment of grouting procedures and estimation of grout take were made in order to constitute a basis for analysis of groundwater composition, as well as a basis for cost calculations.

In design step D1 the assessment of rock grouting procedures and estimation of grout take were carried out in the following two steps:

1. Assessment of suitable grouting procedures (for example number of grouting sequences and grout composition) for specified sealing levels. According to /SKB 2004a/ it is assumed that only cement based grouts shall be used.
2. Estimation of the quantity of grout take for specified points in time (excavation stages) and sealing levels (grouting levels according to /SKB 2004a/).

Sealing levels and points in time were introduced in Chapter 7 (see Sections 7.2.3 and 7.2.4).

It should be noted that the assessment of grouting procedure was based on site conditions assessed from "preliminary data". The site conditions with respect to the hydraulic properties of the rock mass have been described in Chapters 6 and 7. From a comparison with information given in /SKB 2005a/ it was concluded that that the hydraulic conductivities assessed in Chapter 6, which were based on "preliminary data", are higher than those given in /SKB 2005a/. The difference is pronounced for the deterministically determined deformation zone ZFMNE0060. With regard to the rock mass between the deformation zones, it can be concluded that the corresponding differences in hydraulic conductivities are small. Thus, the estimated quantity of grout take would be less if the calculation was based on information from /SKB 2005a/. A more precise estimation of grout quantities will require further calculations.

8.2 Input data and assumptions

8.2.1 Sealing levels

It was decided together with SKB that the following two sealing levels, corresponding to level 1 and level 2 according to /SKB 2004a/, should be used as the basis for the assessment of grouting procedures and grout take (see Chapter 7):

- Sealing level 1: $K_t = 1\times10^{-7}$ m/s.
- Sealing level 2: $K_t = 1\times10^{-9}$ m/s.

The symbol K_t denotes the hydraulic conductivity of the grouted zone around the tunnel.

It should be noted that the sealing levels given in /SKB 2004a/ are given as preliminary values only. These values were considered suitable since these sealing levels can be anticipated to be achieved when using cement based grouts. Alternative grouting media must be used in order to achieve sealing levels corresponding to lower hydraulic conductivities.

8.2.2 Points in time (excavation stages)

According to Chapter 7 the excavation of the deep repository has been divided into one early and one late stage of excavation, defined as a radius of the repository equal to 50 m and 500 m respectively. Thus the radius in the early point in time is assumed to be 10% of the radius in the late point in time. For the purpose of simplification it has been assumed that the quantity of grout in the early stage of excavation corresponds to approximately 10% of the volume for the complete repository.

8.2.3 Site specific conditions for grouting

The following input data were used for execution of the specific design task:

1. Layouts of the hard rock facility at a repository depth of 400 m according to Chapter 5.
2. Properties of deformation zones and identified problems involved in passing through these zones according to Chapter 6.
3. Hydrogeological conditions of the rock mass between deformation zones, sealing levels, and calculation of inflows to the hard rock facility according to Chapters 6 and 7.

These input data are described in greater detail in the following.

Since no requirements for acceptable levels of inflow of water have been specified, the assumption of grouting procedures has been based on the sealing levels specified above as well as on the hydraulic properties of the different parts of the rock mass and the composition of the groundwater. The groundwater composition affects the grouting procedure in terms of the chemical composition of the grout.

Table 8-1 presents a summary of typical hydraulic conductivities for different parts of the rock mass.

It can be concluded from Table 8-1 that the rock mass including deformation zones is associated with low hydraulic conductivities, and that a sealing level of 1×10^{-7} m/s only applies for the sub-horizontal near-surface zones and the deformation zone ZFMNE0060. The hydraulic conductivity of other parts of the rock mass is lower than 1×10^{-7} m/s.

One aspect influencing the sealing effect is the type of rock mass to be grouted. Based on experience of normal grouting procedure it should be possible to achieve a hydraulic conductivity of about 1×10^{-8} m/s. Using a controlled grouting procedure, values of 1×10^{-9}–1×10^{-10} m/s have been reported from grouting tests at Äspö HRL /Emmelin et al 2004/. These grouting tests were performed in a rock mass with a number of discrete fractures surrounded by very tight rock. For more fractured water-bearing zones containing both large and small fractures it can be concluded from experience that a grouting result

Table 8-1. Hydraulic conductivity of different parts of the rock mass in which the hard rock facility is located (based on "preliminary data").

Part of rock mass	Hydraulic conductivity, K m/s
Rock mass between deformation zones	1×10^{-14} / 5×10^{-11} / 2×10^{-9} (1% percentile, median and 99% percentile values)
	Hydraulic conductivities of 1×10^{-8}–1×10^{-10} are assumed to represent rock mass with stochastically determined fractures/fracture zones.
Deformation zones	See Table 6-2

corresponding to hydraulic conductivities of about 1×10^{-7}–1×10^{-8} m/s is more likely (see for example /Chang et al 2005/). Based on this experience, it is assumed in the forthcoming analyses that a sealing level of 1×10^{-8} m/s is possible to achieve for the sub-horizontal near-surface zones. For other parts of the rock mass described in Table 8-1 it is reasonable to assume that a sealing level of 1×10^{-9} m/s can be achieved.

In Table 8-2 the sealing levels used in the calculation of grout take are presented.

The sealing levels which can be achieved are controlled by the mechanisms affecting the penetration and spreading of grout into the fracture system of the bed rock. Different grout compositions have different penetration and reological properties, influenced for example by the particle size and distribution of the cement, the mixing procedures and the additives used in the grout. The design of the grouting procedures must therefore include requirements for the grout in terms of the minimum aperture that needs to be grouted. The calculation of apertures requiring grouting should preferably be analysed in 3D, since fracture systems are complex. More detailed analyses of which tightness that can be achieved by grouting in different parts of the rock mass must therefore be performed. For later stages in the design it is also recommended that alternative grouting materials should be studied if the maximum acceptable ingress of water will be low. Alternative grouts may comprise special types of cement and/or chemical solutions. New grouts and/or grouting methods may also require to be developed.

It should be noted that large fractures and cavities of between several centimetres and one decimetre were observed when performing BIPS investigations in boreholes intersecting sub-horizontal near-surface zones (see Figure 8-1).

When designing a suitable grout composition, the ground water quality must be considered. No grout composition requirements are specified in /SKB 2004a/ with respect to groundwater quality. However, according to advisory text in BV-Tunnel /Banverket 2002/, which according to /SKB 2004a/ may be regarded as best practice, sulphate-resisting cement should be used if the sulphate content (mg/l SO_4^-) of the groundwater ≥ 600 mg/l. From analysis of water sampled in boreholes KFM02A, KFM03A and KFM04A it can be concluded that the sulphate content at the repository depth is < 600 mg/l /Wacker et al. 2004a–c/.

Table 8-2. Sealing levels for different parts of the rock mass.

Part of rock mass	Hydraulic conductivity, K, m/s	Sealing levels (corresponding to conductivity of grouted zone around tunnel, K_{inj}), m/s
Rock mass between deformation zones	1×10^{-14} / 5×10^{-11} / 2×10^{-9} (1% percentile, median and 99% percentile values) Hydraulic conductivities of 1×10^{-8}–1×10^{-10} are assumed to represent rock mass with stochastically determined fractures/fracture zones.	$K_{inj} = 10^{-9}$ Grouting is assumed to be required in rock mass with stochastically determined fractures/fracture zones (where conductivity, $K > 1\times10^{-9}$ m/s).
Sub-horizontal near-surface zones, depth 0–200 m	1×10^{-4}–1×10^{-6}	$K_{inj} = 10^{-7}$ $K_{inj} = 10^{-8}$
ZFMNE0061	1×10^{-7}–1×10^{-9}	$K_{inj} = 10^{-9}$
ZFMNE0060	1×10^{-6}–1×10^{-8}	$K_{inj} = 10^{-7}$ $K_{inj} = 10^{-9}$
ZFMNE0401	1×10^{-8}–1×10^{-9}	$K_{inj} = 10^{-9}$
ZFMNE103A and B	1×10^{-8}–1×10^{-9}	$K_{inj} = 10^{-9}$
ZFMNE1188	1×10^{-8}–1×10^{-10}	$K_{inj} = 10^{-9}$

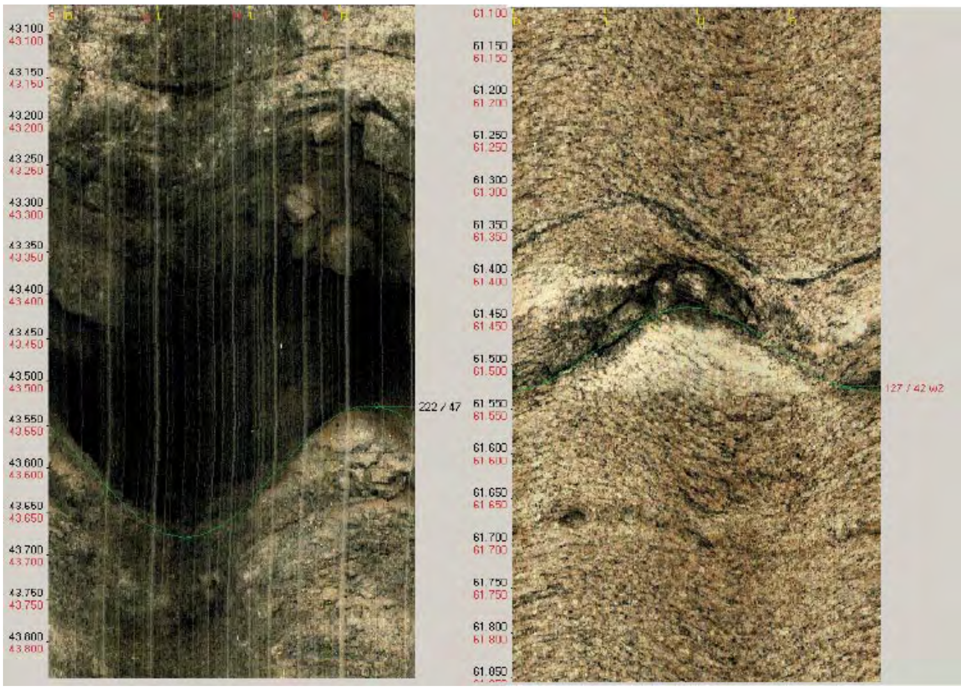


Figure 8-1. *Fractures and cavities in the sub-horizontal near-surface zones (BIPS pictures from /Nordman 2003ab/)***.**

8.3 Grouting procedures

8.3.1 General

In design step D1 the proposed grouting procedures were used mainly for calculation of costs and evaluation of groundwater composition /SKB 2004a/. A more detailed design of grouting procedures will be performed in later design steps. Approximate calculations together with experience from other grouting works were used in design step D1. The state-of-the-art in grouting of hard rock described in /Eriksson and Stille 2005/ was also used.

According to /SKB 2004a/ it has also been assumed that grouts should consist of cement based grouts. The proposed grouting procedures and calculations of quantities are based on commercially available, normally used grout compositions. A grout composition provided by SKB was used for calculation of grout quantities.

One further premise used when assuming grout procedures was that conventional drill-and-blast techniques would be used for excavation of the underground facility except for the elevator shaft and the ventilation shafts, which are planned to be excavated by raise boring techniques.

In order to achieve the sealing levels stipulated in Table 8-2, the difficulty in grouting according to /Eriksson and Stille 2005/ can be described by a value of 2–3 on a scale of 3 (in which 3 represents the most difficult grouting conditions). For the most difficult grouting condition /Eriksson and Stille 2005/ suggest that grouts and procedures must be selected carefully based on actual site conditions. Based on the proposal given by /Eriksson and Stille 2005/, this implies that the grouting must be performed as efficient as possible, particularly if a sealing level of 1×10^{-9} m/s should be achieved. With respect to the minimum aperture requiring to be grouted, it is in the present design step assumed that all fractures having an aperture > 50 µm are to be grouted. Fractures of smaller aperture are not likely to be grouted successfully with cement-based grouts. This assumption is based on the present state of knowledge and experience (see for example /Eriksson and Stille 2005/ and /Emmelin et al. 2004/. Larger fractures, which are more easily grouted, are based on "preliminary data" and considered to be present to a significant extent only in the sub-horizontal zones and possibly in the zone ZFMNE0060.

Based on chemical analysis of groundwater (see Section 8.2.3) it can be concluded that sulphate-resisting cement is not necessary.

8.3.2 Grouting in tunnels and caverns at repository depth

Grout in tunnels and caverns at repository depth have been assumed to be required for the following parts of the facility:

- Deposition tunnels.
- Main tunnels.
- Transport tunnels in deposition area.
- Rock caverns, transport tunnels, installation tunnel, pedestrian tunnel and skip tunnel in the central area.

Grouting during construction of tunnels and caverns is assumed to be executed as selective grouting, which means that grouting is not continuously executed along the tunnel. Depending on water inflows or other observations in probe holes drilled ahead of the tunnel face spot grouting is carried out approximately after every third round of blasting. It is proposed that the probe holes should be drilled inside the tunnel face in order to minimise the volume of holes outside the tunnel section.

In all tunnels grout holes are drilled around the tunnel section. The grout injection holes are about 20 m long with a diameter of 50–60 mm, and will be deviated at a distance corresponding to 4–5 m from the tunnel periphery at the end of the boreholes. In order to seal fine fractures, a small distance of about 1–2 m between the ends of grout holes is required (see /Eriksson 2002/).

It is further assumed that the grouting is executed in a single grout injection round when aiming to achieve a sealing level of 1×10^{-7} m/s. For sealing levels corresponding to lower hydraulic conductivities two grout injection rounds are assumed. Additional injection rounds may be necessary, but this is anticipated to be required only in the sub-horizontal near-surface zones. Figure 8-2 illustrates the principles of a grouting fan.

The grout composition is to be designed with the objective of sealing fine fractures (small apertures). Grouts with good penetration ability are therefore recommended. The penetration ability can be measured with various testing devices (see /Eriksson and Stille 2005/).

Table 8-3 indicates possible grout compositions in terms of the water/cement ratio for different sealing levels, and for different parts of the rock mass. The water/cement ratios are based on experience from testing of grouts performed at various other projects. More detailed analysis of suitable grouts including grout testing must be carried out in later design steps, and the design should also be revised if needed during the tunnel excavation. For the present design step the water/cement ratio is assumed to be sufficient to describe the grout. Water/cement ratios are also important input data for the calculation of grout quantities. Both fine grained cements (micro cements) and coarser grouting cements should be used.

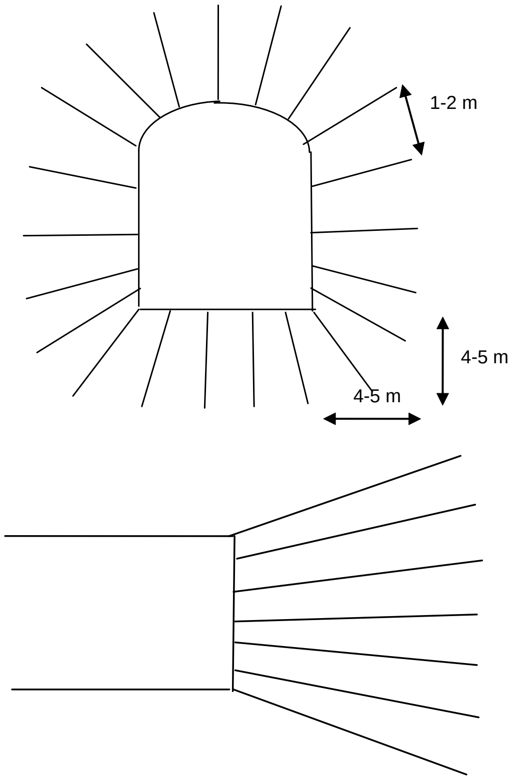


__Figure 8-2.__ Principles of boreholes in a grouting fan in a tunnel. Cross-section (top), profile (lower).

Table 8-3. Assumed grout composition, with respect to the water/cement ratio, for tunnels at repository level. Grouts are indicated for different sealing levels and parts of the rock mass. Wcr = water/cement ratio.

Part of the rock mass	Sealing level, 1×10^{-7} m/s	Sealing level, 1×10^{-9} m/s
Deformation zones excluding ZFMNE0060		Grouting rounds 1 & 2: Wcr = 2.0
ZFMNE0060	Wcr = 1.0	Grouting round 1: Wcr = 1.0
		Grouting round 2: Wcr = 2.0
Rock mass with stochastically determined fractures/ fracture zones (rock mass between deterministically determined deformation zones)		Grouting rounds 1 & 2: Wcr = 2.0

It is expected that drilling and grouting will be executed with conventional equipment, although supplementary equipment may be required if larger water inflows together with high water pressures are encountered in grout injection holes. Such supplementary equipment may include, for example, devices for mechanical installation of grouting packers and facilities to prevent packers from being forced out of grout holes.

The maximum grouting pressure should be in the order of 20 bars above groundwater pressure.

8.3.3 Grouting in ramp

Grouting in the ramp down to the repository depth is required mainly when passing the sub-horizontal near-surface zones, which will intersect the ramp between 0 and 200 m below ground surface. Grouting in other parts of the ramp will be executed in accordance with the same principles as described for the tunnels at repository depth. A proposed procedure for grouting of the sub-horizontal zones is described in the following.

Planning and execution of the probe drilling is an important aspect in excavating the ramp. The number, length and orientation of probe holes must be chosen with great care in order to enable early identification of water-bearing zones.

Grouting may be executed in accordance with the principles illustrated in Figure 8-3. It should be noted that the orientation of the zone affects the geometry of the fan of grouting holes. It may also be necessary to drill grout holes in the tunnel face.

Experiences from grouting water-bearing zones are among others described in /Chang et al. 2005/. Large water inflows together with high water pressures may result in difficulties in drilling, installation of packers and grouting. Working conditions may also be unacceptable. However, with careful planning and suitable procedures the conclusion from /Chang et al. 2005/ is that drilling and grouting can be executed successfully.

Since large water flows are to be expected in the permeable sub-horizontal zones, it is assumed that drilling and grouting must be executed with the assistance of Blow-Out-Preventers (BOP) to enable control of the water flow from the grout injection holes. Procedures for drilling and grouting with BOPs in highly permeable zones are described in /Chang et al. 2005/ (see Figure 8-4).

According to /Chang et al. 2005/ the system for grouting described in Figure 8-4 has the following advantages:

- Problems associated with high water pressures during installation of traditional packers are eliminated, as are packer sliding problems due to high grouting pressures.
- Problems of large water inflows from boreholes can be controlled more easily during the grouting operations.
- Clogging of the grouting equipment can be prevented by opening the T-valve and discharging a small quantity of grout onto the tunnel floor in order to introduce fresh material into the system.

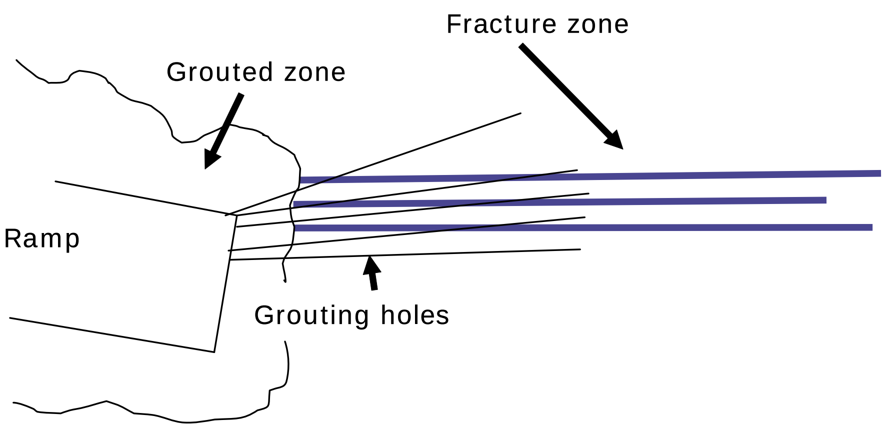


***Figure 8-3.** Example of principles for the grouting of sub-horizontal near-surface zones intersected by the ramp. The length and orientation of grout holes will be chosen with respect to the orientation between the ramp and the zone.*

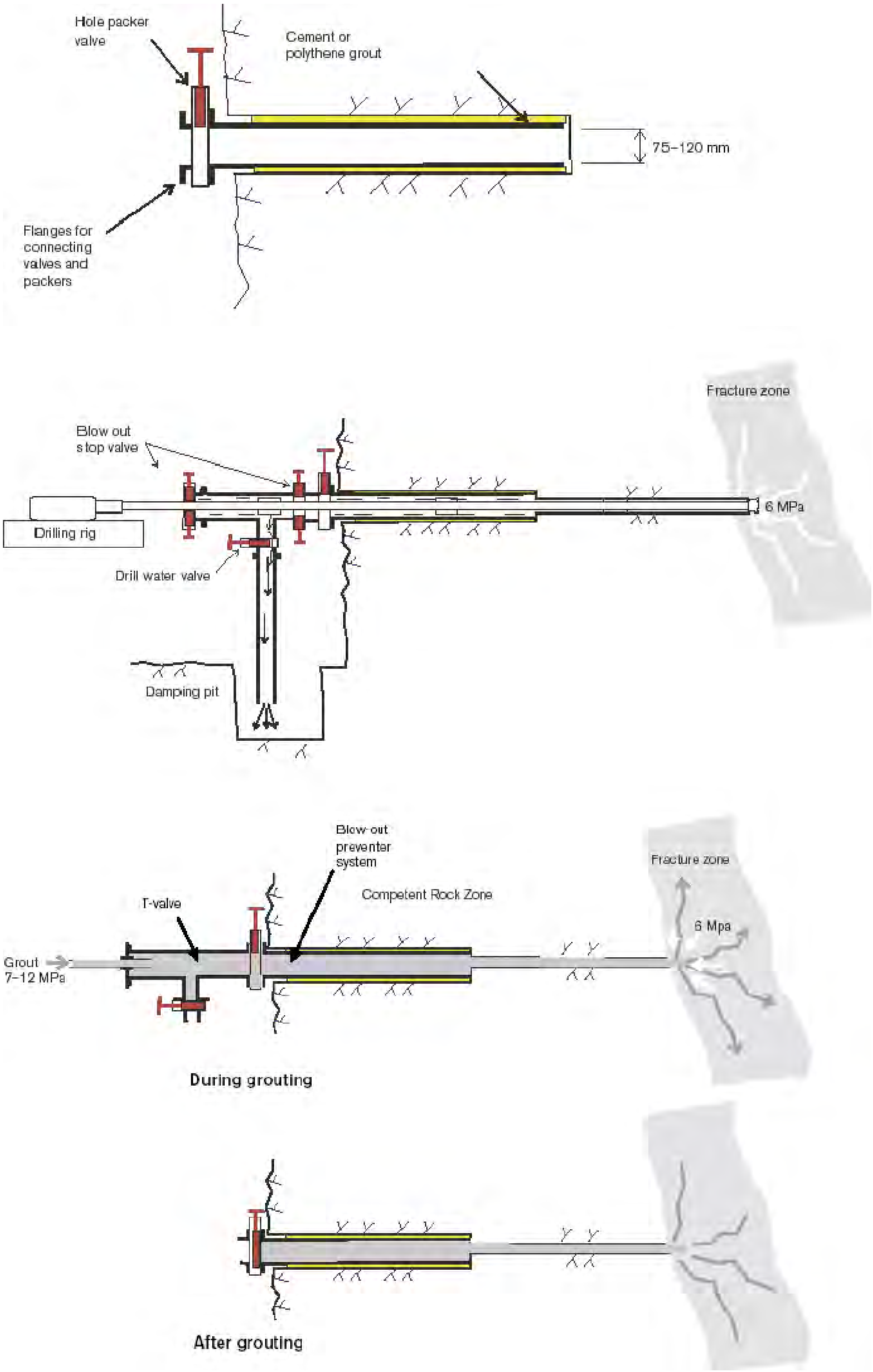


Figure 8-4. *Principles for drilling and grouting using Blow-Out-Preventer systems (from /Chang et al. 2005/). It should be noted that water and grout pressures stated in the figure relate to the drilling and grouting regime described in /Chang et al. 2005/.*

The grouting system including Blow-Out-Preventers is also suitable for the case of a potential large water inflow, since high demands for working safety could be fulfilled.

As mentioned previously, the length of grout injection holes must be chosen with respect to the orientation and width of the zone. In the present design step the holes are estimated to be about 30 m long with a diameter of 50–60 mm diameter, and to deviate corresponding to a distance of 6 m from the tunnel periphery in the end of the holes. Depending on the properties of the zone and the grouting required, grouting will be executed in one or more grout injection rounds. For the forthcoming analyses two grouting rounds are assumed.

Different types of grout will be required depending on the properties of the zones. For example, grouts with good penetration ability must be used if the larger fractures are filled with sediments. Owing to the high hydraulic conductivities (see Table 8-1) and experiences of grouting works in borehole KFM01A (see /Claesson and Nilsson 2004/), it is considered that the first injection round must be executed with a relatively low-viscosity grout corresponding to a water/cement ratio of about 0.5. Table 8-4 indicates possible grout consistencies with respect to water/cement ratio for different sealing levels. The assumed water/cement ratios are based on experience from testing of grouts at different projects. More detailed analysis of suitable grouts, including testing of grouts, must be undertaken in later design steps, and the design should be revised if needed during the tunnel excavation. In the present design step the water/cement ratios are assumed to be sufficient to describe the proposed grout. Water/cement ratios also constitute important input data for estimation of grout quantities. Both fine grained cements (micro cements) and coarser grouting cements should be considered.

Table 8-4. Assumed grout consistencies, with respect to water/cement ratios, for grouting of sub-horizontal zones in the ramp. Grouts are indicated for different sealing levels. Wcr = water/cement ratio.

Sealing level, 1×10^{-7} m/s	Sealing level, 1×10^{-8} m/s
Grouting round 1: Wcr 0.5, accelerating additives may be necessary (grout with no bleed)	Grouting round 1: Wcr 0.5, accelerating additives may be necessary (grout with no bleed)
Grouting round 2: Wcr 1.0	Grouting round 2: Wcr 2.0

For zone depths down to 200 m, comprising the major part of the grouting works, the grout pressure is set at twice the groundwater pressure in order to minimise the risk of grout being flushed out. For an average depth of the zones of 100 m, grouting pressures of 20 bars (10 bars in excess of groundwater pressure) or more should be used.

8.3.4 Grouting in skip shaft

Grouting in the shafts is required mainly when passing the sub-horizontal near-surface zones, which will intersect the shafts between 0 and 200 m below ground surface. Grouting in other parts of the shafts will be executed in accordance with the same principles described for the tunnels at repository depth. Procedures for grouting the sub-horizontal zones are described in the following.

In the skip shaft, which is to be excavated by shaft sinking techniques, the drilling and grouting will be executed according to the same principles as described for grouting in the ramp. There is however the advantage that the shaft intersects the fracture zones at a favourable angle, such that a more conventional grout fan can be drilled ahead of the face. The principles for grouting of the skip shaft are illustrated in Figure 8-5.

Since large water flows are expected in the permeable sub-horizontal zones, it is assumed that drilling and grouting must be executed with the assistance of Blow-Out Preventers (BOP), enabling the water flow from the grout injection holes to be controlled. Procedures for drilling and grouting with BOPs in highly permeable zones are described in /Chang et al. 2005/ (also see Section 8.3.4, Figure 8-4).

The grout injection holes are assumed to be about 20 m long with a diameter of 50–60 mm diameter, and to deviate corresponding to a distance of about 2 m from the shaft periphery at the end of the holes. Depending on the properties of the zone and the grouting required, grouting will be executed in one or more grout injection rounds. For the forthcoming analyses two grouting rounds are assumed.

Different types of grout will be required depending on the properties of the zones. For example, grouts with high penetration ability must be used if the larger fractures are filled with sediments. Owing to the high values of hydraulic conductivity (see Table 8-1) and experiences of grouting works in borehole KFM01A (see /Claesson and Nilsson 2004/), it is estimated that the first injection round must be executed with a relatively low-viscosity grout, corresponding to a water/cement ratio of about 0.5.

Assumed possible grout compositions are given in Table 8-4. The assumed water cement/ratios are based on experience from testing of grouts at different projects. More detailed analysis of suitable grouts including testing of grouts must be undertaken in later design steps. In the present design step the water/cement ratios are assumed to be sufficient to describe the grout. Water/cement ratios also constitute important input data for estimation of grout quantities. Both fine grained cements (micro cements) and coarser grouting cements should be considered.

For zone depths down to 200 m, comprising the major part of the grouting works, the grout injection pressure is set to twice the groundwater pressure in order to minimise the risk of grout being flushed out. For an average depth of the zones of 100 m, grouting pressures of 20 bars (10 bars in excess of groundwater pressure) or more must be used.

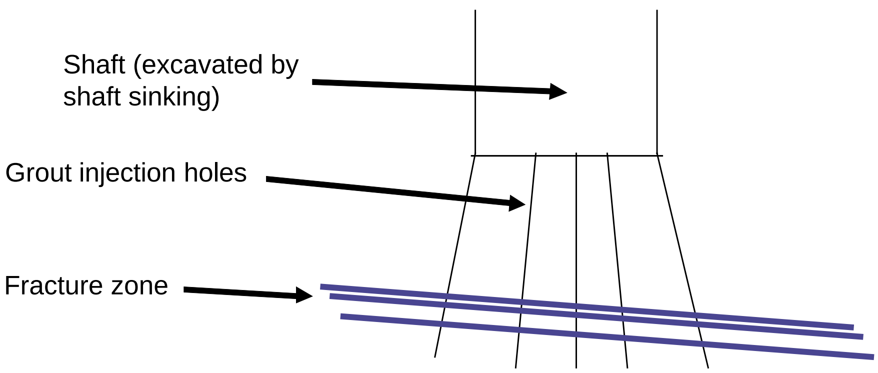


Figure 8-5. *Principles for grouting sub-horizontal zones in the skip shaft. The grout holes deviate corresponding to a distance of about 2 m from the tunnel periphery at the end of the holes.*

8.3.5 Grouting in elevator- and ventilation shafts

Grouting in the elevator- and ventilation shafts is required mainly when passing the sub-horizontal near-surface zones, which will intersect the shaft between 0 m and 200 m below ground surface.

In shafts excavated by raise boring it may be difficult to grout in the pilot hole during the drilling operation. Experiences from grouting works indicate that it may be difficult to accomplish adequate grout spread and sealing effect around the shafts. There is also some risk of losing packers or hoses or grout pipes becoming stuck in the holes when grouting is executed. It is therefore assumed that a more robust procedure will be to drill and grout in vertical holes around the periphery of the shafts prior to the raise boring.

Drilling and grouting of deep vertical holes may be difficult if not carefully planned and executed. Requirements must be specified with regard to drilling equipment and borehole deviation tolerances. Drilling may be carried out by down-the-hole equipment or core drilling techniques. The composition of the grout must be such that grout bleed or dilution does not occur. Few experiences from this type of grouting have been found in literature for this specific application. However, some experience from grouting works in the Forsmark area indicates that it should be possible to grout water-bearing near-surface zones. It should be noted that the grouting works in the Forsmark area were undertaken with the main purpose of sealing the annulus between the borehole and the casing pipes. When performing this grouting, the fracture zones were grouted at the same time. The grout was typically a cement-based grout with a water/cement ratio of 0.5. The grouting work is reported in /Claesson and Nilsson 2004/, and no specific problems are described. Experiences from drilling and grouting in deep vertical holes may also be found in the oil industry.

In summary, drilling and grouting is assumed to be possible with no major problems down to a depth of about 200 m. Further studies should however be made in order to complete the final design of the procedure. Figure 8-6 indicates the principles for grouting of the elevator- and ventilation shafts.

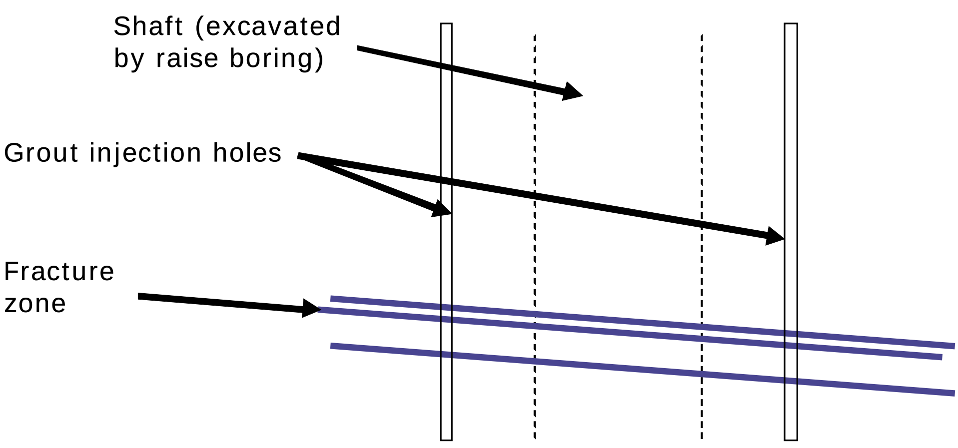


Figure 8-6. *Principles for grouting of sub-horizontal zones in the elevator – and ventilation shafts.*

The grout holes are drilled vertically around the planned location of the shafts. Depending on the location of the zones, the boreholes will be between 50 and 200 m deep. The diameter of the bore holes depends on the drilling method chosen. Depending on the properties of the zone, and the sealing level to be achieved, the grouting will be executed in one or more injection rounds. Two rounds are assumed for the forthcoming analyses. The number of holes in each grouting round is assumed to be four for the sealing level 1×10^{-7} m/s, and eight for the sealing level 1×10^{-8} m/s.

Different types of grout will be required depending on the properties of the zones. Owing to the high hydraulic conductivities (see Table 8-1) and experience of grouting works in borehole KFM01A (see /Claesson and Nilsson 2004/), it is considered that the first grouting round must be executed with a relatively low-viscosity grout, corresponding to a water/cement ratio of about 0.5. Assumed grout compositions are given in Table 8-4. The assumed water/cement ratios are based on experience from testing of grouts in different projects. More detailed analysis of suitable grouts, possibly including testing of grouts, must be undertaken in later design steps. In the present design step the water/cement ratios are assumed to be sufficient to describe the grout. Water/cement ratios also constitute important input data for estimation of grout quantities.

Due to the long boreholes, grout stabilising additives will probably be required since grout bleed should be avoided in the boreholes. Grouts resistant to bleed can be produced by addition of bentonite or silica. Grouts with good penetration ability must be used if the larger fractures are filled with sediments.

The same principles of grouting in vertical long bore holes could also be used for pre-stabilisation/sealing in advance of the high permeable zones crossed by the inclined ramp (see Section 8.3.3).

The grout is injected through a packer inserted into the grout hole at the location of the zones. In order to treat individual zones the grout may be injected by means of double packers of suitable size or with a single packer from the bottom of the injection hole and upwards.

For zone depths down to 200 m, comprising the major part of the grouting works, the grout injection pressure is set at twice the groundwater pressure in order to minimise the risk of grout being flushed out. For an average depth of the zones of 100 m, grouting pressures of 20 bars (10 bars in excess of groundwater pressure) or more must be used.

When using grout stabilising additives, it can be difficult to compose a grout with the best possible penetration properties. It may thus be reasonable to assume that there is some risk of achieving a reduced sealing effect. A suitable procedure for post-grouting in the shafts should therefore be devised if supplementary sealing is required. Such post-grouting may require equipment and other devices to be developed for grouting within cramped spaces.

8.3.6 Execution and control of grouting works

One success factor when grouting fine fractures in order to meet demanding sealing levels is that grouting should not be stopped when excessive grout takes are encountered /Eriksson 2002/. Criteria for terminating grout injection should be analysed further in later design steps.

Following grouting, control holes should be drilled between the grout holes to enable evaluation of the efficiency of grouting by means of water loss measurements, other hydraulic tests or water inflow measurements. The results of these tests will determine the need for further grouting. Decisions regarding changes of the grouting procedure will be made based on the results of the grouting work undertaken and/or measurement or observation of water inflows into the tunnels.

When grouting in fractured zones, differences in grout take should be expected between different grout injection holes. In order to achieve good grouting result the grout holes should be grouted individually with one pump (only holes with hydraulic communication should be grouted at the same time from the same pump). It must also be considered that control of grout spread and volumes injected will be required when grouting large fractures. Control of grout spread can be made by observations in the tunnel and the volume is controlled by criteria for terminating the grouting in a grout hole. Such criteria should be developed in later design steps.

8.3.7 Summary of grouting procedures

In Table 8-5 the proposed grouting procedures for the hard rock facility are summarized based on the descriptions of grouting procedure for the respective parts of the facility. The different procedures are designated as grouting classes. In Table 8-6 the grouting classes for the different parts of the facility are described. It should be noted that more grout injection rounds may be required than indicated in Table 8-5. The number of rounds indicated should be interpreted only as an estimation of the minimum number of rounds required.

Table 8-5. Description of grouting classes.

Grouting class	Description of grouting procedure
1	Grouting of single water-bearing fractures/minor fracture zones (stochastically determined fractures/fracture zones) and deterministically determined deformation zones except ZFMNE0060. Used for sealing level 1×10^{-9} m/s. Two grout injection rounds. Water/cement ratio Wcr = 2.0.
2	Grouting of the deformation zone ZFMNE0060 for sealing level 1×10^{-7} m/s. One grout injection round. Water/cement ratio Wcr = 1.0.
3	Grouting of the deformation zone ZFMNE0060 for sealing level 1×10^{-9} m/s. Two grout injection rounds. Water/cement ratio Wcr 1.0 and 2.0 respectively.
4	Grouting of sub-horizontal near-surface zones for sealing level 1×10^{-7} m/s. Two grout injection rounds. Water/cement ratio Wcr 0.5 and 1.0 respectively.
5	Grouting of sub-horizontal near-surface zones for sealing level 1×10^{-8} m/s. Two grout injection rounds. Water/cement ratio Wcr 0.5 and 2.0 respectively.
6	Grouting of sub-horizontal near-surface zones from ground surface, sealing level 1×10^{-7} m/s. Two grout injection rounds. Water/cement ratio Wcr 0.5 and 1.0 respectively.
7	Grouting of sub-horizontal near-surface zones from ground surface, sealing level 1×10^{-8} m/s. Two grout injection rounds. Water/cement ratio Wcr 0.5 and 2.0 respectively.

Table 8-6. Grouting classes for the various parts of the facility.

Part of the facility	Part of rock mass	Grouting class
Deposition tunnels	Rock mass with stochastically determined fractures/fracture zones and deterministically determined deformation zones.	1
Main tunnels	Rock mass with stochastically determined fractures/ fracture zones.	1
Transport tunnels in deposition area	Rock mass with stochastically determined fractures/ fracture zones and deterministically determined deformation zones except ZFMNE0060.	1
Transport tunnels in deposition area	Deformation zone ZFMNE0060	2 or 3
Rock caverns in central area	Rock mass with stochastically determined fractures/fracture zones.	1
Transport tunnels in central area	Rock mass with stochastically determined fractures/fracture zones.	1
Installation tunnel and pedestrian tunnel in central area	Rock mass with stochastically fractures/ fracture zones.	1
Skip tunnel	Rock mass with stochastically determined fractures/fracture zones.	1
Skip shaft	Rock mass with stochastically determined fractures/fracture zones.	1
Skip shaft	Sub-horizontal near-surface zones (depth 0–200 m)	4 or 5
Elevator- and ventilation shafts	Sub-horizontal near-surface zones (depth 0–200 m)	6 or 7
Ramp (access tunnel)	Rock mass with stochastically determined fractures/fracture zones.	1
Ramp (access tunnel)	Sub-horizontal near-surface zones (depth 0–200 m)	4 or 5

8.4 Estimation of quantity of grout

8.4.1 General

The calculation of the quantity of grout take was assessed for the sealing levels and points in time according to Sections 8.2.1 and 8.2.2, and for the following parts of the deep repository:

- Deposition tunnels.
- Main tunnels.
- Transport tunnels in deposition area.
- Rock caverns, transport tunnels, installation tunnel, pedestrian tunnel and skip tunnel in the central area.
- Ramp.
- Shafts.

8.4.2 Input data and assumptions

Calculation of grout volume

The calculation of quantities is based on volumes in the single grout holes (excluding volume of the grout hole) assumed according to /SKB 2004a/, including Chapter 4.6 in Appendix 2. In /SKB 2004a/ it is stated that the volume (grout take) in a grout injection holes should be estimated using the following analytical methods:

1. Calculation of grouted volume in a single grout hole based on the assumption that grout spreads in plane parallel fractures (Equation 8-1).
2. Calculation of grouted volume in a single grout hole with the assumption that the porosity in the rock mass is filled by grout to a distance, which correspond the estimated penetration length from the grout hole (Equations 8-2 and 8-3).

$$V = \left(\frac{\Delta p}{2 \cdot \tau_0} \right)^2 \cdot \frac{12 \cdot K_b \cdot L \cdot \mu_w \cdot \pi}{\rho_w \cdot g} \qquad \text{Equation 8-1}$$

where

V = grouted volume (m^3)

Δp = grouting pressure (over ground water pressure) (Pa)

τ_0 = yield value of the grout (Pa)

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

L = length of grout hole (m)

ρ_w = specific weight of water (kg/m^3)

μ_w = viscosity of water (Pas)

g = specific gravity (m/s^2)

It should be noted that Equation 8-1 is associated to a number of uncertainties, since it does not consider some of the factors affecting the grout spread in the rock mass. These factors are for example the variations in aperture of the fractures, limitations in the penetration ability of the grout due to filtration, hardening and bleed of the grout, and finally the criteria normally used for ending the pumping of grout. In /Eriksson 2002/ these factors are studied with respect to the effect on the calculation result, and the conclusion is that the volume may vary with a factor between 1 and 100. How much the volume is affected depends both on the properties of the grout and the fractures. Especially when the apertures are small the effects of different factors are evident. Calculated volumes should thus be used with caution.

Due to the uncertainties in the volumes calculated with Equation 8-1, the volumes used for the estimation of grout quantity were assessed by dividing the calculated volumes with a factor of 2–50. Assumed values are based on calculations presented in /Eriksson 2002/ (see section above). Based on the calculations in /Eriksson 2002/ the factors are given values depending on the type of fractures that will be grouted, where grout volumes in fine fractures are assumed to be associated with the largest uncertainty.

It should also be noted that Equation 8-1 only is valid for single grout holes. When grouting of several grout holes in a grout fan, the conductivity of the rock mass will more or less be reduced to the sealing effect of the grout. Thus, it is not obvious that the volume in a grout fan can be calculated by multiplying the volume in one grout hole with the number of holes.

The grout volume in a grout hole was also calculated, based on the assumption that the porosity in rock mass is filled by grout to a distance, which corresponds to the estimated penetration length from the grout hole. The grout volume is in this case calculated with Equation 8-2.

$$V = I^2 \cdot \pi \cdot p \cdot L \qquad \text{Equation 8-2}$$

where

V = grouted volume (m^3)

I = penetration length/grout spreading distance (m)

p = porosity of rock mass

L = length of grout hole (m)

The penetration length has been assessed from engineering judgement, and the porosity can according to /SKB 2004a/ be calculated with Equation 8-3.

$$\log p = 0{,}17 \cdot \log K_b - 1{,}7 \pm 0{,}3 \qquad \text{Equation 8-3}$$

where

p = porosity

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

Table 8-7 and Table 8-8 show the grout volumes which are used as a basis for the estimation of total quantities for the facility parts.

Table 8-7. Grout volume for a single grout hole in a grout fan. The volumes are used in the estimation of quantities for the different facility parts, grouting classes 1-3.

Grouting class/ grouting round	Volume of grout, min (m^3)	Volume of grout, max (m^3)
1 / round 1 and 2	0.05	0.25
3 / round 2		
2 / and 3 / round 1	0.1	1.0

Table 8-8. Grout volumes estimated for grouting of a subhorizontal near surface zone (grouting classes 4–7). The volume is used for all sealing levels.

Facility part	Volume of grout, min (m^3)	Volume of grout, max (m^3)
Ramp (access tunnel)	4	50
Shaft (for each shaft)	2	10

Experiences from grouting works

Experiences from groutings executed in a low conductive rock mass indicate that many holes only takes a volume corresponding to the volume of the grout hole and that the volume in groutable holes is in the order of some hundred litres. Larger grout volumes are only experienced in few grout holes in a fan.

Comparing volumes based on experience and volumes presented in Table 8-7 it may thus be concluded that the maximum volumes probably are overestimated, if these volumes are to be used for all holes in a grout fan. Some holes may consume the maximum volumes given in Table 8-7, but it is not judged likely to have this grout take in all grout holes in a grout fan.

Regarding the grout take in the subhorizontal zones comparisons can be made with grouting works at Äspö HRL. At Äspö HRL two water bearing zones at the depth of about 200 m were grouted and the grout take was about 50–70 m^3 in each zone /Janson 1996/. These volumes are larger than those given in Table 8-8, but it should be noted that the zones at Äspö HRL was about 50 m wide, which is wider than the subhorizontal zones found in Forsmark. Experiences in general are that large grout takes (several hundred litres up to several thousands of litres in a single grout hole) normally are only encountered in fracture zones. It is therefore judged likely that the volumes presented in Table 8-8 reflect the grout take in major fracture zones.

Estimation of quantity of grout for facility parts

The estimated quantities for the individual facility parts is based on the grouting classes given in Table 8-5 and grout volumes given in Table 8-7 and Table 8-8. In addition, the estimation is based on the following assumptions:

- In order to take into account that all of the grout holes in a grout fan are not groutable, it has been assumed that 40–70% of the grout holes only are filled with a volume corresponding to the volume of the hole. The grout for filling the holes is assumed to have a water cement ratio, wcr, of 0.5.
- Since grouting mainly is assumed to be needed in the deterministically deformation zones and the near surface zones, together with the fact that the differences in layouts (see Chapter 5) are relatively small, the estimation of grout quantities is only executed for one depth, 400 m.
- Based on the distribution of conductivity values for the rock between deformation zones it is assumed that only 2% of the rock mass between the deformation zones will be groutable. These parts of the rock mass are assumed to correspond to stochastically determined fractures/fracture zones. The value 2% can be derived from the distribution of the hydraulic conductivity given in Table 8-1 (see also Figure 7-2), and the assumption that the hydraulic conductivity of the rock mass, K, should be higher than 1×10^{-9} m/s in order to make grouting possible.
- Possible control holes are not included in the calculation of quantities.
- Since the grout holes are drilled outside the periphery of the tunnel, the total grout quantity is also assumed to be located outside the tunnel.

The geometries for different facility parts are based on the cross sections given in section 4.2 and the length of different tunnels presented in Chapter 5.

The compositions of grout are based on proposals of grouts for different grouting classes (see Table 8-5). Another composition of grout has also been proposed by SKB (see Table 8-9). It should be noted that SKB is previously working with the development of grouts, which have a pH < 11 (so called low pH-grouts). No such compositions are yet available, which means that the compositions given in Table 8-9 should be regarded as preliminary.

Table 8-9. Preliminary composition of "low pH-grout" (given by SKB).

Material	Quantity kg/m^3
Cement (Ultrafin 16)	299
Silica slurry (Grout aid)	419
Plasticizer (SP40)	11
Water	599

According to SKB the "low pH-grout" in Table 8-9 should be used as an alternative grout. The estimation of quantities for the "low pH-grout" has been made in such a way that the volumes calculated for the grouts proposed by the design team (see Table 8-3 and Table 8-4) also have been used to estimate the volume for the "low pH-grout". Finally, the quantities of different materials have been calculated and presented based on the compositions of grout proposed by the design team as well as on the composition given in Table 8-9.

8.4.3 Results

In Table 8-10 the estimated quantity of grout for different facility parts are presented for the sealing levels and points in time according to Sections 8.2.1 and 8.2.2. A more detailed description of the estimation of grout quantity together with the quantity of other items such as grout holes, grout for filling of grout holes, cement and additives is presented in Appendix D.

Table 8-10. Quantity of grout given as m^3 excluding hole filling for different facility parts, sealing levels and points in time. For sealing level 10^{-9} m/s it is assumed that sub horizontal zones will have a sealing level of 10^{-8} m/s. Empty fields indicate that no grouting is proposed or that the grouting need is very small.

Facility part	Sealing level 10^{-9} m/s and early point in time		Sealing level 10^{-9} m/s and late point in time		Sealing level 10^{-7} m/s and early point in time		Sealing level 10^{-7} m/s and late point in time	
	Min	Max	Min	Max	Min	Max	Min	Max
Deposition tunnels	14.6	78	146	784				
Main tunnels	0.7	3.5	7	35				
Transport tunnels in deposition area	0.5	5.1	5.3	51				
Ramp	1	8.3	10.7	83.3	0.4	5	4	50
Tunnels and rock caverns in central area	0.2	1.1	2.3	10.5				
Shafts	1.2	12.3	12.6	123.2	1.2	12	12	120
Total	18	108	184	1,087	2	17	16	170

8.5 Conclusions

The following conclusions can be made with regard to grouting in the hard rock facility.

In order to achieve a sealing level corresponding to a hydraulic conductivity of 1×10^{-9} m/s, grouting will be required in all parts of the facility. Most of the grouting at the repository level will be executed in the deformation zones passed by the deposition tunnels. With these very high sealing requirements, grouting must be planned and executed with great care and proper control of the grouting process, and with grouts with high penetration ability.

In order to achieve a sealing level corresponding to a hydraulic conductivity of 1×10^{-7} m/s, grouting will basically be required in the ramp and shafts when passing through the sub-horizontal near-surface zones, which can be very permeable. Based on "preliminary data" it can also be expected that some grouting probably will be required in the deterministically determined deformation zone ZFMNE0060.

There are some uncertainties concerning which sealing levels that can be achieved. Further analyses, based for example on grouting tests in the Forsmark area, should be undertaken to reduce these uncertainties. Possible sealing levels are closely related to the properties of the sub-horizontal near-surface zones, which should be studied further.

More detailed analyses of suitable grouting procedures must be carried out in later design steps, and the design should be revised if needed during the tunnel excavation. Demands regarding sealing levels for the various parts of the facility also need to be specified further.

The grout volumes estimated by means of the equation for calculating the volume in one single grout hole (Equation 8-1) includes significant uncertainties attributable to the simplified descriptions of the grout, fractures in the rock mass, and the execution of grouting. It is essential to apply engineering judgement when estimating the volumes of grout by means of this equation, the use of which is obligatory according to /SKB 2004a/. Such engineering judgement should be based on experience of grouting works executed under similar conditions. The use of numerical calculation tools more closely resembling the actual conditions should also be considered. One such numerical method is described in /Eriksson 2002/.

9 Assessment of rock support need

9.1 Introduction

Assessment of the need for rock support and estimation of quantities for rock support elements were executed in order to constitute a basis for analysis of harmful elements in the groundwater composition and as a basis for cost calculations.

In design step D1 the assessment of rock support need was carried out for all underground openings in the layout according to Chapter 5, and included the following two steps:

1. Assessment of suitable rock support systems.
2. Estimation of quantities for all rock support elements.

In design step D1 the need for rock support was assessed by empirical methods, based on, for example, the Q-system /Barton 2002/. Requirements on verification of the suggested rock support measures will be specified in later design steps.

9.2 Input data and assumptions

9.2.1 General

The following input data were used for execution of the specific design task:

- Rock stresses in tunnels (Chapter 4.4).
- Layouts for the hard rock facility at repository level (Chapter 5).
- Properties of deformation zones and the problems involved in passing through the deformation zones during construction (Chapter 6).

The rock support need was assessed for the parts of the facility presented in the layouts in Chapter 5. The rock support is also based on the requirements with regard to construction, durability, operation, and maintenance described in Sections 4.4–4.6 in /SKB 2004a/.

It is, in addition, assumed that the rock support measures will mainly involve conventional rock support techniques such as rock bolts, shotcrete, and steel mesh. According to /SKB 2004a/ shotcrete should be avoided in deposition tunnels.

9.2.2 Description of the rock mass between deformation zones

The Preliminary site description version 1.2 (Table 6-4 in /SKB 2005a/) provides Q-values for the rock mass between deformation zones. These Q-values are based on logging of rock cores from boreholes KFM01A, KFM02A, KFM03A and KFM04A, and apply to an SRF value of 1.0 and to a joint water reduction factor, J_w , of 1.0. For the rock mass between the deformation zones the corresponding Q-values are approximately between 40 and 100.

Based on this classification, the need for rock support is based on the assumption that the Q-value for the rock mass between the deterministically determined deformation zones is 40–100 ("very good"). It should be noted, however, that the variation is considered to be large. Consequently, both better and poorer rock quality will probably occur.

Rock spalling in the tunnels is not likely to be a major stability problem (see Section 4.4). Variations in stress levels are probable, however, as is a lower rock mass strength, such that local spalling may occur. These local problems may arise for example at intersections between main and deposition tunnels. Supplementary support may be necessary, for example using steel mesh reinforced shotcrete, which is anchored with rock bolts.

Spalling may also occur in the lower parts of the shafts due to their circular geometry and the in-situ stress field. Based on the results of analyses of spalling in deposition holes /Martin 2005/, major problems due to spalling are not expected. No special consideration has therefore been given to this potential problem during assessment of the rock support need. However, the technical risk assessment, presented in Section 10.3.1, includes an analysis of potentially higher stress levels.

Potentially unstable wedges may be present. According to /Martin 2005/ the potential wedges will however be either stable or handled by standard rock support systems. Further analyses of potentially unstable wedges were for this reason not executed in design step D1.

9.2.3 Description of the rock mass in deformation zones

The rock quality for the deformation zones passed in the layouts by different parts of the facility is described in Chapter 6 and the Q-values for the deformation zones are indicated in Table 6-6. In Chapter 6 it was also concluded, that the probability of spalling is probably less in the deformation zones than in the surrounding rock and that other stress induced stability problems are not expected (see Section 6.4.2). If spalling occurs it is expected to be encountered in the transition between the deformation zone and the surrounding rock.

The rock mass in the deformation zones is also considered to be more fractured compared to the surrounding rock. It is therefore possible that problems may occur due to outfall of loose rock. The probability of this is expected to be low due to the generally high stress levels and relatively good rock quality.

A more detailed analysis must be carried out in later design steps in order to assess potential stability problems in the deformation zones.

9.2.4 Summary of potential stability problems

Table 9-1 presents a summary of the rock mass descriptions, described as Q-values, for all layouts, including the central area.

Table 9-1. Summary of the rock mass quality, described as Q-values, for all layouts, including the central area, depth 400 and 500 m.

Part of rock mass	Q-value	Potential stability problems
Deformation zone ZFMNE0060	1–4	Outfall of wedges and rock pieces, loose rock may occur[1]
Deformation zones ZFMNE0061, ZFMNE0401, ZFMNE103A and B and ZFMNE1188	4–10	Outfall of wedges and rock pieces
Rock mass between deformation zones	40–100	Some outfall of wedges and small rock blocks, local spalling

[1] The probability of larger volumes of loose rock is considered to be low due to the generally high stress levels and relatively good rock quality.

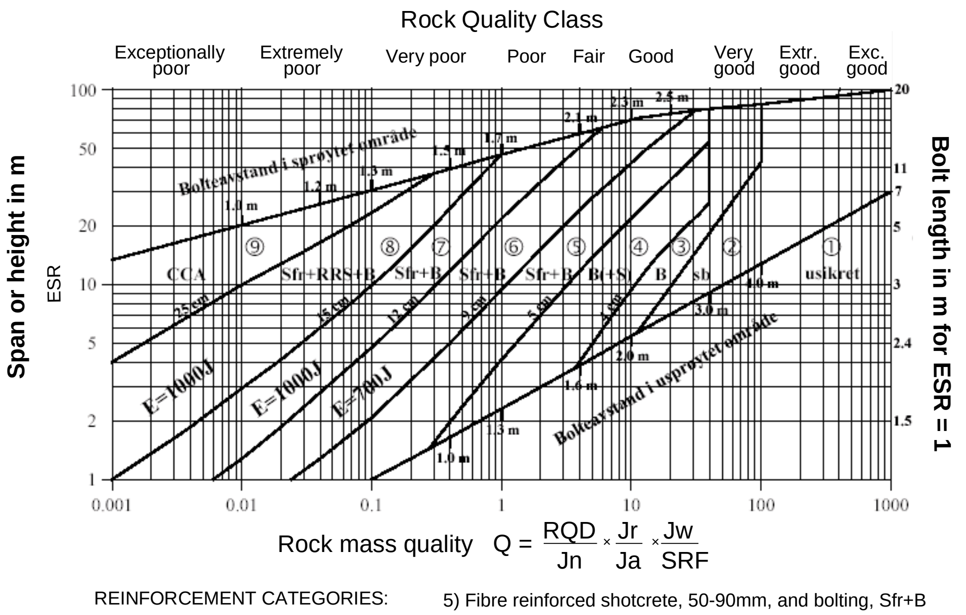
9.3 Estimation of rock support need

9.3.1 General

Rock support need was assessed in accordance with the following principles:

- The needed rock support is assessed based on a description of the rock mass in deterministic deformation zones, and the rock mass between these zones. It is assumed that stochastically determined fracture zones are of limited width, and that the rock support for these zones is included in the rock support for the surrounding rock mass. The rock mass description is based on the rock quality in terms of the Q-value in addition to a description of potential stability problems.
- The proposed rock support will be assessed with respect to stability, durability and safety for the various parts of the facility. Stability and safety issues include working safety as well as damage to tunnels, equipment, and installations. The rock support will be assessed from empirical relationships between the Q-value and rock support together with engineering judgements.

The relationship between rock quality and rock support according to the Q-system is presented in Figure 9-1. In order to estimate the rock support need based on Figure 9-1, the ESR factor (Excavation Support Ratio) is set at 1.0 for all parts of the facility. It is assumed that possible grouting of the deterministic deformation zones will not affect the Q-value or the type of rock support.



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting (with 40-100mm unreinforced shotcrete), B(+S)
5) Fibre reinforced shotcrete, 50-90mm, and bolting, Sfr+B
6) Fibre reinforced shotcrete, 90-120mm, and bolting, Sfr+B
6) Fibre reinforced shotcrete, 120-150mm, and bolting, Sfr+B
8) Fibre reinforced shotcrete, > 150mm, with reinforced ribs of shotcrete and bolting, Sfr+RRS+B
9) Cast concrete lining, CCA

E) Energy absorption in fibre reinforced shotcrete at 25mm deflection in plate test

Figure 9-1. *Rock support based on Q-value (based on figure from the home page of NGI* ***www.ngi.no****; see also /Grimstad and Barton 1993/).*

Rock support need have been assessed for the following parts of the facility:

- Deposition tunnels.
- Main tunnels.
- Transport tunnels in deposition area.
- Rock caverns.
- Transport tunnels in the central area.
- Installation tunnel and pedestrian tunnel in the central area.
- Skip tunnel.
- Shafts.
- Ramp (access tunnel).

Table 9-2 presents the span width for various parts of the facility.

Table 9-2. Span width for various parts of the facility (rounded up to the next whole metre). Tunnel type is according to /SKB 2002/.

Part of the Facility	Tunnel type	Span, m
Deposition tunnel[1]	D	5
Main tunnel	A	10
Transport tunnel in deposition area and central area	B	7
Rock caverns		12–15
Installation tunnel and pedestrian tunnel	F	3
Skip tunnel	C	8
Ramp (access tunnel)[1]	E	6
Elevator and skip shaft		6 (diameter)
Ventilation shafts		3 (diameter)[2]

[1] Modified by SKB from /SKB 2002/.

[2] Diameter ranges between 2.5 and 3.5 m.

The rock support presented in the following is valid for all proposed layouts since the information in /SKB 2005a/ does not indicate any differences in the rock quality between 400 and 500 m depth. The probability of spalling at 500 m depth may be higher, but was not considered to influence the proposed rock support systems.

9.3.2 Durability of rock support

Consideration should be given to exposure and corrosivity classes in accordance with /SKB 2004a/ when using rock support consisting of rock bolts and shotcrete. The exposure and corrosivity classes depend on the part of the facility where the rock support is to be installed, and on the chemical composition of the groundwater. Criteria for different classes are given in Table 4-2 in /SKB 2004a/.

The determination of exposure and corrosivity classes with respect to carbonatisation/chloride corrosion depends on the presence of marine environment /SKB 2004a, Table 4-2/. No definition of marine environment is provided in /SKB 2004a/. However, according to /SKB 2004a/, BV Tunnel /Banverket 2002/ is regarded as representing current best practice, and according to BV Tunnel a marine environment is considered to prevail if the groundwater is aggressive.

According to /SKB 2004a/ aggressive water is defined as water, which exhibits one or more of the following properties:

- pH < 6.5,
- hardness < 20 mg Ca/l (total hardness),
- alkalinity < 1 meq/l (1 meq/l corresponds to 62.5 mg HCO_3^- /l),
- conductivity > 100 mS/m.

The exposure class is also to be determined with regard to chemical attack, which is influenced by the chemical composition of the groundwater according to Table 2 in SS EN 206-1 /SIS 2001/.

From analysis of water samples from boreholes KFM02A, KFM03A and KFM04A (see /Wacker et al. 2004a–c/) it can be concluded that the water is not aggressive with respect to pH, hardness and alkalinity at 400 to 500 m depth. However, the samples show conductivities exceeding 100 mS/m, indicating that the water should nevertheless be classified as aggressive.

Aggressive water requires the shotcrete composition to be chosen according to class XS3 (see Table 4-2 in /SKB 2004a/). The presence of aggressive water also requires rock bolts to be selected, which fulfil the requirements given by class R3 (see Table 4-2 and 4-3 in /SKB 2004a/). Class R3 corresponds to hot-dip galvanised steel combined with surface protection of thermo set epoxy with a layer thickness of 80 µm, and grouting with cement mortar with a water/cement ratio ≤ 0.32. According to /SKB 2004a/, corrosivity class R1 may be used in deposition tunnels regardless of the environment. Class R1 corresponds to untreated steel, and grouting with cement mortar with a water/cement ratio ≤ 0.32.

According to BV Tunnel /Banverket 2002/, no covering layer of shotcrete is required over the rock bolt ends since the corrosion protection of bolts in class R3 fulfils the requirements of all corrosivity classes when exposed to air.

With respect to chemical attack, class XA1 is to be chosen, since the sulphate content in the water samples from boreholes KFM02A, KFM03A and KFM04A is 200–600 mg/l SO_4^- (see Table 2 in SS-EN 206-1 /SIS 2001/ and /Wacker et al. 2004a–c/).

No requirements are specified for steel mesh in deposition tunnels, although the requirements of corrosivity class C3 should be considered in other parts of the facility.

Table 9-3 provides a summary of the durability requirements of rock support measures. The choice of exposure class for shotcrete was based on recommendations in SS 13 70 03 (Table 5.3.2a) /SIS 2002/ with regard to the composition and properties of concrete. For exposure Class XS3 the water/cement ratio should not exceed 0.40, and the cement content should be not less than 200 kg/m³ shotcrete according to SS 13 70 03.

Table 9-3. Summary of rock support durability requirements.

Part of the facility	Corrosivity class for bolts	Exposure class for shotcrete (carbona-tisation)	Exposure class for shotcrete (chemical attack according to SS 13 70 03)	Choice of exposure class for shotcrete
Deposition tunnel	R1	No shotcrete	No shotcrete	No shotcrete
Other facility parts	R3	XS3	XA1	XS3

9.3.3 Rock support with respect to stability and safety

Since personnel, equipment and installations will be present in the facility over an extensive period, certain rock support measures should be allowed for as a safety precaution. It is therefore assumed that, despite an overall very good rock quality, a certain minimum level of rock support is to be installed in the roof of all parts of the facility, except in deposition tunnels. In design step D1 it is proposed that these parts of the facility should be supported with spot bolting and 50 mm thick steel fibre reinforced shotcrete. This level of rock support is hereafter referred to as the proposed minimum rock support. The choice of fibre reinforced shotcrete is preferred in this design step since the rock support easily could be upgraded to shotcrete with systematically installed rock bolts, without applying additional fibre reinforced shotcrete. Since deposition tunnels will be used for a relatively short time it is considered that acceptable safety levels will be achieved by spot bolting as a minimum level of rock support, provided scaling of loose rock is executed continuously. It should be noted that shotcrete is to be avoided in deposition tunnels according to /SKB 2004a/.

Steel mesh anchored with rock bolts could be used as minimum rock support as an alternative to shotcrete. Various types of plastic mesh may also be used, as mesh with suitable mechanical properties are available. It will also be more convenient to install plastic mesh compared to steel mesh owing to its lower weight. Before selecting the type of mesh the material composition, mechanical properties and fire resistance of the different types require further investigation.

Certain aspects of minimum rock support should be investigated further by SKB, since this is primarily an issue for the operator of the deep repository. The determination of minimum levels of rock support depends for example on the rock quality, the acceptable risk level with respect to safety of personnel and equipment, but also on the operation and maintenance procedures. The choice of steel fibre reinforced shotcrete should be studied further as well. The use of unreinforced shotcrete may for example yield lower rock support costs compared to fibre reinforced shotcrete.

9.3.4 Summary of rock support

Table 9-4 summarises the proposed rock support for the various parts of the facility. The tunnel roof for the tunnels in Table 9-4 corresponds to the arched shaped part of the tunnel.

Table 9-4. Summary of rock support for different parts of the facility; s = distance between bolts.

Part of the facility	Part of the rock mass	Rock support based on Q-value	Rock support selected
Deposition tunnels	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting to be installed for safety, based on geological mapping in the tunnel.
Deposition tunnels	Deformation zones (Q = 4–10)	Systematic bolting, s = 1.5–2 m	Systematic bolting at 1.5 m centres in roof and walls, with mesh if necessary to prevent outfall of rock blocks between bolts.
Main tunnels	Rock mass between deformation zones (Q = 40–100)	No support, or spot bolting	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Transport tunnels in deposition area	Rock mass between deformation zones (Q = 40–100)	No support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Transport tunnels in deposition area	Deformation zones (Q = 4–10)	Systematic bolting, s = approx. 2 m, possibly unreinforced shotcrete, 40–50 mm	Systematic bolting with washer and nut, s = 2 m, steel fibre reinforced shotcrete on walls and in roof, thickness 50 mm.
Transport tunnels in deposition area	Deformation zones (Q = 1–4)	Systematic bolting, s = 1.7–2.1 m, steel fibre reinforced shotcrete, 50–100 mm	Systematic bolting with washer and nut, s = 1.5 m, steel fibre reinforced shotcrete on walls and in roof, thickness 100 mm.
Rock caverns in central area	Rock mass between deformation zones (Q = 40–100)	Spot bolting	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Transport tunnels in central area	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Installation tunnel and pedestrian tunnel in central area	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Skip tunnel	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support)
Elevator and skip shaft	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm (minimum rock support)
Ventilation shafts	Rock mass between deformation zones (Q = 40–100)	No rock support	No rock support since the area is limited. If necessary (for example if protection of installations is needed) some form of rock support such as mesh must be installed.
Ramp (access tunnel)	Rock mass between deformation zones (Q = 40–100)	No rock support	Spot bolting and steel fibre reinforced shotcrete, thickness 50 mm, in roof (minimum rock support) Depending on width and rock mechanics properties of sub-horizontal near-surface zones, local supplementary rock support may be necessary.

Based on the selection of rock support for the various parts of the facility as indicated in Table 9-4, a compilation of rock support classes is presented in Figure 9-2, Table 9-5 and Table 9-6. The length of rock bolts are based on Figure 9-1.

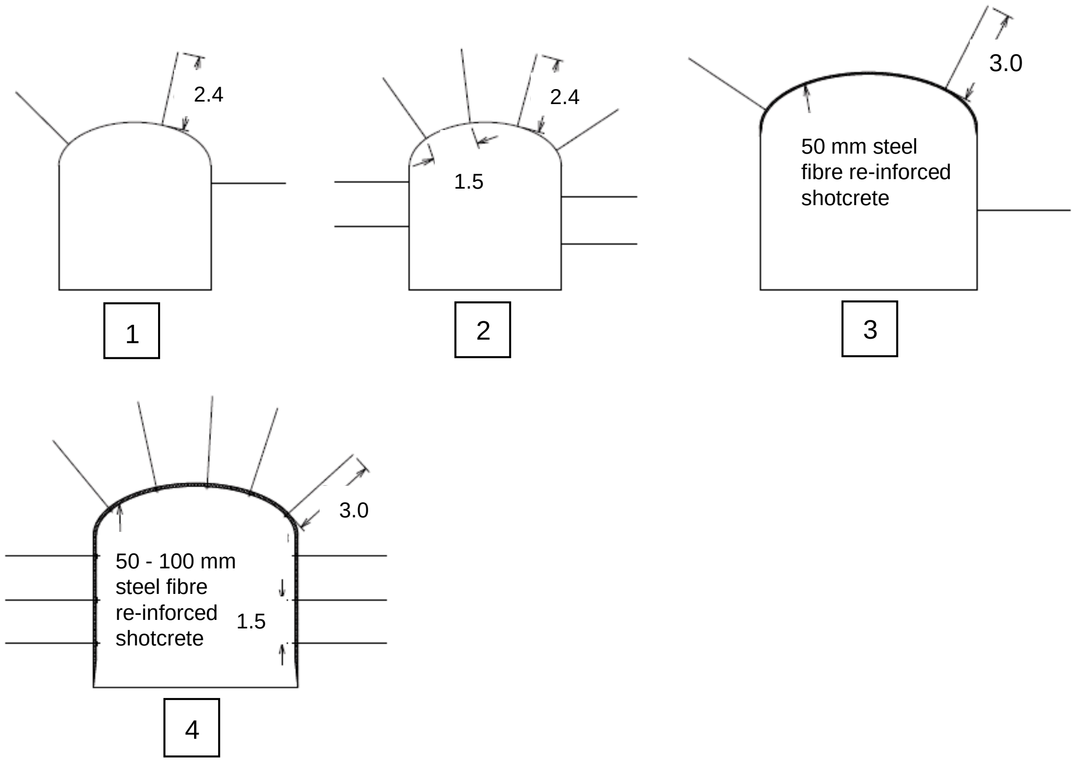


Figure 9-2. *Illustration of the application of rock support classes according to Table 9-5 and Table 9-6. The tunnel sections in the figure correspond to deposition tunnels for rock support classes 1 and 2. The application of rock support classes 3 and 4 are illustrated with tunnel sections corresponding to transport tunnels.*

Table 9-5. Description of rock support classes, s = distance between bolts.

Rock support class	Description of rock support class
1	Spot bolting, 2.4 m long, untreated steel, grouted with cement mortar
2	Systematic bolting 2.4 m long, s = 1.5 m, untreated steel, grouted with cement mortar. Mesh if necessary. Bolting on walls and in roof.
3	Spot bolting, 3 m long, galvanised/epoxy coated steel, grouted with cement mortar, steel fibre reinforced shotcrete in roof, thickness 50 mm (minimum rock support). 2.4 m long bolts used in installation tunnel, pedestrian tunnel and shafts.
4	Systematic bolting with washer and nut, 3 m long bolts, s = 1.5 m, galvanised/ epoxy coated steel, grouted with cement mortar, steel fibre reinforced shotcrete, thickness 50–100 mm. Bolting and shotcrete on walls and in roof.

Table 9-6. Rock support classes for the different parts of the facility.

Part of the facility	Type of rock mass	Rock support class
Deposition tunnels	Rock mass between deformation zones	1
Deposition tunnels	Deformation zones	2
Main tunnels	Rock mass between deformation zones	3
Transport tunnels in deposition area	Rock mass between deformation zones	3
Transport tunnels in deposition area	Deformation zones	4
Rock caverns in central area	Rock mass between deformation zones	3
Transport tunnels in central area	Rock mass between deformation zones	3
Installation tunnel and pedestrian tunnel in central area	Rock mass between deformation zones	3
Skip tunnel	Rock mass between deformation zones	3
Elevator and skip shaft	Rock mass between deformation zones	3
Ventilation shaft	Rock mass between deformation zones	No rock support. If necessary (for example where protection of installations is required) some form of rock support such as mesh must be installed.
Ramp (access tunnel)	Rock mass between deformation zones	3

Supplementary rock support may be needed in some facility parts in the central area due to for example specific demands regarding fire resistance. The need of supplementary rock support in these facility parts must thus be evaluated in later design steps.

Supplementary rock support may also be required in the ramp when passing the sub-horizontal near-surface zones. The rock in these zones is considered to be of relatively good quality and generally of limited thickness, although the orientation of the ramp in relation to the zones is unfavourable, which may lead to stability problems. Supplementary rock support may comprise shotcrete anchored by rock bolts and/or pre-bolting ahead of the tunnel face. It is considered especially important to install bolts in the roof close to the tunnel face. In the present design step, however, it is assumed that the minimum rock support will ensure adequate stability. Another important issue is to investigate the rock mass ahead of the tunnel face before excavation. This can be done by careful probe drilling.

Supplementary support in the shafts when passing the sub-horizontal near-surface zones is not considered necessary owing to favourable orientation of the shafts in relation to the zones.

In order to minimise the problem of local minor spalling at intersections between main tunnels and deposition tunnels, the shotcrete will be applied both to the walls and to the roof at these locations (approx. 10 m into the deposition tunnels and 5 m in the main tunnels at both sides of the deposition tunnels). Systematic bolting will also be needed in the walls and roof at these locations (approx. 5 m into the deposition tunnels and 5 m in the main tunnels at both sides of the deposition tunnels).

Locally more extensive spalling may occur and can be supported where necessary by supplementary measures, such as steel mesh reinforced shotcrete. This supplementary rock support is however not included in the estimation of quantities.

It is assumed that all spot bolts are installed without washer and nut before application of shotcrete, whereas all systematic bolts are installed with washer and nut after the shotcrete is applied. The diameter of all rock bolts is set at 25 mm, and the size of the boreholes for installation of bolts should be 45 mm.

Finally it should be noted that no covering layer is required over the bolt ends or the fibre reinforced shotcrete since the bolt and all ancillary components such as washers and nuts fulfil the requirement of rock bolts exposed to air. Furthermore, the steel fibre reinforced shotcrete will be applied mainly to the roof, resulting in insignificant risks of injury to personnel due to exposed fibres.

9.4 Estimation of rock support quantities

The geometries of different parts of the facility are based on the alternative layouts presented in Chapter 5, and the cross-sections given in Sections 4.2 and 4.7.

The compositions of shotcrete and grout for anchoring of rock bolts were provided by SKB as were quantities for temporary concrete plugs. It should be noted that SKB has previously worked on the development of compositions of concrete and grout with a pH < 11. No such compositions are yet available, and consequently the compositions provided by SKB should be regarded as preliminary. Also, the composition of the shotcrete does not meet the recommendations given in SS-13 70 03 /SIS 2002/ with respect to water/cement ratio, cement content, and maximum content of silica fume for exposure class XS3.

In Table 9-7 and Table 9-8 a summary of quantities for different facility parts are presented. The estimation of quantities, which is based on the rock support shown in Table 9-5, refers to the permanent rock support in the hard rock facility.

Supplementary quantities of rock bolts and shotcrete in the intersections between deposition and main tunnels are included in the estimate for the main tunnels (i.e. not for the deposition tunnels). Furthermore, the estimation of quantities does not include the possible need for mesh in deposition tunnels.

A more detailed description of the premises for the estimation of quantities together with the quantity of other items such as cement, steel and additives is presented in Appendix E.

Table 9-7. Quantity of rock support for different facility parts.

Facility part	Layout 1 & 2 Shotcrete, m^3		Layout 1 & 2 Rock bolts, no		Layout 3 & 4 Shotcrete, m^3		Layout 3 & 4 Rock bolts, no	
	Min	Max	Min	Max	Min	Max	Min	Max
Deposition tunnels			17,769	30,317			17,561	30,439
Main tunnels	5,656	6,685	25,515	28,030	5,590	6,606	25,462	27,923
Transport tunnels in deposition area	1,335	1,578	1,661	3,212	1,413	1,670	1,707	3,325
Ramp	1,584	1,872	2,000	4,000	1,980	2,340	2,500	5,000
Tunnels and rock caverns in central area	992	1,173	1,238	2,475	992	1,173	1,238	2,475
Shafts	807	954	425	850	997	1,179	525	1,050
Total	10,374	12,262	48,608	68,884	10,972	12,967	48,993	70,212

Table 9-8. Quantity of concrete and steel reinforcement for concrete plugs.

Facility part	Layout 1 & 2 Concrete for concrete plugs, m^3	Layout 1 & 2 Steel reinforcement for concrete plugs, tonne	Layout 3 & 4 Concrete for concrete plugs, m^3	Layout 3 & 4 Steel reinforcement for concrete plugs, tonne
Deposition tunnel	26,180	1,571	26,600	1,596

9.5 Conclusions

The following conclusions can be made with regard to the assessment of rock support need and estimation of quantities for all rock support elements.

The rock quality is generally very good and no major stability problems are expected. The rock support is installed primarily to ensure that no isolated blocks or smaller pieces of rock fall out. Most of the rock reinforcement will be installed as minimum support, including spot bolting and a 50 mm thickness of steel fibre reinforced shotcrete in the roof. It is assumed that this minimum level of rock support will be installed irrespective of the rock quality.

The quantity of rock support is larger for 500 m depth (layout 3 and 4) than for 400 m depth (layout 1 and 2), but the differences are small.

The reported levels of rock stress are not expected to give rise to major problems with spalling. Since variations in the stress levels are likely and/or the strength of the rock mass may be lower in some locations, some local minor spalling may occur. A certain amount of spalling may be expected in the lower parts of the vertical shaft and at intersections between main tunnels and deposition tunnels.

Supplementary rock bolting and shotcrete are considered appropriate to minimise any problems due to minor spalling at intersections between main tunnels and deposition tunnels. Rock support consisting of steel mesh reinforced shotcrete may be required if more extensive spalling should occur.

The issue of a minimum rock support should be investigated further by SKB, since this is primarily an issue for the operator of the hard rock facility. The extent to which shotcrete or steel mesh should be used must also be investigated. Whether or not shotcrete or mesh should be used depends on the quantities of the different materials considered acceptable based on a safety assessment, as well as on the time and costs involved in installing the various reinforcing elements. When shotcrete is to be used it should also be determined whether unreinforced shotcrete should be used instead of steel fibre reinforced shotcrete.

Supplementary reinforcement in the ramp may be required where it crosses sub-horizontal near-surface fracture zones owing to unfavourable orientation of the ramp in relation to these zones. For example, rock anchored shotcrete and/or pre-bolting may need to be installed. It is considered particularly important to continuously install roof bolting close to the tunnel face. Continuous probe drilling ahead of the tunnel face should be carried out during excavation of the ramp in order to locate where these zones intersect the ramp, enabling suitable sealing and support measures to be determined.

Further investigation of the rock support need for the shafts should be undertaken. The extent of reinforcement in the shaft may for example need to be increased where sensitive installations are present.

10 Technical risk assessment

10.1 Introduction

According to /SKB 2004a/, design step D1 should include a technical risk assessment. The risk assessment is based on the completed design according to Chapters 3–9.

The objective of the technical risk assessment was to establish a feedback between the design results and the main goals of rock engineering in design step D1 (see Chapter 1). The purpose of the feedback was to ensure that the premises comprising the design basis should be illuminated from several aspects with a view towards the aforementioned goals.

According to /SKB 2004a/ the technical risk assessment should focus on the following three goals of design step D1, which are to:

1. Determine whether the deep repository can be accommodated within the site studied.

2. Identify site-specific facility-critical issues and provide feedback to:
   - the design organisation regarding additional studies that need to be done,
   - the site investigation and modelling organization regarding further investigations required,
   - the safety assessment team.

3. Test and evaluate the design methodology specified in /SKB 2004a/.

Goal 1 was analysed by performing a sensitivity analysis and a probabilistic analysis of the influence of different parameters on the layout.

Based on the results of the probabilistic analysis with regard to Goal 1 and the completed design according to Chapter 3-9, a summary was made of the critical issues according to Goal 2. Due to the high rock stresses in the Forsmark area, a “what-if scenario” regarding even higher stress levels was also described in more detail. Feedback to the site organisation, design- and safety assessment teams were given in terms of measures to be made in order to increase the probability that the repository will be accommodated within the Forsmark site.

Risks related to the design methodology (according to Goal 3) were described based on the experience gained from the design work. This description of risks includes an evaluation of methods, analyses performed, specific requirements and criteria as well as background material.

The technical risk assessment did not include events that are associated with the construction and operating phases or the post-closure phase. Technical risk assessments with respect to the construction phase will be carried out in later design steps.

10.2 Assessment of the possibility to accommodate the repository

10.2.1 Input data and assumptions

The proposed layout for the depth 400 m was used as a basis for a probabilistic analysis (see Chapter 5). This layout was then supplemented with additional areas available for deposition within the “preferable repository area” defined in Chapter 3. These additional areas comprise the area north-west of deformation zone ZFMNE00A2 and the excessive area, which is not currently utilized for deposition, between the zones ZFMNE0062A and ZFMNE0060. In the following text this enlarged layout is denoted the base layout (see Figure 10-1).

The area closer to the power plant and further out from the shore line was excluded, since it was desirable to asses the probability of the layout to be accommodated within the area regarded as most favourable (see Chapter 3). If deposition tunnels should be constructed closer to and below the power plant or further out from the shore line, this would result in approximately 800 more available canister positions (after reduction of loss of deposition holes). Additional 200–300 canister positions will also be available if a new main tunnel and about 20–30 short (minimum length is 100 m) deposition tunnels are constructed further to the north, close to the deformation zone ZFMNE1193. As an alternative to the construction of a new main tunnel to the north, 200–300 canister positions would probably also be available if an optimisation of the present layout is made. In total 1,000–1,100 extra positions are thus available within the defined “priority site” based on the geological model in /SKB 2005a/.

It should be noted that the positions of all tunnels besides the deposition tunnels are fixed in the analyses. The base layout should therefore be seen as a possible, probably conservative, layout since no optimisation has been made with respect to different combinations of parameters. It is however possible that after long sections of the main tunnels have been constructed, unforeseen geological conditions may be encountered, which will influence the location of the deposition tunnels.

Table 10-1 summarises data for the base layout. Presented figures are based on a total length of main tunnels of 6,524 m and a total length of deposition tunnels of 55,864 m. The used loss of deposition holes was based on “preliminary data” (see Section 4.5).

In the base case the volume at U=1 is $V_{U=1}$=1,627,155 m^3.

The volume considered in the present report relates to the main tunnels and deposition tunnels only. The transport tunnels are thus not considered. In Table 10-2 the input data for the probabilistic analysis are summarized. The input data were chosen together with SKB and was based on the information given in /SKB 2005a/ regarding properties of deformation zones and the results from the design tasks, which are presented in previous chapters. It should be noted that the parameters are the same as those studied in the sensitivity analysis (see Chapter 5).

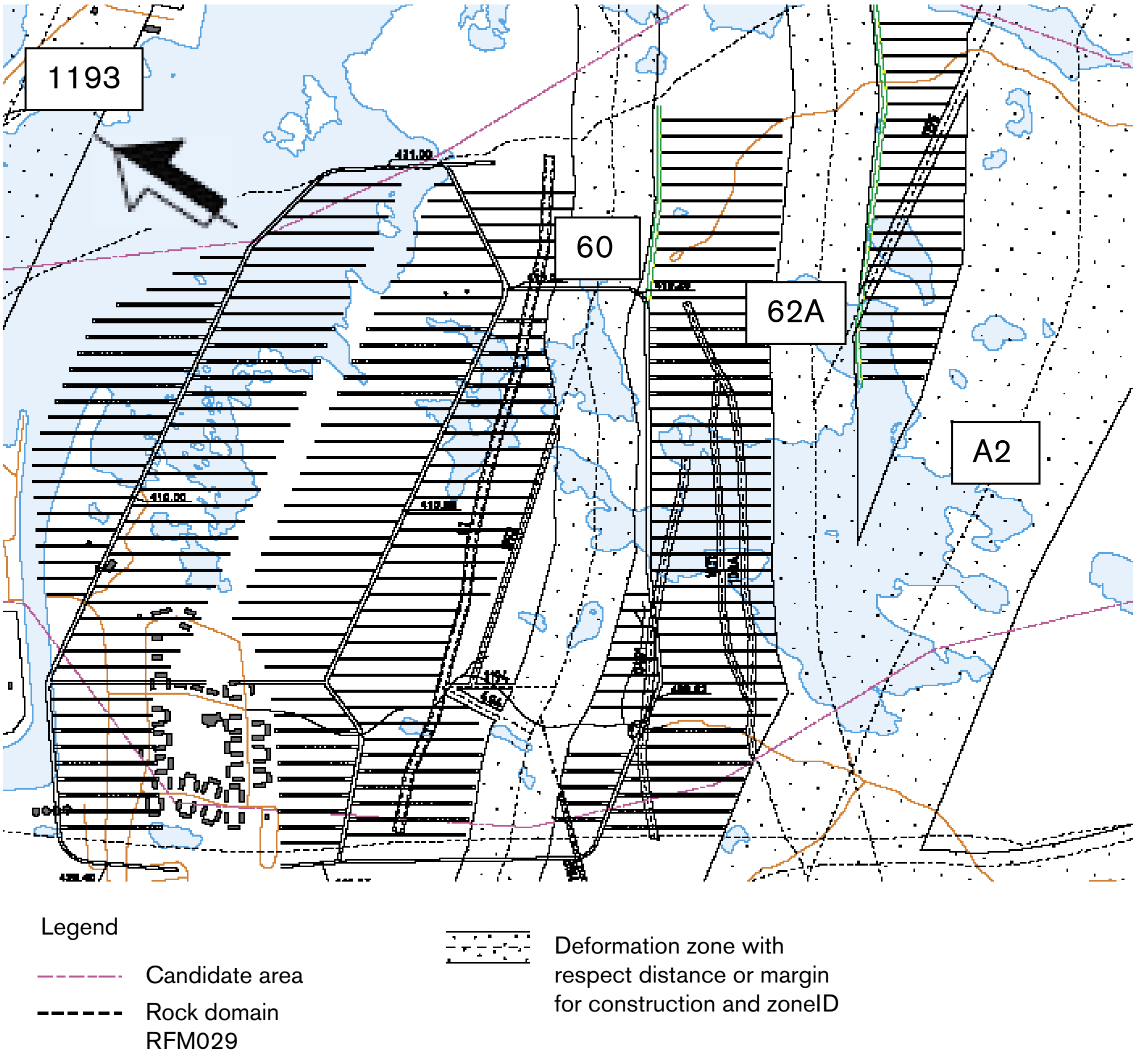


__Figure 10-1.__ Base layout for probabilistic analysis, depth 400 m. Main tunnels in green and the associated deposition tunnels are principal locations of possible complementary deposition areas.

Table 10-1. Data for base layout in Figure 10-1. N_T is defined as the number of positions available after reduction with respect to a loss of deposition holes of 11%.

Parameter	Description	Layout parameters
V	Volume of rock excavated (m^3) (theoretical volume)	1,853,280 m^3
U	Utilisation ratio defined as 6,000/N_T, where N_T is the number of canister positions available after taking the loss of deposition holes into account.	83.8% (N_T = 7,157 available positions)

Table 10-2. Input data for probabilistic analysis.

Parameter	Variation/probability
Presence and length of deformation zones	1a) The probability of ZFMNE0060 being longer than 3,000 m, P(length > 3,000 m), is taken as 75%, while the probability of it being shorter than 3,000 m, P(length < 3,000 m), is taken as 25%. These probabilities are based on ZFMNE0060 being a medium confidence zone. 1b) The probability of the frequency of deformation zones shorter than 3,000 m in the north-west part is assumed to be the same as for the south-east part (base layout) and is set to 50%. (No variation in the intensity is assumed – the intensities are identical in case there are zones shorter than 3,000 m in the north-west part.)
Dip of deformation zones	The variation is based on the information given in /SKB 2005a/. ZFMNE0060 87° –10° ZFMNE062A 73° ±10° ZFMNE00A2 24° –10° The estimates are taken as the mean value and the ±10° as 5-percentiles in a triangular distribution (see Figure 10-2, Figure 10-3 and Figure 10-4). (In the description of the variation a distribution with limited values is preferred, as this places a limit on the extremity of the values.)
Margin for construction (for deformation zones shorter than 3,000 m)	The distance is taken as 5 m (base layout) with a probability of 50% and as 10 m with a probability of 50%.
Distance between deposition holes (depends on thermal properties)	The distance is taken as 6 m (base layout) with a probability of 75% and as 5.5 m with a probability of 25%.
Loss of deposition holes	The estimate of 11% (based on "preliminary data") is taken as the mean value, and 1% and 21% as the cut-off values in a triangular distribution.

One prerequisite for the analysis is also that the variations of different parameter are assumed to be uncorrelated.

Limitations in tunnel length between deformation zones were considered when estimating the length and number of positions available for deposition in deposition tunnels. The procedure for estimating the limitations of tunnel length between deformation zones is described in the sensitivity analysis in Chapter 5. The minimum length is set to 100 m and the maximum length is set to 300 m.

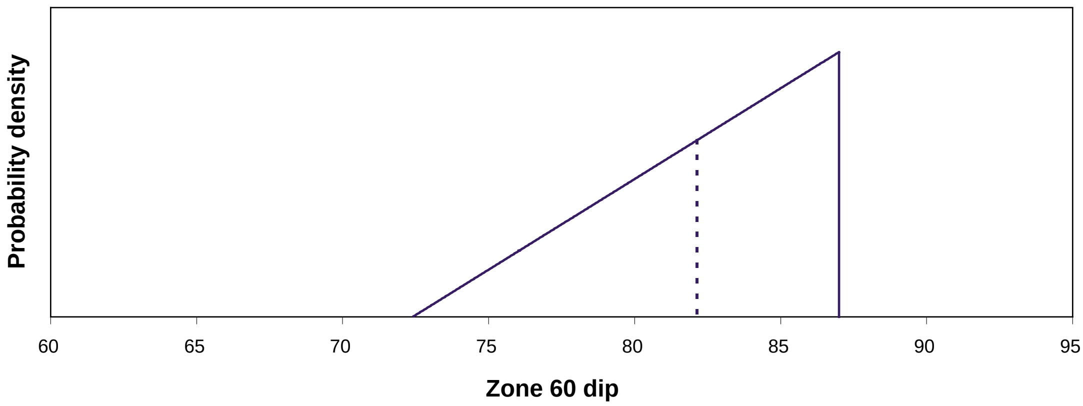


Figure 10-2. *Probability variation for the dip of deformation zone 60.*

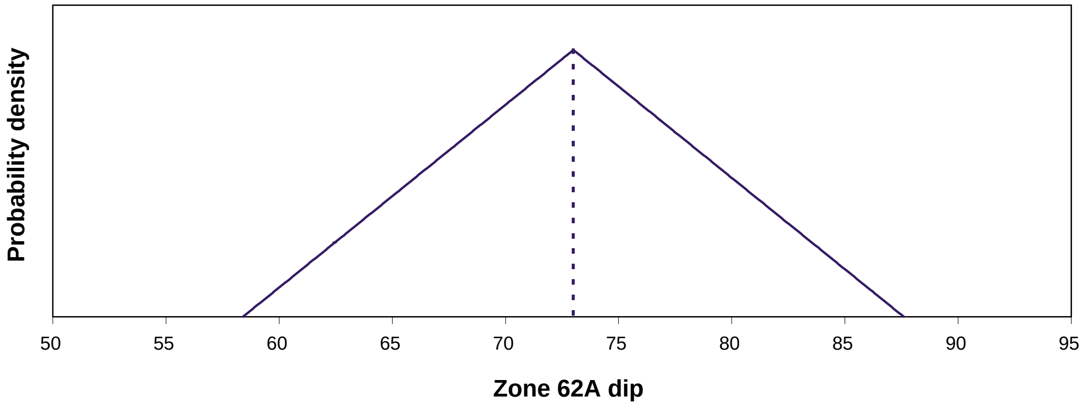


Figure 10-3. *Probability variation for the dip of deformation zone 62A.*

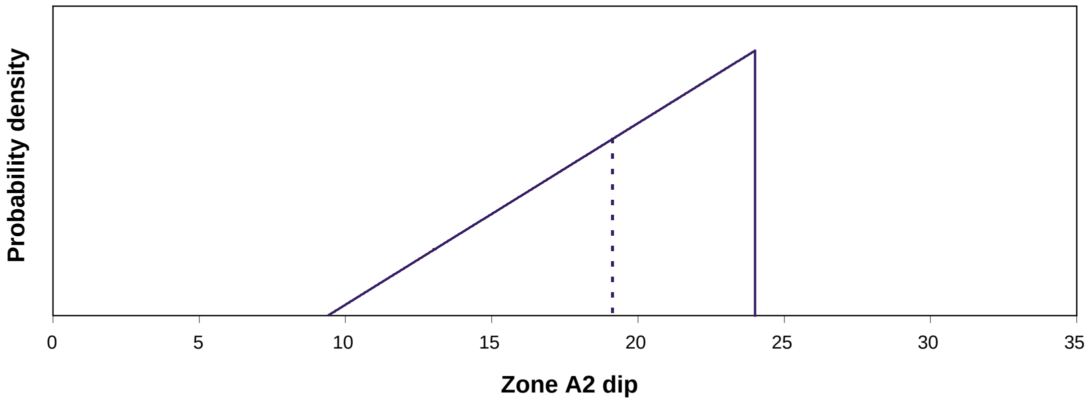


Figure 10-4. *Probability variation for the dip of deformation zone A2.*

10.2.2 Execution

The assessment of the possibility to accommodate the repository within the "priority site" was analysed by performing a probabilistic analysis of the influence of different parameters on the layout. This analysis was based on a Monte Carlo simulation in which all parameters are varied at the same time. The result of the probabilistic analysis provided:

- A quantitative estimate of the possibility of accommodating the repository.
- A ranking of different events with regard to their influence.

The results were achieved based on 10,000 simulations and considered the variation in:

- N_T – the number of positions available.
- U – utilisation ratio (6,000/N_T).
- $V_{U=1}$ – the volume corresponding to 6,000 available positions.

10.2.3 Results

N_T –number of positions available

The distribution curve for N_T is indicated in Figure 10-5, based on a Monte Carlo simulation with a sample size of 10,000 simulations.

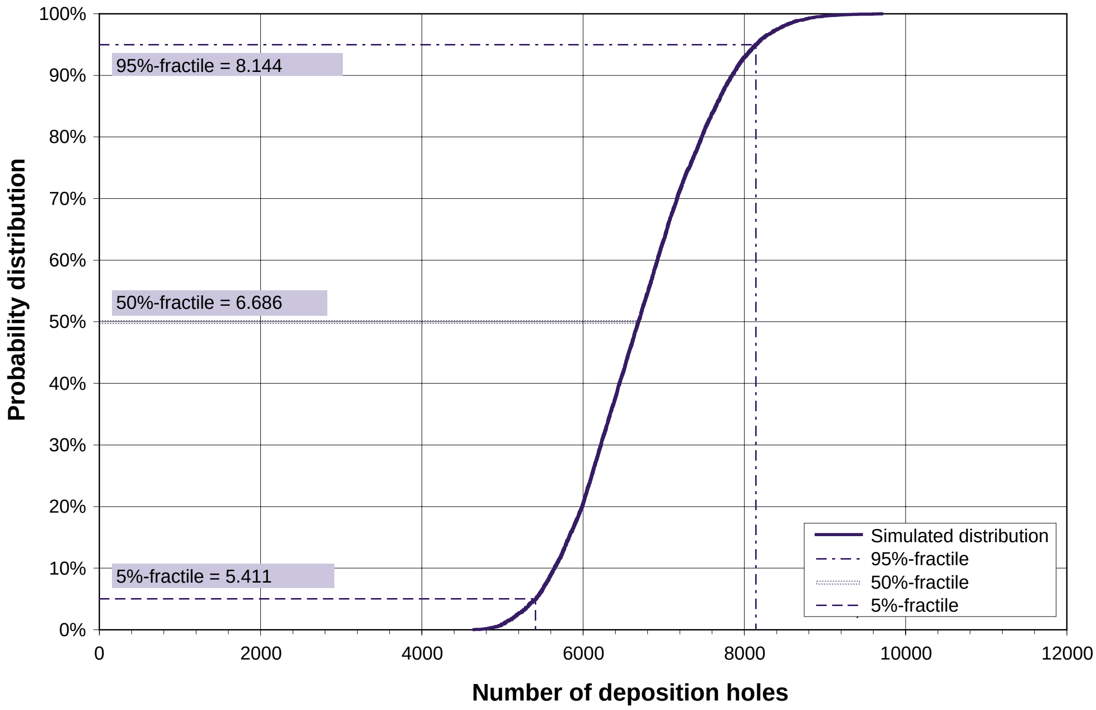


Figure 10-5. *Distribution curve for the number of deposition holes N_T.*

Besides the variation the following main results are also obtained:

- Mean value of N_T is $E(N_T) = 6,723$.
- The probability of the number of deposition holes being less than 6,000 $(P(N_T < 6,000)) = 20\%$.
- The probability of the number of deposition holes being less than 5,400 $(P(N_T < 5,400)) = 5\%$.
- The probability of the number of deposition holes being less than 5,000 $(P(N_T < 5,000)) = 1\%$.

It can be seen that the expected number, 6,723, of deposition holes available is 6% less than the number calculated in the base layout (7,157). The reason for this is that the variation of N_T with the uncertain parameter is highly non-linear, in addition to the effect of on-off variables. Consequently the expected number of positions available in the base layout is overestimated.

This also implies that the probability of the repository area being unable to accommodate 6,000 canisters is estimated to be as high as 20%, based on the parameters considered and their variation. In view of the effect of the main tunnel locations being fixed in the simulations, this value is assumed to be higher than should be encountered if the layout is optimised for the actual conditions. The probability of 6,000 canisters not being accommodated is therefore likely to be less than 20%. Also, it must be emphasized that this risk assessment only involves the area within the defined “preferable repository area”, and consequently did not consider the possible available additional area for deposition underneath the sea and the nuclear plant. If this area also would be utilised, the additional available canister capacity would be 1,000–1,100 positions. This number of canister positions is of the same order as the number of available canister positions needed to have a confidence of 99% that the repository will be accommodated within the “priority site”. Furthermore, there is a potential to accommodate even more canisters south of the deformation zone ZFMNE00A2 (see Appendix A) and it may also be possible to allow for 10°C higher temperature when evaluating the canister spacing according to Figure 5-4 in /SKB 2004a/ (see Section 2.3.2).

The influence of the uncertainty of the various parameters on the overall uncertainty is illustrated in the sensitivity graph in Figure 10-6.

From Figure 10-6 it can be seen that more than 40% of the overall uncertainty is due to the uncertainty of the deformation zone ZFMNE0060 being longer than 3,000 m, while more than 25% of the overall uncertainty is due to the dip of the zone.

U – utilisation

The distribution curve for *U* is given in Figure 10-7 based on a Monte Carlo simulation with a sample size of 10,000 simulations.

Besides from the variation the following main results are obtained:

- Mean value of U is E(U) = 0.91.
- The probability of the utilisation ratio being less than 1.0: P(U < 1.0) = 80%.

$V_{U=1}$ – volume corresponding to 6,000 positions available

The distribution curve for $V_{U=1}$ is given in Figure 10-8 based on a Monte Carlo simulation with a sample size of 10,000 simulations. Reduction of the number of tunnels in the event of over-capacity in the area has been achieved by eliminating the tunnels furthest from the access ramp.

It should be noted that simulations giving an outcome of the uncertain parameters for which the utilisation ratio exceeds 1.0 have been excluded from the sample.

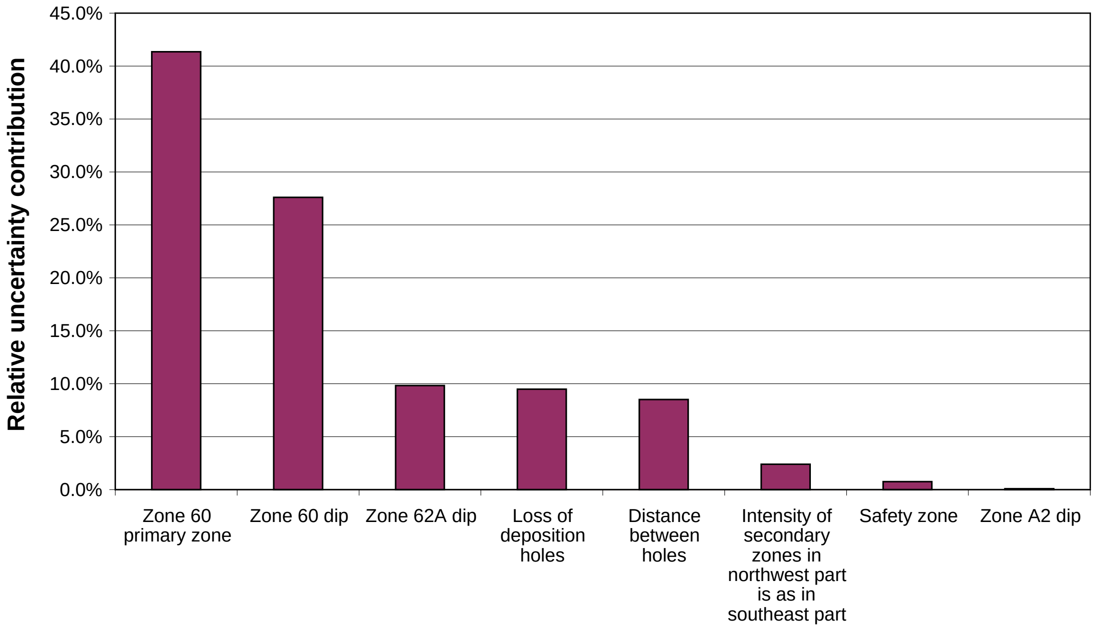


Figure 10-6. *Sensitivity graph for N_T. In this figure a "primary zone"= deformation zone longer than 3,000 m and a "secondary zone" = deformation zone shorter than 3,000 m.*

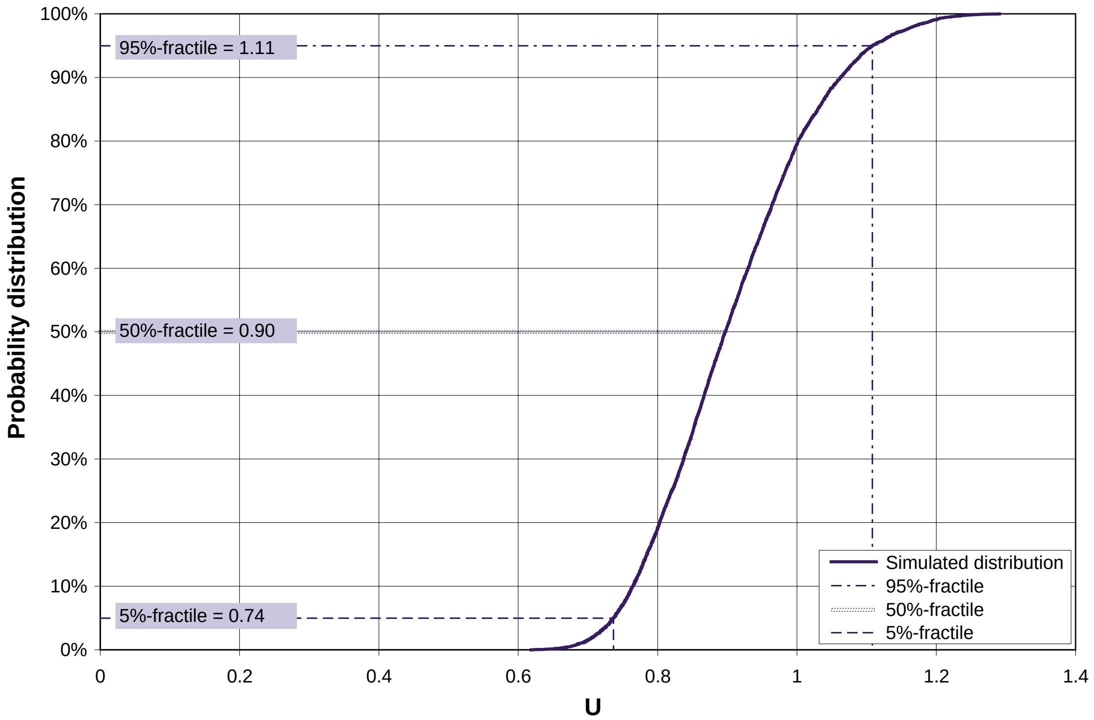


Figure 10-7. *Distribution curve for utilisation ratio U.*

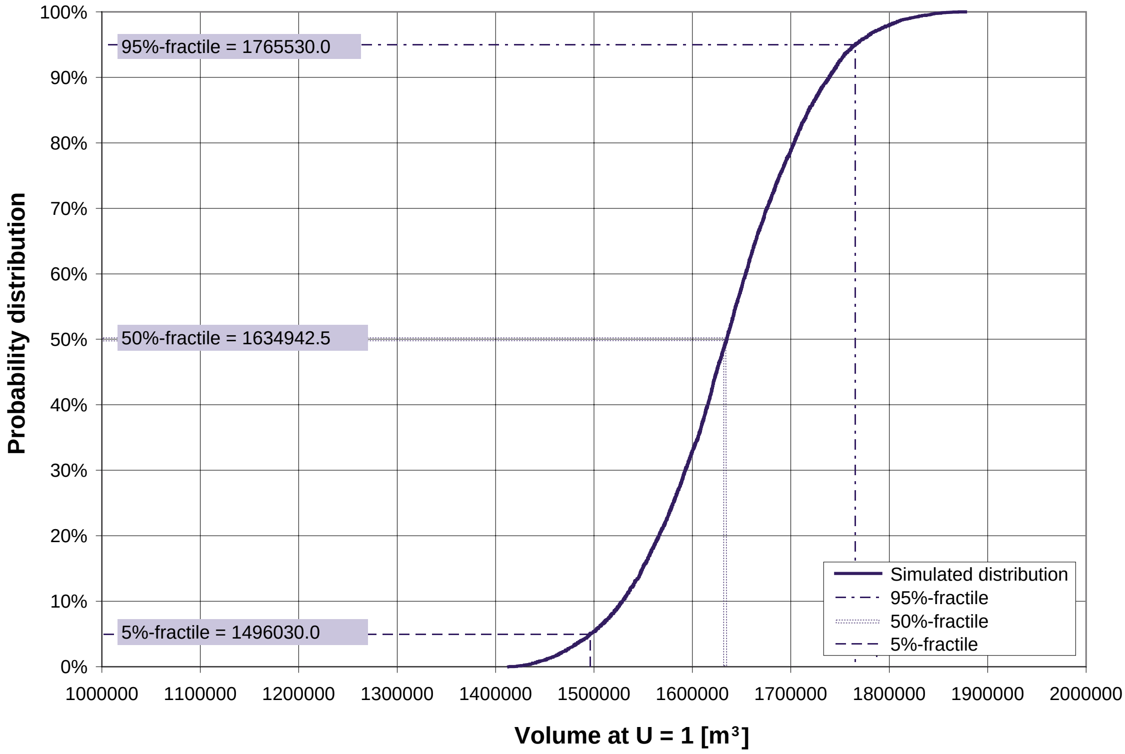


Figure 10-8. *Distribution curve for the volume $V_{U=1}$ given U=1. (Samples with utilisation ratio greater than 1.0 are excluded.)*

Besides from the variation the following main result was obtained:

- Mean value of volume given U=1 is $E(V_{U=1})$ = 1,631,841.4 m³. (More or less equal to the base case 1,627,155 m³).

In the above situation it is not straightforward to illustrate the sensitivity as the sample is incomplete owing to the exclusion of samples with utilisation ratio exceeding 1.0. In order to provide an indication of the sensitivity, the distribution curve for "$V_{U=1}$ given U=1" and the corresponding sensitivity are illustrated in Figure 10-9 and Figure 10-10, where samples with an utilisation ratio exceeding 1.0 are *not* excluded.

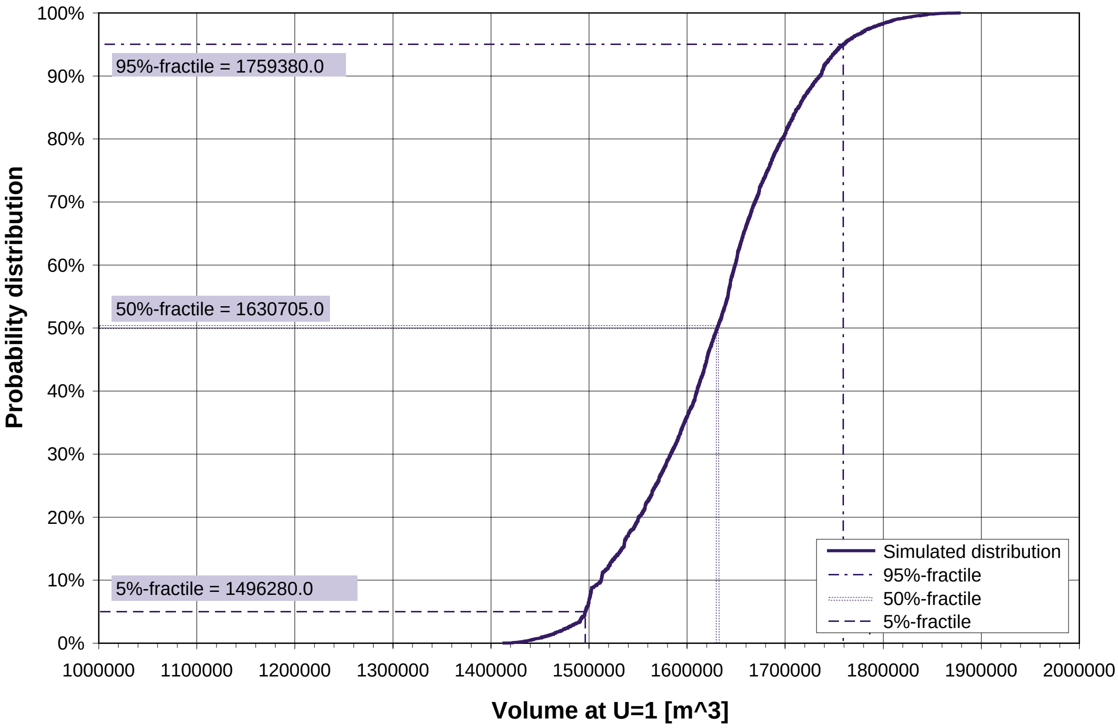


Figure 10-9. *Distribution curve for the volume "$V_{U=1}$ given U=1". (Samples with utilisation ratio exceeding 1.0 are not excluded.)*

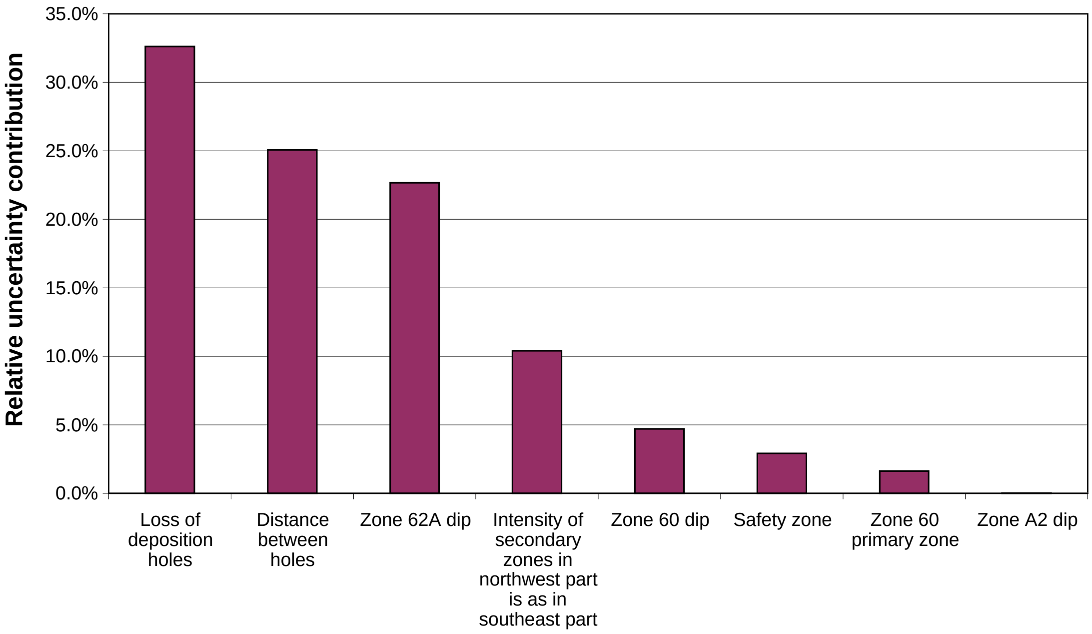


Figure 10-10. *Sensitivity graph for "$V_{U=1}$ given U=1". (Samples with utilisation ratio exceeding 1.0 are <u>not</u> excluded.) In this figure a "primary zone"= deformation zone longer than 3,000 m and a "secondary zone" = deformation zone shorter than 3,000 m.*

Besides from the variation the following main result was obtained:

- Mean value of volume given U=1 is $E(V_{U=1})$ = 1,626,891 m³. (The estimate decreases as a consequence of having samples with U > 1.)

The results indicate that the loss of deposition holes and the distance between deposition holes are the major contributory factors to the overall uncertainty of the volume given that U=1.

10.2.4 Sensitivity analysis

In order to evaluate the sensitivity of the number of positions available (N_T) due to changes in input data, a simulation was made with other variations of the dip of two deformation zones. In this analysis the dip of the following deformation zones was set at:

ZFMNE0060	87° ±10° (instead of 87°–10°)
ZFMNE062A	73° ±10°
ZFMNE00A2	24° ±10° (instead of 24°–10°)

The distribution curve for N_T with the new input data for the dip of deformation zones is given in Figure 10-11. It should be noted that the locations of all tunnels except deposition tunnels are fixed. The effect of this premise is that the result is probably conservative, and that the probability of accommodating 6,000 canisters or more is higher than calculated.

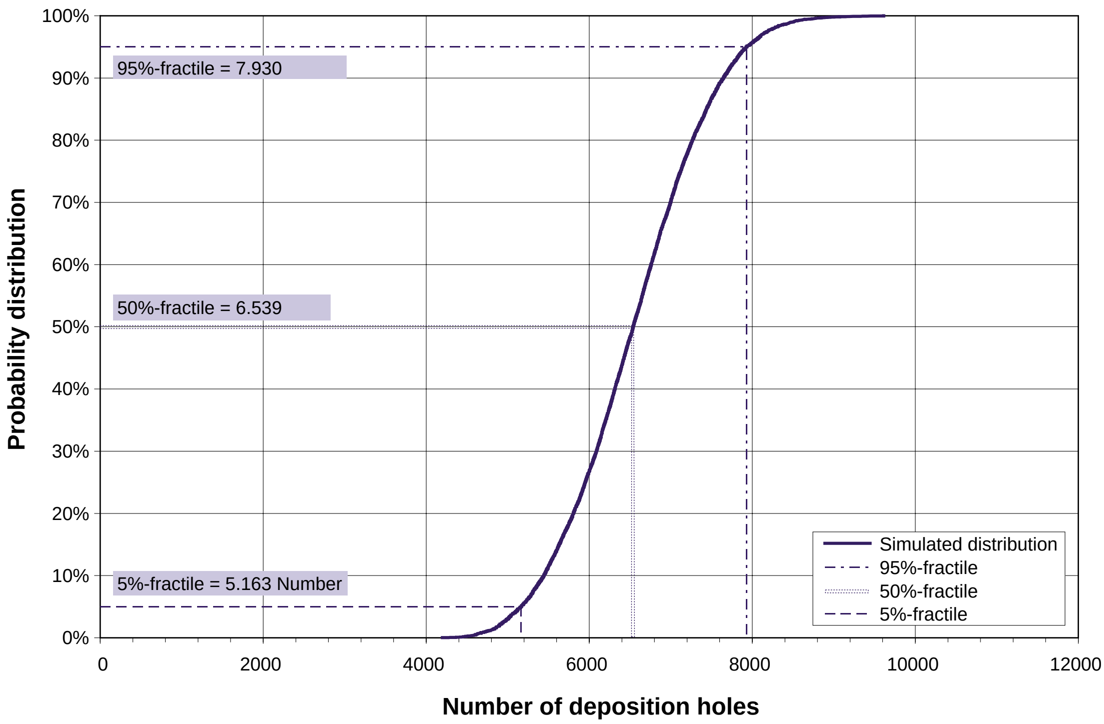


Figure 10-11. *Distribution curve for the number of deposition holes N_T, changed input data with regard to the dip of deformation zones.*

Besides from the variation the following main results were obtained:

- Mean value of N_T is $E(N_T) = 6,544$.
- The probability of the number of deposition holes being less than 6,000 $P(N_T < 6,000) = 27\%$.

This also implies that the probability of the area being unable to accommodate 6,000 canisters is estimated to be as high as 27%, based on the parameters considered and their variation. With reference to the effect of fixed main tunnels in the simulations, this value is assumed to be higher than would be encountered if the layout were optimised for the actual conditions. The probability of 6,000 canisters not being accommodated is therefore probably less than 27%. Nevertheless the probability of being unable to accommodate 6,000 canisters is higher than in the simulation illustrated in Figure 10-5. This is due to the significant effect of a more steeply dipping deformation zone ZFMNE00A2 on the area available for deposition (refer also to the sensitivity analysis in Chapter 5).

The influence of the uncertainty of the various parameters on the overall uncertainty is illustrated in the sensitivity graph in Figure 10-12.

It can be seen from Figure 10-12 that more than 40% of the overall uncertainty is due to the uncertainty of the dip of deformation zone ZFMNE00A2, while more than 15% of the overall uncertainty is due to the dip of deformation zone ZFMNE0060.

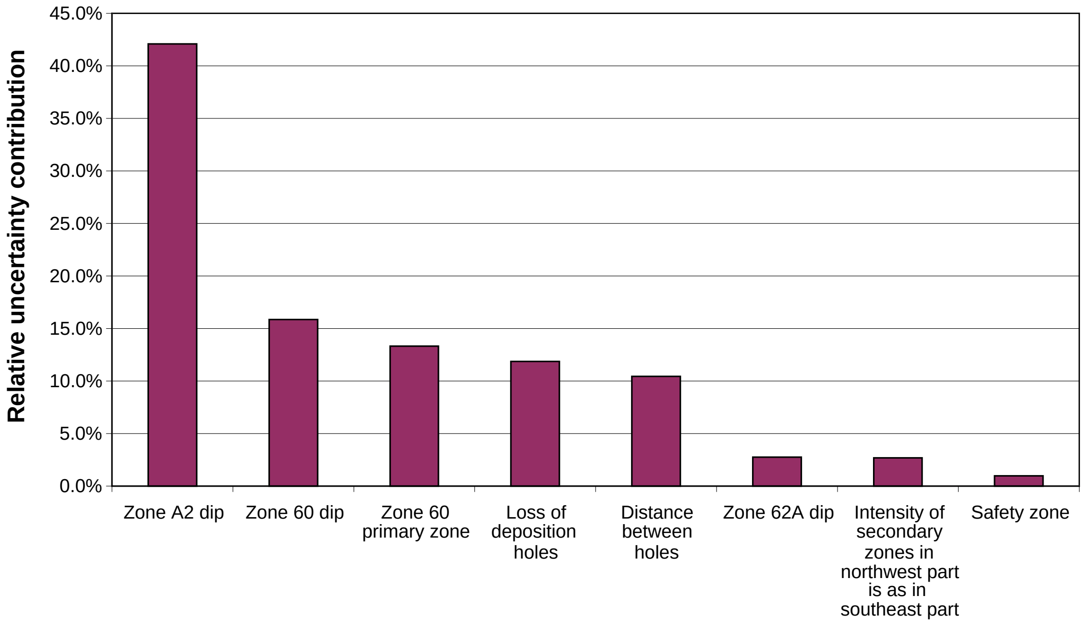


Figure 10-12. *Sensitivity graph for N_T, changed input data with regard to dip of deformation zones. In this figure a "primary zone"= deformation zone longer than 3,000 m and a "secondary zone" = deformation zone shorter than 3,000 m.*

10.3 Critical issues

10.3.1 Analyses of higher stress levels

Due to the high rock stress levels in the Forsmark area, and their potential influence on the possibility of the site to accommodate the repository, it was justified to make a complementary analysis of the effect of even higher stress levels. High stress levels that result in stability problems will also imply a potential risk for workers in the tunnels. With regard to safety for workers in the tunnels, some kind of risk assessment must be executed according to demands specified in AFS 2003:2 /Arbetsmiljöverket 2003/.

From the data used for /SKB 2005a/ it was concluded by SKB that an alternative conservative estimation of the maximum horizontal stress also could be possible (see Figure 10-13). It should however be noted, that the borehole DBT-1, for which the highest stress magnitudes are reported, is located north of the candidate area (near the power plant) and according to /SKB 2005a/ is located in a local geology that differs from the geology found in KFM01A and KFM01B.

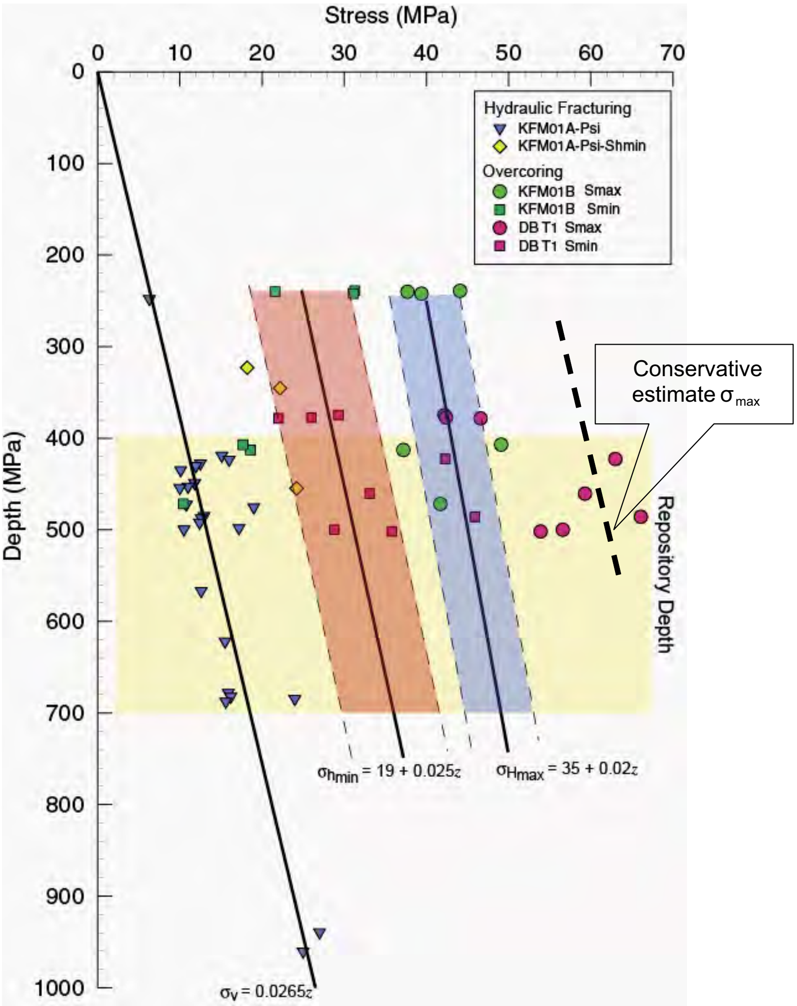


Figure 10-13. *Stress measurements and stress modelling for rock domain RFM029 given in /SKB 2005a/ and an estimation of a conservative maximum horizontal stress (given by SKB). The trend lines for* σ_v*,* σ_{hmin} *and* σ_{Hmax} *correspond to the in-situ stresses given in /SKB 2005a/. The dots refer to stress measurements in different boreholes. The cored boreholes KFM01A and B are located between the residential area and "Bolundsfjärden"(see Figure 3-1) and DBT1, which was drilled during the construction of the third unit for the nuclear power plant, is located in the area of the power plant.*

From Figure 10-13 it can be seen that that the maximum horizontal stress is about 10–15 MPa higher at 400–500 m depth when using the conservative estimate. Considering a variability of ±10% (see Table 4-3 in Section 4.4) a major horizontal stress of up to about 65 MPa should thus be possible.

A higher major horizontal stress will influence the stability of the deposition holes (loss of deposition holes) as well as the stability of tunnels and shafts.

Regarding the deposition holes the maximum boundary stress can be calculated with Equation 4-5 (see Section 4.5). In Table 10-3 the results for different input data regarding the major and minor horizontal stresses are presented. The variability of the in-situ stresses are assumed according to Table 4-3 in Section 4.4.

If the results in Table 10-3 are compared with the rock mass spalling strength (σ_{sm}), which is 120±5 MPa according to /Martin 2005/, it can be concluded that the maximum boundary stress exceeds the rock mass spalling strength (σ_{sm}) even for the case with calculated minimum value of the boundary stress. Thus, the probability of spalling will be very high in deposition holes, which would result in a high loss of deposition holes. It should be noted that results are calculated for the depth 450 m. Due to the small stress gradients the difference in stresses at 400 m and 500 m depth will however be small.

Regarding the stability in the tunnels a higher maximum horizontal stress will not influence the stability in the deposition tunnels, since these tunnels will be oriented parallel with the maximum horizontal stress. From the stress analyses, which are presented in Section 4.4, it can thus be concluded that the probability of spalling should still be low in the deposition tunnels. In the main tunnels, which are orientated approximately perpendicular (70–80 degrees) to the major horizontal stress, and in the curved parts of the ramp, the probability of spalling will however increase. In Table 10-4 the maximum boundary stresses are presented as calculated from Equation 4-5 and for a circular tunnel section orientated perpendicular to the major horizontal stress.

Table 10-3. Maximum boundary stresses for a deposition hole, when the stresses are calculated with Equation 4-5 for 450 m depth.

Maximum horizontal stress, conservative estimate (MPa)	Minimum horizontal stress given in /SKB 2005a/, (MPa)	Maximum boundary stress (MPa)
54 (min value)	36 (max value)	126
60 (estimated mean value)	30 (mean value)	150
66 (max value)	24 (min value)	174

Table 10-4. Maximum boundary stresses for a circular opening orientated perpendicular to the major horizontal stress and parallel to the minimum horizontal stress. The stresses are calculated with Equation 4-5 for 450 m depth.

Maximum horizontal stress, conservative estimate (MPa)	Vertical stress given in /SKB 2005a/, (MPa)	Maximum boundary stress (MPa)
54 (min value)	12 (mean value)	150
60 (estimated mean value)	12 (mean value)	168
66 (max value)	12 (mean value)	186

From the stress analyses in Section 4.4, which were executed with numerical calculations considering the actual tunnel geometry, it was however concluded that the stresses will be lower than if circular openings are assumed. Furthermore, the main tunnels are orientated with a smaller angle than 90 degrees to the major horizontal stress, which is favourable with respect to stability.

The extent of spalling will depend on the ratio between the stress and the strength of the rock. In Figure 10-14 an empirical classification of stability is presented. Regarding the uniaxial compressive strength it can from Figure 4-7 (Section 4.4) be concluded that the mean value of the uniaxial compressive strength (UCS) is 225 MPa, the 5% percentile value 189 MPa and 95% percentile value 261 MPa. With a major in-situ stress of 54–66 MPa, corresponding to the major horizontal stress at 450 m depth (see Table 10-4) and a uniaxial compressive strength (UCS) of 189–261 MPa, a ratio between the stress and strength of 0.21–0.35 can be calculated. The corresponding ratios, when using stress data from /SKB 2005a/, are 0.15–0.26. Based on Figure 10-14, only minor outfall of rock slabs and damage should thus be expected, even if the conservative estimate of the maximum horizontal stress is used.

It should however be noted that the classification of stability according to Figure 10-14 is derived for square tunnels. In tunnels with an arced tunnel roof, higher stresses will be encountered in the roof than for a square tunnel, and therefore the extent of spalling will be higher. The possibility of more extensive outfall and damage can thus not be excluded.

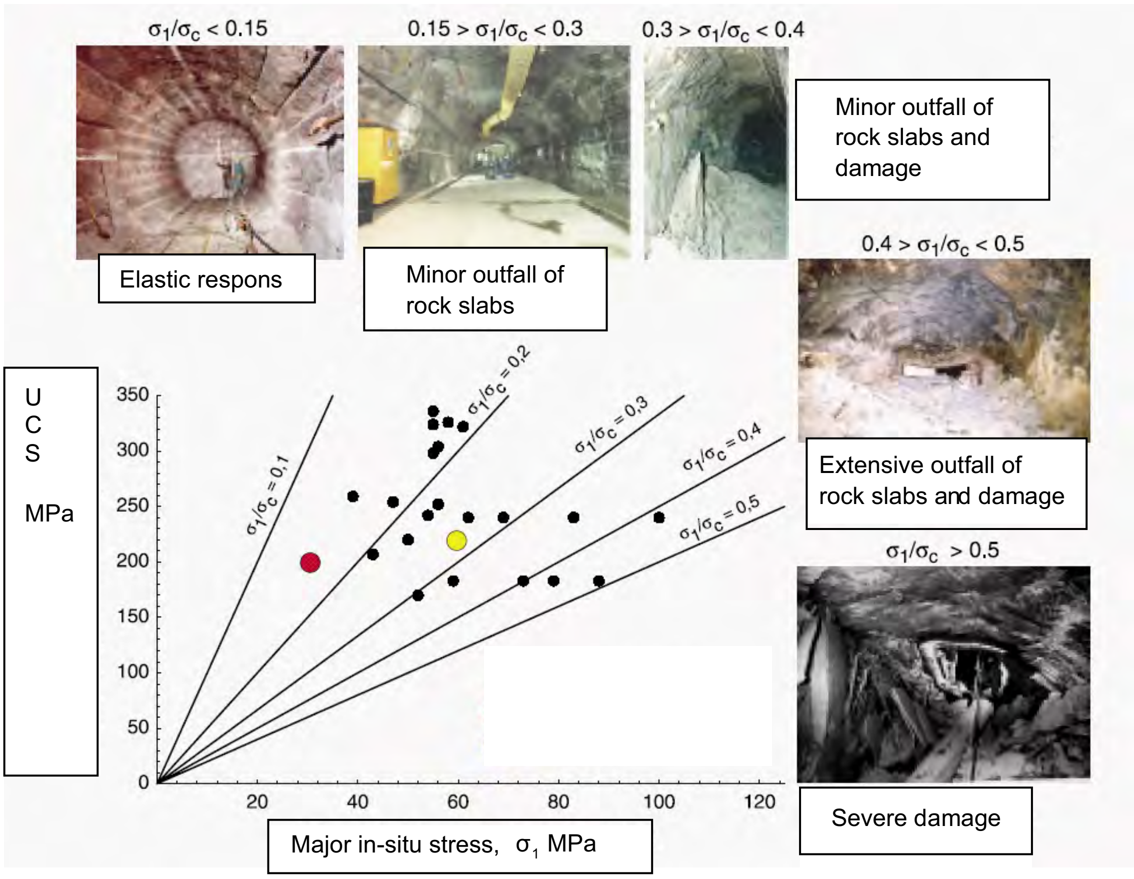


Figure 10-14. *Empirical classification of stability developed for tunnels with square sections in Southafrican mines. The figure is based on the Figure 5-1 in /Andersson et al. 2000/. The blue, red and yellow dots refer to data from different field measurements (no measurements are from the Forsmark area). UCS is the uniaxial compressive strength of intact rock.*

The extent of spalling for a circular opening, can also be estimated by using the empirical relationship in Figure 10-15, which is presented in /Martin 2005/.

From the Figure 10-15 the depth of spalling, S_d, from the contour of a circular can be derived as (D_f –a). For the calculated maximum boundary stresses given in Table 10-3 and Table 10-4 the corresponding depth of spalling, S_d, is presented in Table 10-5.

From Table 10-5 it can be concluded that the depth of spalling, S_d, can be relatively large compared to the radius of the opening. It should again be noted, that the maximum boundary stress in the main tunnels will be lower than the stresses calculated for a circular opening. The highest maximum boundary stress should then be about 130 MPa, which corresponds to a depth of spalling, S_d, of only 0.09 m instead of 1.0 m.

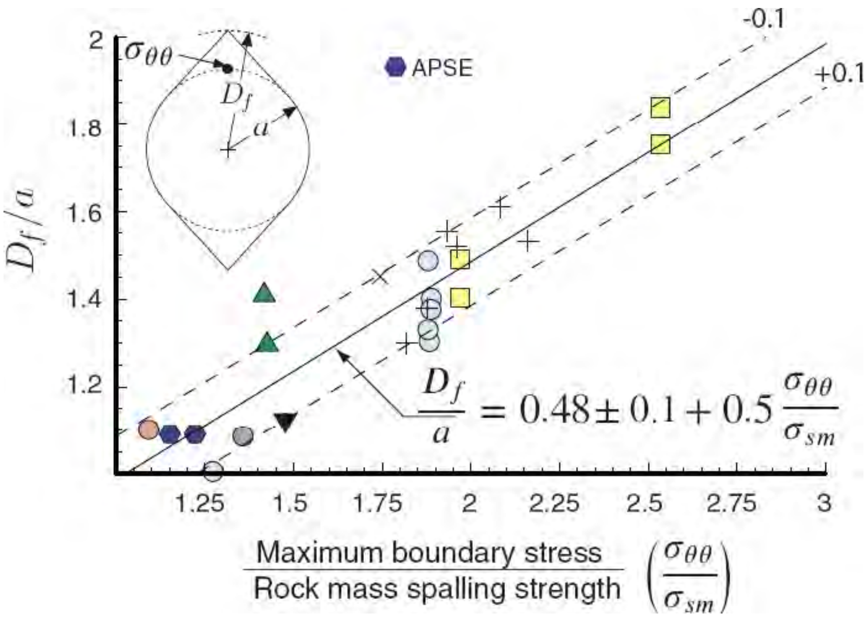


Figure 10-15. *Empirical relationship for the estimation of the extent of spalling in circular openings (from /Martin 2005/).*

Table 10-5. Depth of spalling, S_d, for a circular opening derived from Figure 10-15, when the rock mass spalling strength (σ_{sm}) is 120 MPa. D_f/a is the ratio according to Figure 10-15.

Facility part	Radius, a (m)	Maximum boundary stress, $\sigma_{\theta\theta}$ (MPa)	D_f/a	Depth of spalling, S_d (m)	S_d/a
Deposition hole	0.9	126	1.004	0.004	0.0044
		150	1.1	0.09	0.1
		174	1.2	0.18	0.2
Main tunnel	4 (assumed equivalent radius)	150	1.105	0.42	0.1
		168	1.18	0.72	0.18
		186	1.25	1.0	0.25

For the evaluation of stability in the shafts, the calculated boundary stresses which are presented in Table 10-3 can be used. Compared with the value of the rock mass spalling strength (σ_{sm}), which is 120±5 MPa according to /Martin 2005/, it can be concluded that spalling is likely to occur in the shafts. Assuming a mean value of the radius of 2 m for the shafts, a rock mass spalling strength (σ_{sm}) of 120 MPa the depth of spalling according to Figure 10-15 will be 0.01–0.41 m.

If the rock stress situation at Forsmark will be re-evaluated, based among others on additional stress measurements, and if higher stress in the range of the presented conservative assumption will be obtained, it is important that more detailed analyses are executed. Analyses of the extent and depth of spalling for non circular openings should for example be executed with finite element programs together with a failure envelope for spalling (see /Martin 2005/). An evaluation of possible methods for tunnel excavation and rock support must also be executed.

Based on the calculated depth of spalling, an estimation of the volume of spalling in a circular opening can be made according to equations presented in /Martin 2005/. For the forthcoming design steps it should be an important issue to asses the maximum acceptable volume of spalling for deposition holes and tunnels. It may be possible that some volume of outfall can be accepted even in the deposition holes. For comparison, a wedge breakout corresponding to a volume < 0.15 m^3 is accepted in /SKB 2004a/. Based on the the principles for estimating the volume of spalling according to /Martin 2005/, a volume of breakout in a deposition hole of 0.15 m^3 corresponds to a maximum boundary stress of about 130–135 MPa. A higher stress than the rock mass spalling strength (σ_{sm}) used in the previous analysis may thus be accepted. Accordingly, if some volume of breakout due to spalling is accepted, the loss of deposition holes may be lower than if the volume of breakout is not considered. However, with a conservative estimate of the magnitude of the maximum horizontal stress, the maximum boundary stress in a deposition hole according to Table 10-3 may still be higher than 130–135 MPa, resulting in a high probability of spalling.

10.3.2 Summary of critical issues

A summary of identified critical issues based on the probabilistic analysis and the analyses of possible higher stresses, is presented in Table 10-6. These critical issues are related to the number of canister positions available, N_T, and volume excavated, V. It can be seen from the analysis that various parameters will influence the number of positions available, N_T, and volume excavated, V. While the location and orientation of the deformation zones affect the number of positions available, N_T, parameters influencing the proportion of deposition holes lost and distance between deposition holes are of more importance when considering the excavated volume, V.

Other uncertainties relating to technical aspects identified in the various design tasks are described in Table 10-7.

Table 10-6. Critical issues based on the probabilistic analysis and analysis of higher stresses together with measures proposed.

Object	Event	Consequence	Measures proposed
Degree of utilisation/number of available canister positions	Higher rock stresses than estimated.	The loss of deposition holes will be too high, which can result in the conclusion that the site should not be used.	Investigations to reduce uncertainties in the magnitudes and orientations of in-situ stresses within the “priority site”. Investigations in order to asses the maximum acceptable volume of breakout in deposition holes.
Excavated volume	Unfavourable conditions leading to higher losses of deposition holes.	Excavated volumes will affect cost, time and environmental impact.	Investigations to reduce uncertainties in the magnitudes and orientations of in-situ stresses within the “priority site”. The proportion of lost deposition holes must be analysed for different orientations of the deposition tunnels. The orientation should then be based on the results of such analysis.
Excavated volume	Excessive distance between deposition holes.	Larger volume of rock excavated than necessary, affecting cost, time and environmental impact.	Analyses executed according to /SKB 2004a/ (see Section 4.3) indicate that a shorter distance between deposition holes than 6 m may be possible with regard to the thermal properties. Investigations should be performed to evaluate the effects of shorter distances between deposition holes. These investigations must take into account the rock mechanical aspects and the distance between deposition tunnels. It may also be possible to allow for 10°C higher temperature when evaluating the canister spacing according to Figure 5-4 in /SKB 2004a/ (see Section 2.3.2).

Table 10-7. Critical issues identified in the various design tasks and measures proposed.

Object	Event	Consequence	Measures proposed
Seepage and hydrogeological situation	Total salinity (TDS) becomes excessive around the deposition area.	Reduced sealing efficiency of the bentonite buffer around canisters.	Further investigations of the upcoming of saline water and maximum steady state seepage for estimation of the total salinity (TDS). The calculation methods should be further evaluated in this investigation.
Grouting need	Unfavourable or unforeseen conditions when crossing shallow, water-bearing, sub-horizontal deformation zones.	Difficulties in constructing the access tunnel and shafts will affect cost, time and environmental impact.	Investigations to determine the location and properties of the shallow sub horizontal deformation zones. Grouting tests to be performed. Further design with respect to drilling, grouting techniques, grouts and control measures.
Grouting need	Stringent requirements on maximum permitted water inflows to various parts of the facility.	A too high sealing demand will render cement grouting difficult or impossible.	Requirements for maximum permitted water inflows to various parts of the facility must be established. Investigations of the possibility of using non cement-based grouts.
Rock support need	Higher rock stresses than estimated will lead to more extensive spalling.	Additional rock support works will affect cost and time. Safety of personnel may be affected. If too extensive spalling will occur, the possibility of constructing the repository may be re-evaluated.	Investigations to establish a high degree of confidence in the magnitude and orientation of in situ stresses in different locations of the “priority site”. Numerical calculations with conservative estimates of the in-situ stresses in order to assess the maximum boundary stresses and the probability of spalling. Especially the stresses in the main tunnels should be analysed due to an unfavourable orientation relative the maximum horizontal stress. Further investigation of the requirement for rock support in tunnels and shafts, taking into account the potential problem with rock spalling. Establishment of a strategy regarding tunnel excavation in rock with potential spalling problems. Orientation of the main tunnels with a smaller angle to the maximum horizontal stress.

10.4 Evaluation of design methodology and design premises

The design premises described in /SKB 2004a/ is considered to constitute an overall design principle covering the design issues requiring analysis in order to achieve the objectives of design step D1. However, it is also considered that the design should be executed in a somewhat different sequence in order to achieve a more optimised layout. In the present version of /SKB 2004a/ the orientation of the deposition tunnels should be determined before analysing the loss of deposition holes and selecting the repository depth. This sequence will achieve a tunnel orientation and depth based primarily on rock mechanics and hydrogeological aspects rather than on an optimal layout. It should be recognized that the number and length of tunnels significantly influence the cost, time and environmental impact. It is therefore possible that a tunnel orientation involving a comparatively large proportion of deposition holes lost will give a more optimal layout with respect to cost, time and environmental impact. Different orientations of the tunnels may also be suitable in different parts of the repository. In many cases it may be evident which orientation and depth will yield the optimal layout. In some cases, however, more detailed analyses of various alternatives may be required.

The following working procedure is proposed in order to achieve an optimum layout design with respect to cost, time and environmental impact (excavated volume):

1. Analysis of the loss of deposition holes for various orientations of the deposition tunnels, angles of intersection between main and deposition tunnels and depths.
2. Design of preliminary layouts for various orientations and depths including calculation of excavated rock volume.
3. Estimation of potential stability problems, principles of tunnel excavation, rock support and grouting for different preliminary layouts.
4. Determination of tunnel orientation and depth.
5. Final layout.

The choice of tunnel orientation and depth can then be made based on both the optimum utilisation of rock with respect to the number and length of tunnels and the requirement for rock support and grouting.

The work procedure indicated above was employed to a certain extent in design step D1, since several tunnel orientations were analysed when calculating the loss of deposition holes. However, it may be preferable to include this working procedure in the design premises for forthcoming design steps since optimisation of the layout will become of greater concern.

It could be stated that the design team may use the above suggested design procedure, also when using the design premises in /SKB 2004a/. There is, however, a risk that a detailed specification of premises like in /SKB 2004a/, will result in a design that too strictly is based on the given premises, and that other issues may not be sufficiently considered. This risk will probably be higher if the time schedule for design is short.

A general risk with regard to the requirements and criteria given in the design premises /SKB 2004a/ is that they influence the layout to varying degrees. The respect distance and criteria for the loss of deposition holes due to spalling and stochastically generated fractures are of particular concern with regard to the layout. Forthcoming estimation of grouting requirements will also be affected if limitations are specified for the maximum allowable water ingress for various parts of the facility. Requirements and criteria should therefore be specified with great care.

10.5 Conclusions

Conclusions from the risk assessment are:

- There is as high as a 20% probability that the deep repository can not be accommodated within the “preferable repository area”, which was defined in Chapter 3. However, it must be emphasized that this risk assessment only involved the area within the defined “preferable repository area”, and consequently did not consider the possible available additional area for deposition underneath the sea and the nuclear plant. If these areas also would be utilised, the additional available canister capacity would be 1,000–1,100 positions. This number of canister positions is of the same order as the number of available canister positions needed to have a probability of 99% that the repository will be accommodated within the defined “priority site”. Furthermore, it should be noted that there is further potential to accommodate canisters south of the deformation zone ZFMNE00A2. The effect of the variation of parameters may also be different if the number and location of main tunnels are changed with respect to new conditions due to the variation, and not being fixed as in the executed analysis. The result of the probabilistic risk assessment is thus probably somewhat conservative. Taking into account the excessive area within the “priority site” as well as the uncertainties in the probabilistic analysis it is concluded that it is very likely that the repository indeed could be accommodated within the Forsmark site.
- Other parameters, such as variations in the location of deformation zones and the presence of not yet identified deformation zones longer than 3 km, would probably affect the result of the analysis regarding the possibility of the site to accommodate the repository. The parameters and their variation should thus be updated as the knowledge of the site conditions improves.
- A repository depth of 400 m is probably favourable with respect to the possibility to accommodate the repository, since a longer distance between the deposition area and the deformation zone ZFMNE1193 is achieved compared to a depth of 500 m.
- The length, location and orientation of the deformation zones ZFMNE0060 and ZFMNE00A2 should be investigated further in order to decrease the uncertainty.
- The effects of crossing the deformation zone ZFMNE00A2 and how many additional canisters may be accommodated southeast of the zone should be investigated.
- Due to probable stability problems that will be encountered if higher rock stresses than presented in /SKB 2005a/ are prevailing, investigations should be performed in order to increase the confidence in understanding of the state of stress at the actual facility depth. The investigations should be performed at different locations of the “priority site”, since the stresses may vary geographically. If the maximum horizontal stress is in the same order as presented in the analysis presented in Section 10.3.1, it is likely that the loss of deposition holes will be much higher. In the worst scenario, with the given rock mass spalling strength (σ_{sm}), spalling may occur in all deposition holes. It may be however be possible that some volume of breakout can be accepted in the deposition holes and that the loss of deposition holes thus will be less than if the volume of breakout is not considered. For comparison, a wedge breakout corresponding to a volume < 0.15 m^3 is accepted in /SKB 2004a/.
- The possibility of a shorter distance between deposition holes than 6 m should be evaluated. This investigation should contain a thermo-mechanical analysis.
- Grouting and rock support need (with respect to spalling), conditions for passing the shallow sub-horizontal fracture zones and the total salinity (TDS) around the repository should be further investigated in the forthcoming design steps or in separate studies.

- A strategy should be developed for the excavation and investigation sequence during the construction phase, enabling flexibility in the layout based on the underground conditions encountered.

In order to increase the area available for deposition and the probability for the Forsmark site to contain 6,000 canisters, it is recommended to perform some additional layout studies in the areas described below. As a base for these studies additional geological information from the areas in concern are probably needed.

- The area below the nuclear power plant, defined as the area northwest of the inlet canal for cooling water.
- The area below the sea, further out from the shore line.
- The area available southeast of the deformation zone ZFMNE00A2.

Another important issue that will influence the possibility to accommodate the repository is the maximum allowed length of deposition tunnels. For example, with longer tunnels the influence of changes in dip of deterministically determined deformation zones will be reduced. If longer deposition tunnels can be used the deposition area will also be utilized more effectively, the excavated volume will be reduced, and the number of concrete plugs will be less.

During the execution of the design work, which is presented in this report, a need for an evaluation of design methods was also identified. For example, the methodology and criteria for the analysis of spalling around deposition holes and the analysis of loss of deposition holes due to stochastically generated fractures/fracture zones need to be further evaluated.

11 Conclusions

11.1 Outcome of design task

11.1.1 Layout

Prerequisites for the layouts were evaluated in Chapters 3–4. In Table 11-1 a summary of theses prerequisites are given.

The layout studies, presented in Chapter 5, resulted in four proposed layouts of the repository:

Layout 1 (base layout): 400 m depth. The surface facility is located within the so called residential area (central area with skip and a spiral access ramp). This layout was considered as the base layout.

Layout 2: 400 m depth. One part of the surface facility is located within the so called residential area and the other part is located close to the office building of the SFR-facility (central area without skip and with a modified ramp to the SFR-facility).

Layout 3: 500 m depth. The surface facility is located within the so called residential area (central area with skip and a spiral access ramp).

Layout 4: 500 m depth. One part of the surface facility is located within the so called residential area and the other part is located close to the office building of the SFR-facility (central area without skip and with a modified ramp to the SFR-facility).

Based on the layout studies it was concluded that the differences in location, layout and excavated volume were small between the four proposed alternative layouts. In Table 11-2 some key data for Layout 1 (base layout) is presented.

Table 11-1. Summary of prerequisites for the layout studies evaluated in Chapters 3–4.

Prerequisite	Description of prerequisite	Chapter/section in report
Location of the repository	The repository should be located within the defined "priority site", northwest of the deformation zone ZFMNE00A2, southeast of the inlet canal for cooling water at the nuclear power plant, and not too far out from the shore line.	3
Depth of repository	The repository should be located between 400–500 m depth.	3 and 4.6
Distance between deposition holes and between deposition tunnels.	The distance should be 40 m between deposition tunnels and 6 m between deposition holes	4.3
Orientation of deposition tunnels	Deposition tunnels should be parallel to the maximum horizontal stress, NW/SE (140°) (due to the risk of spalling)	4.4
Loss of deposition holes	9% (due to stochastically determined fractures)	4.5

Table 11-2. Summary of key data for Layout 1 (base layout).

Key data	Value
Enclosed area used for deposition (m^2)	2,730,000
Total length of main tunnels (m)	5,030
Total length of transport tunnels in the deposition area (m)	2,676
Number of deposition tunnels	187
Total length of deposition tunnels (m)	47,503
Number of canister positions excluding loss of deposition holes	6,824
Number of canister positions, allowing for 9% loss of deposition holes	6,210
Excavated volume including central area, ramp and shafts but excluding deposition holes (m^3)	1,975,000

11.1.2 Hydrogeological results and rock support systems

Seepage and hydrogeology

The analytical and numerical calculations performed clearly demonstrated that the very tight rock at the repository level will result in favourable conditions for the repository with low seepage rates. It also resulted in a very limited influence on the hydraulic situation around the repository, and no influence on the groundwater table close to the ground surface.

The total inflow to the repository is expected to be approximately 2–8 l/s for an open repository, depending on the achieved sealing level, and the distance of influence is estimated to be in the range of 100 m to a few hundreds of meters.

No upconing of saline water is discernible in the numerical modelling, whereas the analytical simulations indicate a high probability of saline water migration from below leading to an increase of TDS in the seepage water. The discrepancy between the tools used is probably attributable to the analytical model that has been developed to avoid an increase in salinity in coastal wells.

Grouting

Due to the low hydraulic conductivity of the rock mass at repository level, it is expected that grouting will be executed mainly in the sub-horizontal zones near the surface during excavation of the access ramp and shafts. These zones are expected to have a high hydraulic conductivity and grouting will be needed in order to achieve an acceptable level of leakage into the shafts and the ramp. Continuous probe drilling ahead of the tunnel face should be carried out during excavation of the ramp in order to locate where these zones intersect the ramp, enabling suitable sealing and support measures to be determined. The grouting at repository level will most likely be limited to some minor deformation zones.

A summary of the grout quantity is presented in Table 11-3. Due to the low hydraulic conductivity of the rock mass, the grout quantity for sealing level 1 corresponds only to grouting of the subhorizontal, water bearing fracture zones at the depth 0–200 m. Grouting in a rock mass with a low hydraulic conductivity, will also result in the fact that a significant part of the grout quantity given in Table 11-3 corresponds to filling of grout holes.

Table 11-3. Quantity of grout including hole filling for different sealing levels. For sealing level 2 ($K_t = 10^{-9}$ m/s) it is assumed that sub horizontal zones will have a sealing level of only 10^{-8} m/s. The grout quantity for sealing level 1 corresponds to grouting of the subhorizontal zones only.

	Quantity (m^3)
Total grout quantity for the repository Sealing level 1: $K_t = 1\times10^{-7}$ m/s	60 to 210
Total grout quantity for the repository Sealing level 2: $K_t = 1\times10^{-9}$ m/s	620 to 1,550
Grout quantity in deposition tunnels Sealing level 2: $K_t = 1\times10^{-9}$ m/s	460 to 1,120

Rock support

The rock quality is generally very good and no major stability problems are expected. The rock support is installed primarily to ensure that no isolated blocks or smaller pieces of rock fall out. Most of the rock reinforcement will be installed as minimum support, including spot bolting and a 50 mm thickness of steel fibre reinforced shotcrete at the roof. This rock support will be installed irrespective of the rock quality. The reported levels of rock stress are not expected to give rise to major problems with spalling. Since variations in the stress levels are likely and/or the strength of the rock mass may be lower in some locations, some local minor spalling may occur. A certain amount of spalling may be expected in the lower parts of the vertical shafts, the transition between the deformation zones and the surrounding rock and at intersections between main tunnels and deposition tunnels. Supplementary rock bolting and shotcrete are considered appropriate to minimise any problems due to minor spalling. A further investigation of the rock support need for the shafts should be undertaken. The extent of reinforcement in the shafts may for example need to be increased where sensitive installations are present.

Supplementary reinforcement in the ramp may also be required where it intersects subhorizontal near-surface fracture zones owing to an unfavourable orientation of the ramp in relation to these zones.

A summary of the rock support quantity is presented in Table 11-4. In the deposition tunnels it is estimated that only rock bolts will be needed for rock support. If supplementary support is needed in the deposition tunnels, mesh must be installed since shotcrete is not accepted in the deposition tunnels.

Table 11-4. Quantity of rock support, Layout 1 (base layout), approximate values.

Item	Quantity
Total number of rock bolts for the repository	49,000 to 69,000
Total shotcrete area for the repository (m^2)	190,000
Number of rock bolts in deposition tunnels	18,000 to 30,000

11.1.3 Technical risk assessment

The main objective with design step D1 is to determine whether the deep repository can be accommodated within the studied site. After assessing the technical risks the following conclusions can be drawn:

- It is likely that the repository can be accommodated within the Forsmark site at the depth 400–500 m. The probability that the repository can be accommodated within the defined “priority stite” is estimated to be approximately 99%. If the repository can not be located closer to and below the nuclear power plant or further out from the shore line than specified in the proposed layouts, there is however, a risk that the repository can not be accommodated within the proposed “priority site”, northwest of the deformation zone ZFMNE00A2. More detailed studies are thus needed in order to increase the confidence in the conclusion that the repository can be accommodated within the specified “priority site”.
- Due to probable stability problems that will be encountered if higher rock stresses than presented in /SKB 2005a/ are prevailing, investigations should be performed in order to increase the confidence in understanding of the state of stress at the actual facility depth. If the maximum horizontal stress is in the same order as presented in the analysis presented in Section 10.3.1, it is likely that the loss of deposition holes will be much higher than the loss used for the layout studies.

11.2 Critical issues

Higher rock stresses than those currently estimated (see /SKB 2005a/) will imply systematic breakout of rock into the deposition holes due to spalling. Until the acceptable volume of breakout into the deposition holes is defined by SKB, it could be assumed that a high probability of spalling in the deposition holes will result in a high loss of deposition holes.

This critical issue is mainly related to the possibility of accommodating the repository and the excavated volume for the repository. Other important issues regarding the hydrogeological situation, grouting- and rock support need and which should be considered in the forthcoming design are presented in Section 10.3.2.

11.3 Recommendations

Feedback for forthcoming design, investigation/modelling and safety assessment are described in the following section. Some feedback may be of interest for all the activities – design, investigation/modelling and safety assessment – implying that all the recommendations below should be considered.

11.3.1 Feedback to design

It is recommended that the following issues with regard to design should be further dealt with:

- Studies regarding the loss of deposition holes due to possible higher rock stresses should be executed. These studies should include investigations in order to assess the maximum acceptable volume of breakout in deposition holes. For comparison, a wedge breakout corresponding to a volume < 0.15 m^3 is accepted in /SKB 2004a/. It should be noted that despite a possible acceptable volume of breakout of 0.15 m^3, a conservative estimate of the magnitude of the major horizontal stress would probably result in a higher loss of deposition holes than used for the layout studies. In the worst scenario identified in the technical risk assessment (see Section 10.3.1), spalling may occur in all deposition holes.

- In design step D1 the reference distance between deposition holes of 6.0 m has been used as a prerequisite for the layout studies. In the forthcoming design steps it should be further investigated if a distance of approximately 5.5 m between the deposition holes or a shorter distance between the deposition tunnels could be used. This investigation should contain a thermo-mechanical analysis. It may also be possible to allow for 10°C higher temperature when evaluating the canister spacing according to Figure 5-4 in /SKB 2004a/ (see Section 2.3.2).
- An important issue that will influence the possibility to accommodate the repository is the maximum allowed length of deposition tunnels. For example, with longer tunnels the influence of changes in dip of deterministically determined deformation zones will be reduced. Presented layouts are based on deposition tunnels with a length of 100 to 300 m. The possibility of using longer deposition tunnels should be further investigated. With longer tunnels a more effective use of a specified rock volume will be achieved, the number of concrete plugs will be reduced as will the construction cost.
- The possibility to locate facility parts of the deep repository closer to and below the nuclear power plant and further out from the shore line in the eastern part of the area needs to be analysed. This possibility will add a substantial area for deposition of canisters northwest of the deformation zone ZFMNE00A2.
- In the presented design no canisters are proposed to be deposited southeast of the deformation zone ZFMNE00A2. Available area for deposition of canisters southeast of the deformation zone ZFMNE00A2 should be evaluated together with conditions for excavation, rock support and grouting when passing the deformation zone.
- The hydrogeological situation around the repository should be further studied. The studies should include an analysis of the extension of the low conductive rock volume, lowering of the ground water table (drawdown) and the probability of up-coning of saline water.
- As a base for assessment of sealing levels, acceptable inflow of ground water should be assessed for different facility parts.
- If high demands are specified with respect to sealing levels, the possibility of using others types of grouts, besides cement based grouts, should be investigated further.
- Possible sealing levels should be estimated for the sub-horizontal fracture zones at 0–200 m depth. Grouting tests may for this reason be needed. The link between shaft construction methods and the feasibility for achieving an acceptable sealing level should also be considered in future planning.
- Available and recommended methods for the calculation of grout take include large uncertainties. Principles and methods for calculation of grout take must be evaluated further in order to reduce the uncertainty in calculated quantities of grout take.
- The needs for a minimum rock support, basically for the safety of personnel, should be evaluated further and include a proposal for a suitable support system.
- Problems with spalling may occur locally, for example in the lower parts of the ramp and the shafts, in the transition between the deformation zones and the surrounding rock and at the intersections of main- and deposition tunnels. The risk of spalling, need for rock support and strategies for tunnel excavation should be studied further.

11.3.2 Feedback to investigation/modelling

In order to improve input data for the proposed study work, the following issues should be further investigated:

- Location, orientation, length and properties of deformation zones (especially ZFMNE0060 and ZFMNE00A2).
- The geological conditions closer to and below the nuclear power plant, and further out from the shore line in the eastern part of the area.
- Location, orientation and properties of sub-horizontal fracture zones in the area of the ramp and shafts.
- The existence of possible deformation zones northwest of the deformation zone ZFMNE0061.
- Hydraulic properties for different parts of the rock mass.
- Magnitude and orientation of in-situ stresses at repository depth. The investigations should be made at different locations within the "priority site", since the stresses may vary geographically.
- Rock mechanical and hydraulic properties in rock domains close to rock domain RFM029 (such as rock domain RFM018 and RFM032) should be further investigated. It may prove necessary to locate transport tunnels outside rock domain RFM029.

11.3.3 Feedback to safety assessment

It is preferable that the following issues are addressed within the R&D programme or the safety assessment:

- The respect distances/margins for construction will influence the number of available canister positions in the repository. If possibilities to formulate less restrictive criteria through further studies can be foreseen, such studies should preferably be undertaken.
- The methodology and criteria for calculation of the loss of deposition holes due to stochastically generated fractures will influence the result of the calculations and thus the need of area available for deposition. Further more, the criteria should be related to the possibility of finding the discriminating fractures when constructing the repository.

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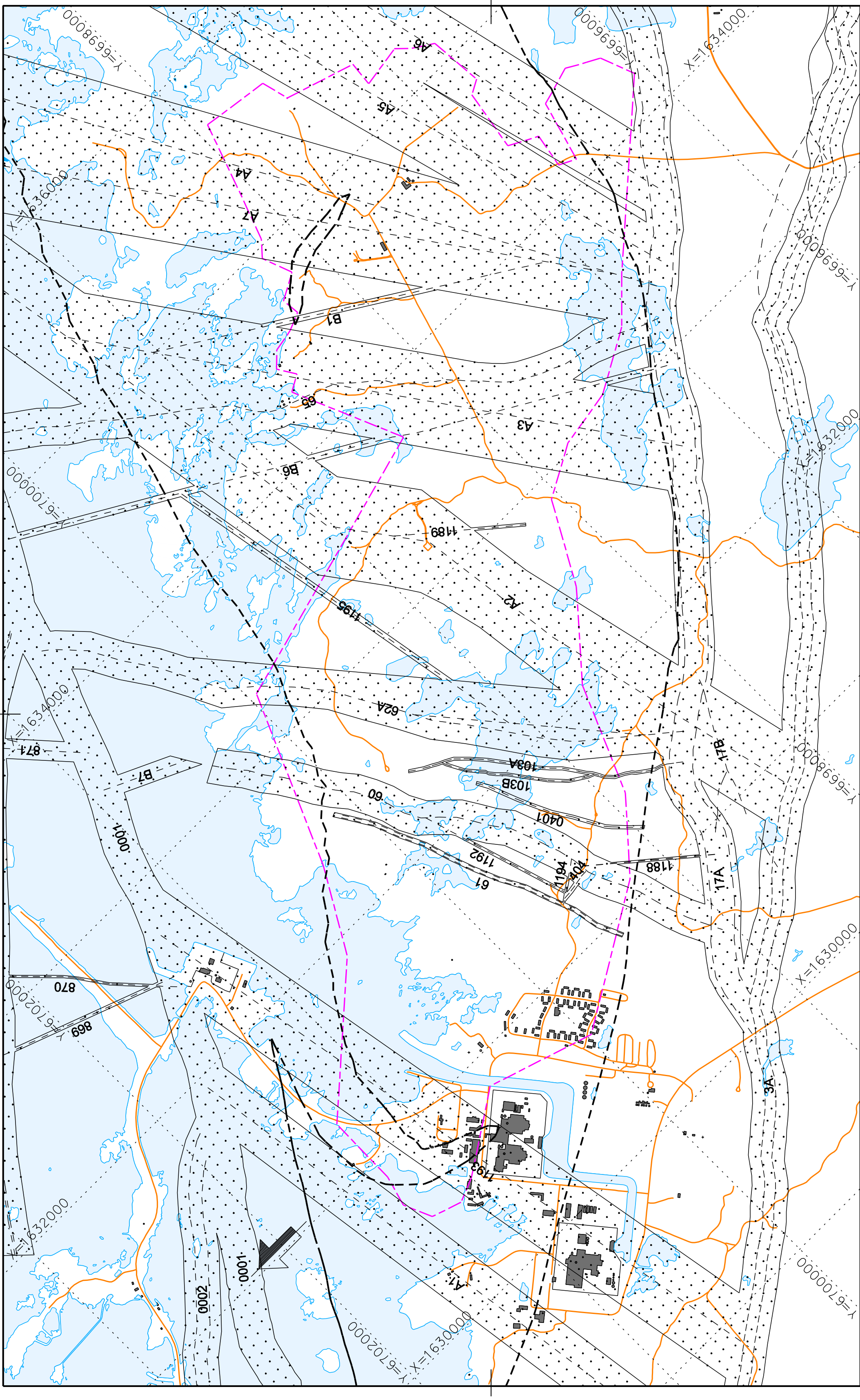
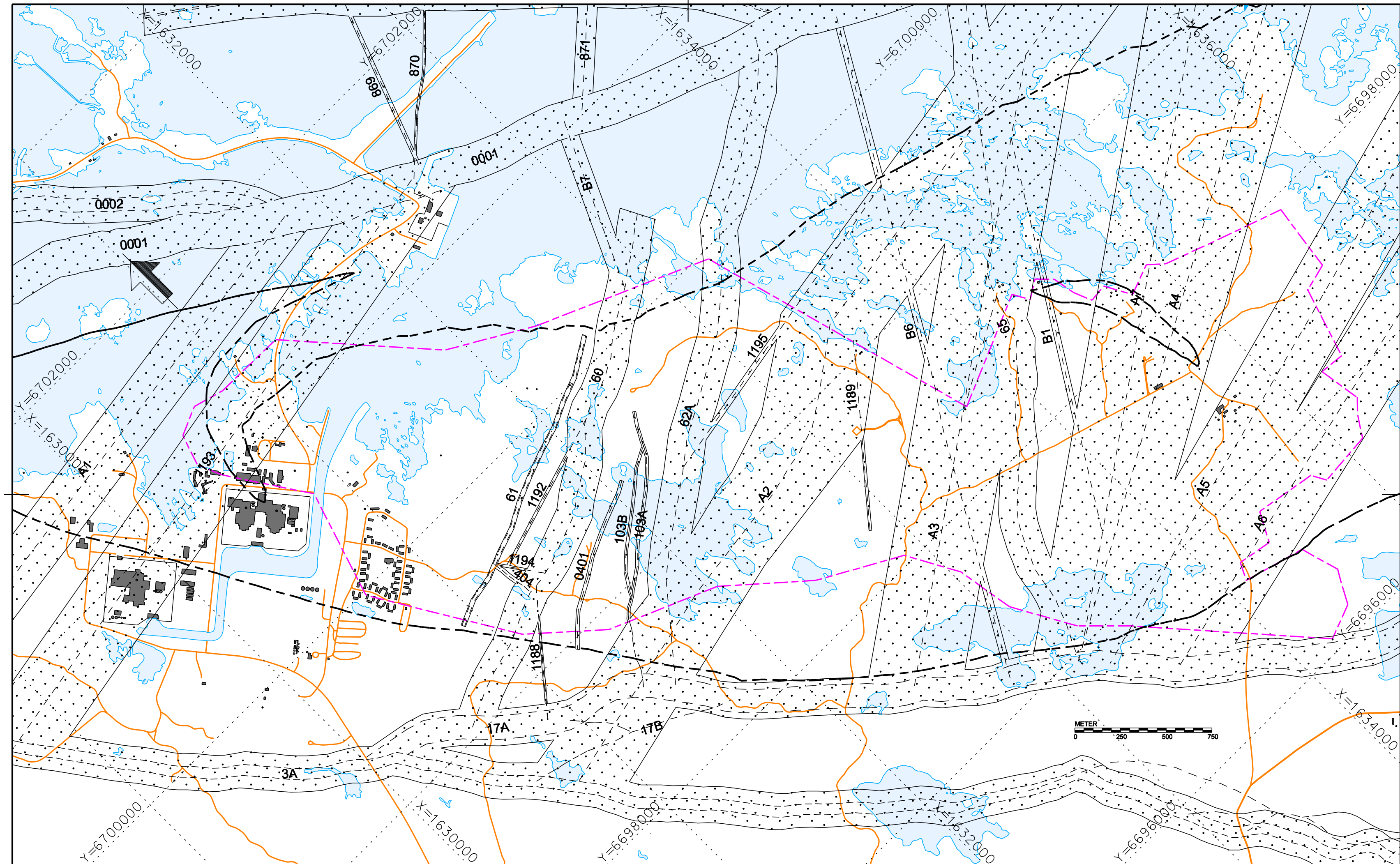
Appendix A

Preliminary respect distances



Appendix A: Deformation zones, preliminary respect distances and margins for construction. 500 m level.



LEGEND

CANDIDATE AREA

ROCK DOMAIN 29

DEFORMATION ZONES AND RESPECT DISTANCES

Appendix A: Deformation zones, preliminary respect distances and margins for constructions. 400 m level.

Appendix B

Basis for calculation of available area

The basis for the calculation of available area, A_T, which is described in Chapter 3.3, is shown in Figure B1 and B2.

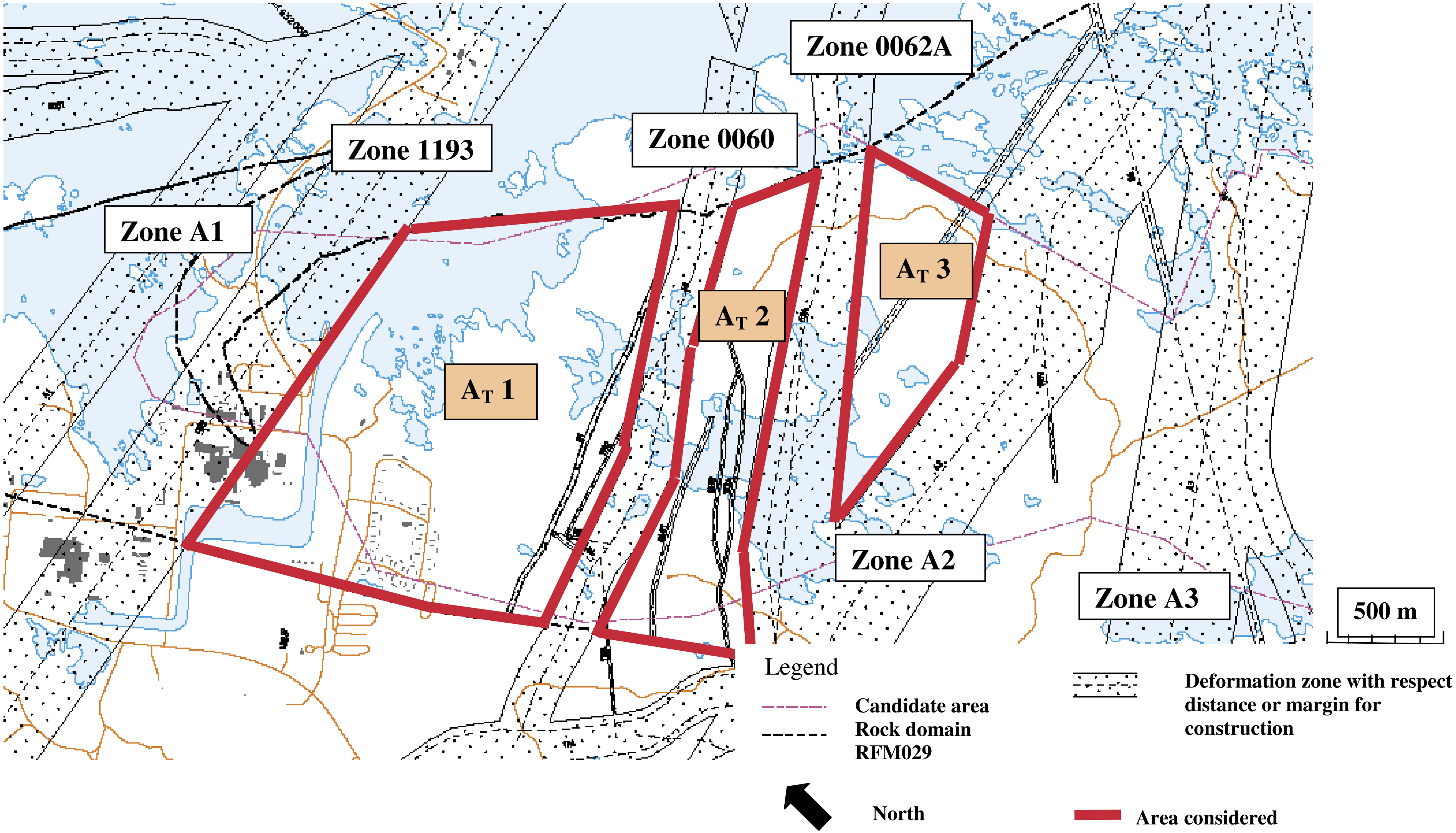


Figure B1. *Basis for calculation of available area, AT, 500 m depth.*

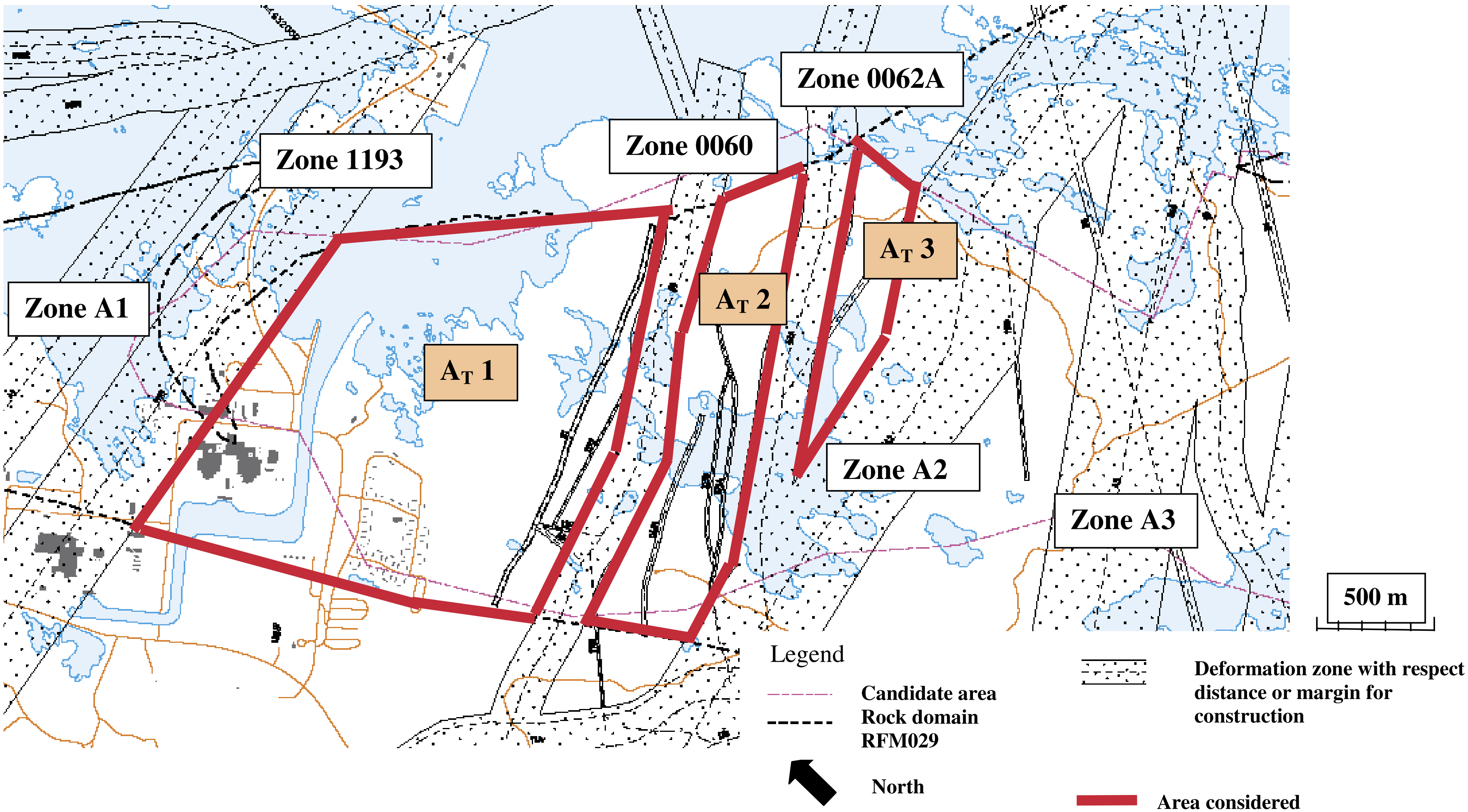
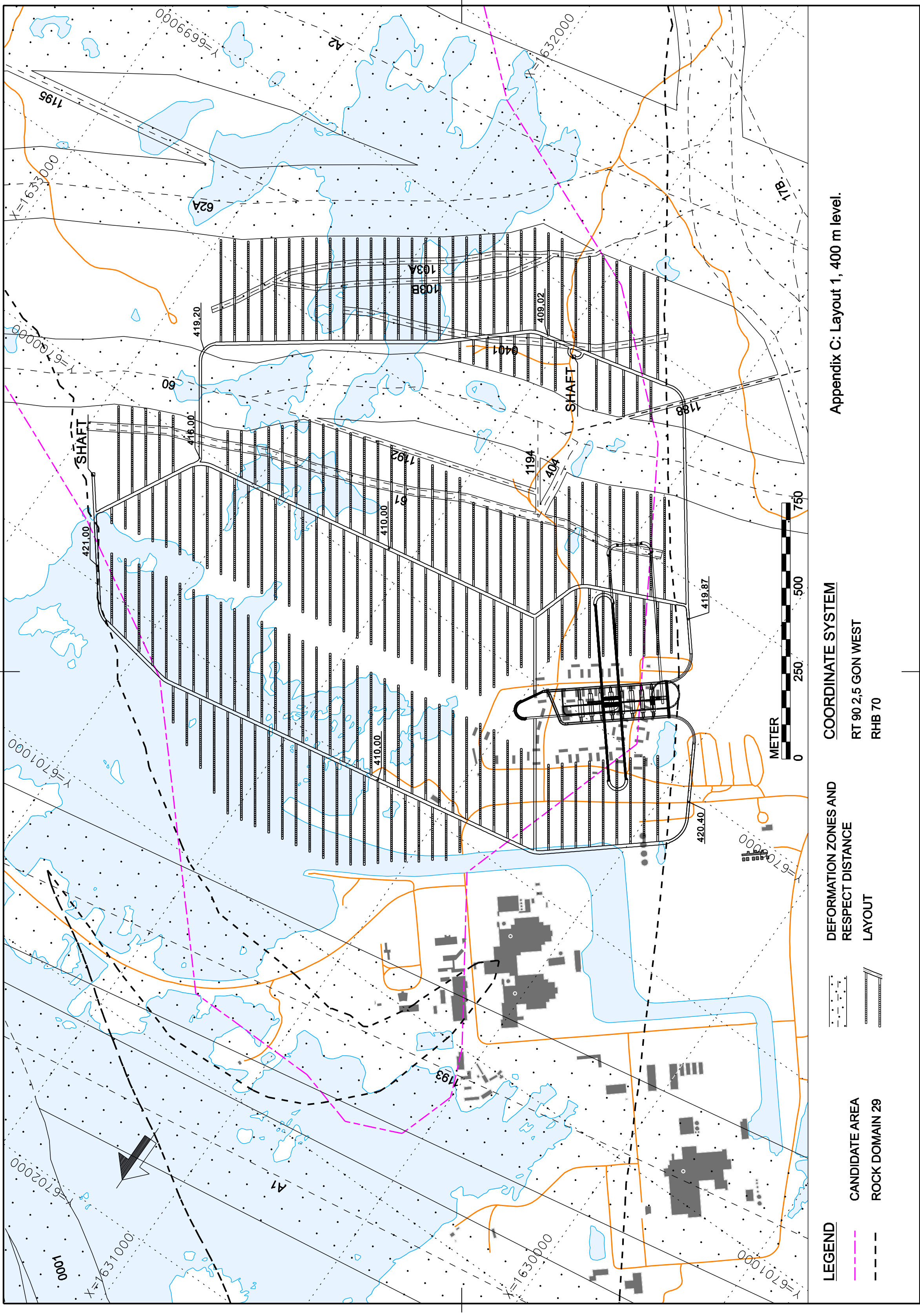


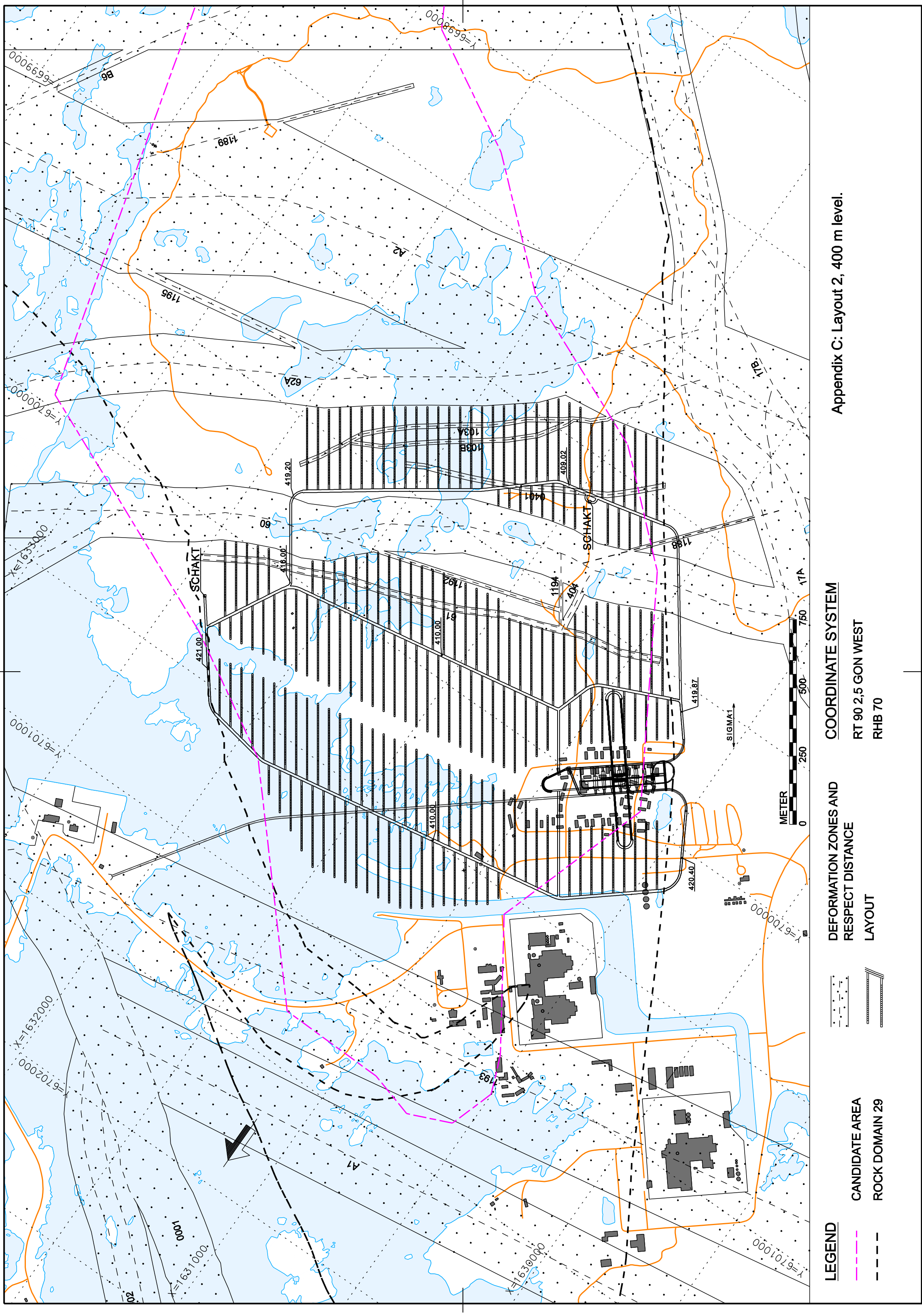
Figure B2. *Basis for calculation of available area, AT, 400 m depth.*

Appendix C

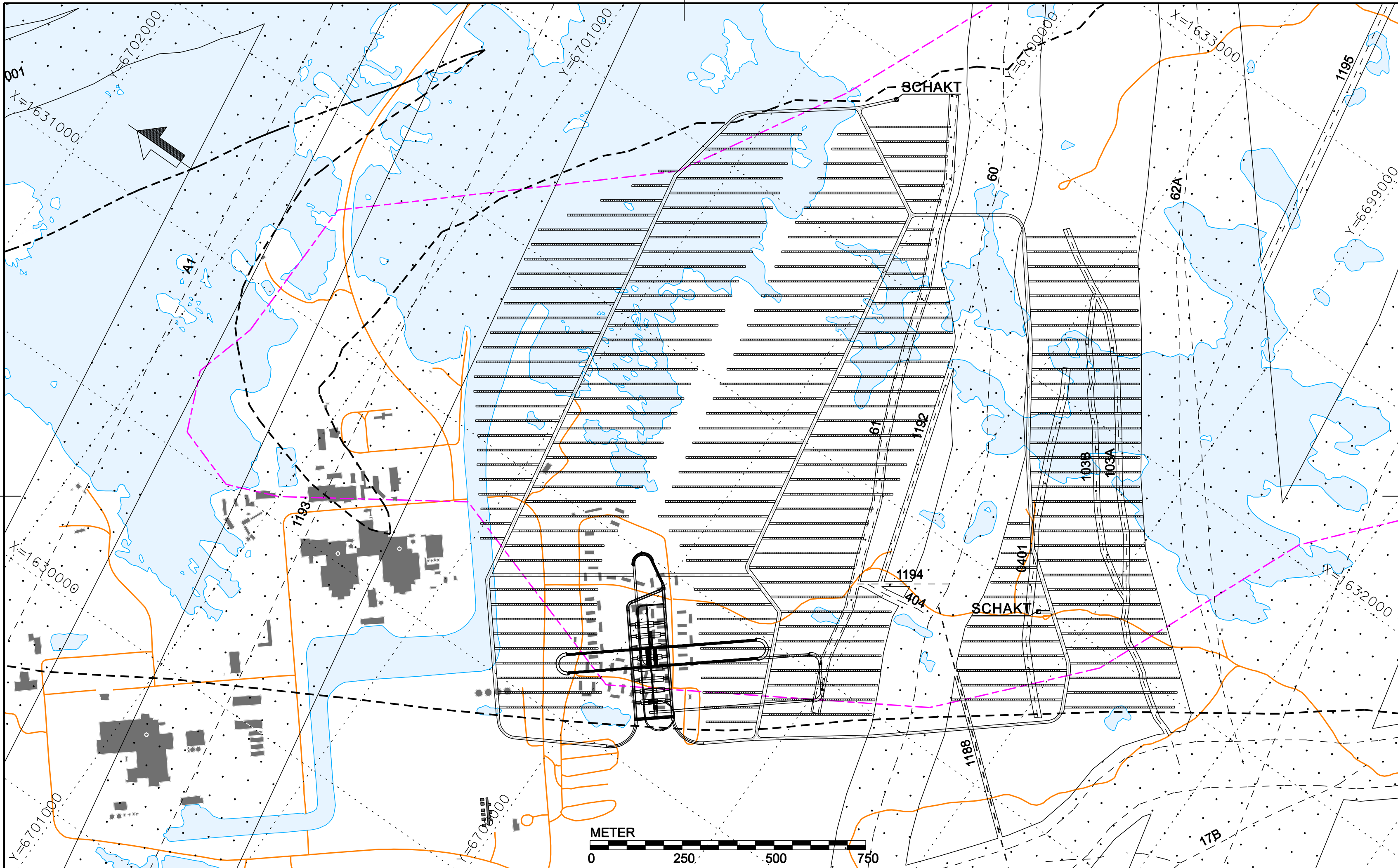
Possible layouts



Appendix C: Layout 1, 400 m level.



Appendix C: Layout 2, 400 m level.



LEGEND

CANDIDATE AREA

ROCK DOMAIN 29

DEFORMATION ZONES AND RESPECT DISTANCE

LAYOUT

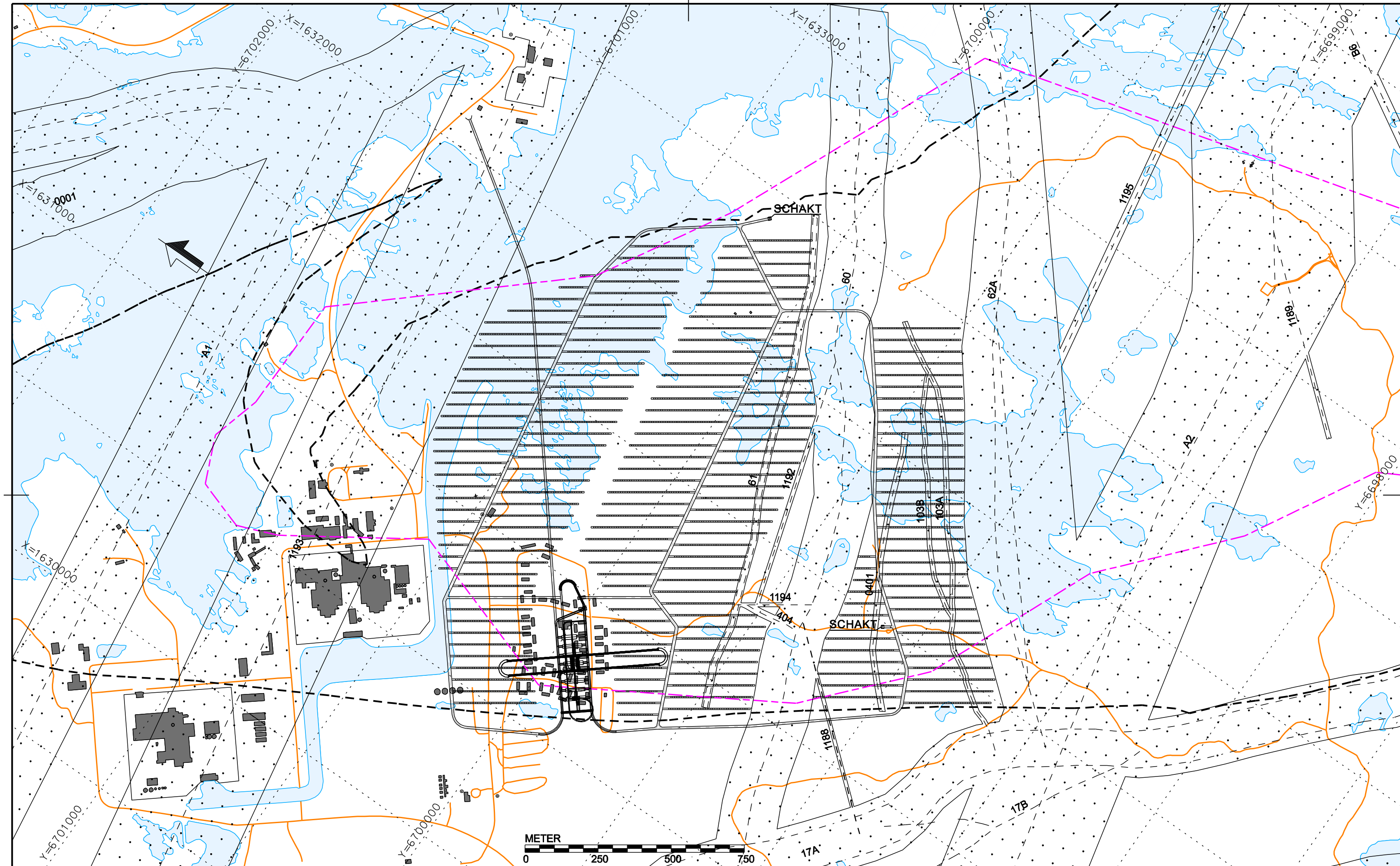
COORDINATE SYSTEM

RT 90 2,5 GON WEST

RHB 70

Appendix C: Layout 3, 500 m level.

RITNINGSFIL:I:\27BRG\2704\11798\B_PROJ\RIT\B13014000 5_ENGELSK.DGN
PLOTDATUM:2006-01-05
PLOTTAD AV:ZETTERQVIST ANDERS



LEGEND

CANDIDATE AREA

ROCK DOMAIN 29

DEFORMATION ZONES AND RESPECT DISTANCE

LAYOUT

COORDINATE SYSTEM

RT 90 2,5 GON WEST

RHB 70

Appendix C: Layout 4, 500 level

Appendix D

Estimation of grout quantity

D.1 Estimation of grout take in a single grout hole

The estimation of quantities is based on volumes in single grout holes in accordance with /SKB 2004a/, Section 4.6 in Appendix 2. In /SKB 2004a/ it is stated that the volume in a grout hole should be analysed using the following analytical methods:

1. Calculation of grouted volume in a single grout hole based on the assumption that grout spreads in plane parallel fractures (Equation D-1).
2. Calculation of grouted volume in a single grout hole with the assumption that the porosity in rock mass is filled by grout to a distance, which correspond the estimated penetration length from the grout hole (Equation D-2 and D-3).

$$V = \left(\frac{\Delta p}{2 \cdot \tau_0}\right)^2 \cdot \frac{12 \cdot K_b \cdot L \cdot \mu_w \cdot \pi}{\rho_w \cdot g} \qquad \text{Equation D-1}$$

where

V = grouted volume (m^3)

Δp = grouting pressure (over ground water pressure) (Pa)

τ_0 = yield value of the grout (Pa)

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

L = length of grout hole (m)

ρ_w = specific weight of water (1,000 kg/m^3)

μ_w = viscosity of water (0.0013 Pas)

g = specific gravity (10 m/s^2)

It should be noted that Equation D-1 is associated to a number of uncertainties, since it does not consider some of the factors affecting the grout spread in the rock mass. These factors are for example the variations in aperture of the fractures, limitations in the penetration ability of the grout due to filtration, the hardening and bleed of the grout and finally the criteria normally used for stopping the grout. In /Eriksson 2002/, these factors are studied with respect to the effect on the calculation result and the conclusion is that the volume may vary with a factor between 1 and 100. How much the volume is affected depends both on the properties of the grout and fractures. Especially when the apertures are small the effects of different factors are evident. Calculated volumes should thus be used with caution.

It should also be noted that Equation D-1 only is valid for single grout holes. When grouting of several grout holes in a grout fan, the conductivity of the rock mass will more or less be reduced to the sealing effect of the grout.

The grout volume in a grout hole should also be calculated based on the assumption that the porosity in rock mass is filled by grout to a distance, which corresponds to the estimated penetration length from the grout hole. The grout volume is in this case calculated with Equation D-2.

$$V = I^2 \cdot \pi \cdot p \cdot L \qquad \text{Equation D-2}$$

where

V = grouted volume (m^3)

I = penetration length/grout spreading distance (m)

p = porosity of rock mass (–)

L = length of grout hole (m)

The penetration length may be assumed from engineering judgement and the porosity can according to /SKB 2004a/, be calculated with Equation D-3.

$$\log p = 0{,}17 \cdot \log K_b - 1{,}7 \pm 0{,}3 \qquad \text{Equation D-3}$$

where

p = porosity (–)

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

According to /SKB 2004a/, the volume should also be calculated based on the porosity from DFN-data (P33). However, no transmissivity values for the individual fractures were included in the delivery to the design team and consequently no P33-values could be calculated.

When calculating the grout volumes, the conductivity values for the different parts of the rock mass are used. The grout hole length is set to 20 m except for grouting in the sub-horizontal fracture zones (grouting classes 4–7). For these grouting classes the groutable length of the grout holes is set to 2 m, which corresponds to an assumption of the mean width of the subhorizontal zones.

Regarding the factors describing the grouting technique in Equation D-1 the following assumptions were made:

- The grouting pressure is set to 20 bars above ground water pressure (10 bars above ground water pressure when grouting subhorizontal zones).
- The yield values are set to values, which are regarded as normal for conventional grouts with water cement ratios according to Chapter 8.

For the calculation of grout take with Equation D-2 and D-3 an engineering judgement is made in order to estimate the grout penetration length/grout spreading distance. It should be emphasised that it is difficult to verify the penetration length of the grout in different geological conditions. When grouting water bearing zones at Äspö HRL, measurements were made, which indicated a penetration length of 5–15 m /Janson 1996/. Based on these findings the same penetration length (5–15 m) is assumed for the conductive sub-horizontal zones at the Forsmark area. For the other grout classes a penetration length of only 0.5–5 m is assumed, due to the low conductive rock mass and hence an estimated low grout take (also see /Emmelin et al. 2004/).

The correction term ±0.3 in Equation D-3 has been set to 0.

The grout volumes based on Equation D-1 is shown in Table D-1 and the volumes calculated with Equation D-2 and D-3 in Table D-2.

Table D-1. Volume of grout in a single grout hole calculated with Equation D-1. The volumes are excluding the volumes needed for filling the holes. The adjustments of volumes are done by dividing the calculated volumes with a factor set to 2 for wcr 0.5, 25 for wcr 1.0 and 50 for wcr 2.0.

Grouting class/ grouting round	Water cement ratio, wcr	Volume of grout, min (m^3)	Volume of grout, adjusted, min (m^3)	Volume of grout, max (m^3)	Volume of grout, adjusted, max (m^3)
1 / grouting round 1 2, 3, 5 7 / grouting round 2	2.0	1.0	0.02	3.9	0.08
2 and 3 / grouting round 1 4,6 / grouting round 2	1.0	24.5	1.0	98.0	3.9
4–7 / grouting round 1	0.5	1.1	0.5	2.4	1.2

Table D-2. Volume of grout in a single grout hole calculated with Equation 8-2 and 8-3. The volumes are excluding the volumes needed for filling the holes.

Grouting class/ grouting round	Water cement ratio, wcr	Volume of grout, min (m^3)	Volume of grout, max (m^3)
1 / round 1 2, 3, 5, 7 / round 2	2.0	0.01	0.49
2 and 3 / round 1 4,6 / round 2	1.0	0.1	3.0
4–7 / round 1	0.5	0.7	5.9

Due to the large uncertainties in the volumes calculated with Equation D-1, the calculated volumes are also shown after dividing the calculated volumes with a factor of 2–50. The factors are given values depending on the type of fractures that will be grouted, where grout volumes in fine fractures are assumed to be associated with the largest uncertainty. Since the water cement ratio, wcr, can be assumed to be higher when grouting fine fractures, the factors are given the value 2 for wcr 0.5, 25 for wcr 1.0 and 50 for wcr 2.0. It should be noted that these values are not given in /Eriksson 2002/, and thus only represent a proposed value given by the designer. Further analyses of this issue should maybe be motivated in later design steps or separate studies.

D.2 Estimation of grout take in a grout fan

In the following chapter the grout take in a grout fan is estimated. This grout take is based on the calculated volumes in single grout holes (see Section D.1) and experiences from grouting works.

Due to the unreasonable high volumes calculated with Equation D-1, the adjusted volumes according to Table D-1 and volumes shown in Table D-2 have been used for the assumption of grout volumes in a grout fan. Due to an estimated short penetration length (except in the sub-horizontal zones) these adjusted volumes are assumed to be likely for all grout holes in a grout fan in grouting classes 1–3. A small overestimation of the amount of grout will maybe be the case.

Regarding the grouting in the sub-horizontal zones (grouting classes 4–7) a longer penetration length/spreading distance is foreseen and thus a high degree of filling of fractures in the zone is estimated after grouting in the first grout hole. The estimation of quantities for the sub-horizontal zones is therefore based on the following assumptions:

- One grout fan is needed when grouting in shafts.
- Five grout fans are needed when grouting in the ramp.
- Two grouting rounds are executed in each grout fan.
- The grout volume in grouting round 1 is set two 2 times the assumed volume for a single grout hole
- The grout volume in grouting round 2 is set to 10% of the volume in grouting round 1.

Table D-3 and Table D-4 show the grout volumes which are used as a basis for the estimation of total quantities for the facility parts.

Table D-3. Grout volume in a single grout hole used in the estimation of quantities for the facility parts, grouting classes 1–3.

Grouting class/ grouting round	Volume of grout, min (m^3)	Volume of grout, max (m^3)
1 / round 1 and 2 3 / round 2	0.05	0.25
2 / and 3 / round 1	0.1	1.0

Table D-4. Grout volumes assumed for grouting in the subhorizontal near surface zones (grouting classes 4–7). The volume is used for all sealing levels.

Facility part	Volume of grout, min (m^3)	Volume of grout, max (m^3)
Ramp (access tunnel)	4	50
Shaft (for each shaft)	2	10

D.3 Summary of total quantities in the hard rock facility

The estimation of quantities for the individual facility parts is based on the grouting classes given in Chapter 8 and grout volumes in Table D-3 and Table D-4. In order to take into account that all of the grout holes in a grout fan are not groutable, it has been assumed that 40–70% of the grout holes only are filled with a volume, which correspond the volume of the hole. The grout for filling the holes is assumed to have a water cement ratio, wcr, 0.5.

Since grouting mainly is assumed to be needed in the deterministically determined deformation zones and the sub-horizontal fracture zones together with the fact that the differences in the different layouts (see Chapter 5) are relatively small, the estimation of quantities is only executed for one depth, 400 m.

Based on the distribution of conductivity values for the rock mass between the deformation zones (see Chapter 8) it is assumed that only 2% of the rock mass between the zones will be groutable.

Control holes are not included in the estimation of quantities.

The geometries for different facility parts are based on the different layouts presented in Chapter 5 and cross sections given by /SKB 2002/. In Table D-5 the geometries for extensive facility parts, which are used for the estimation of quantities, are shown.

The compositions of grout are based on proposals of grouts for different grouting classes. One composition of grout has also been supplied by SKB. It should be noted that SKB is previously working with the development of grouts, which have a pH < 11. No such compositions are available yet, which means that the compositions given by SKB should be regarded as preliminary. Table D-6 shows the composition of grouts used for the estimation of quantities.

According to SKB the low pH-grout should be used as an alternative grout. The estimation of quantities for the alternative grout has been made in such a way that the volumes calculated for the grouts proposed by designer also have been used an estimated volume for the low pH-grout. Finally the quantities of different materials have been calculated based on the compositions of grout given in Table D-6.

Important assumptions for the estimation of quantity of grout are summarized in the Table D-7.

Table D-5. Geometries for extensive facility parts, which are used for the estimation of quantities.

Facility part	Geometry	Value
Main tunnel	Roof	11.3 m
	Walls	10.4 m
	Length, layout 1 and 2	5,030 m
	Length, layout 3 and 4	4,923 m
	Number of intersections with deposition tunnels	115
Transport tunnels in deposition area	Roof	8.5 m
	Walls	10.4 m
	Length, layout 1 and 2	2,676 m
	Length, layout 3 and 4	2,869 m
Deposition tunnels	Roof	6.2 m
	Walls	8 m
	Length, layout 1 and 2	47,503 m
	Length, layout 3 and 4	48,618 m
	Number of concrete plugs, layout 1 and 2	187
	Number of concrete plugs, layout 3 and 4	190
Ramp	Roof	7.2 m
	Walls	8.4 m
	Length, layout 1 and 2	4,000 m
	Length, layout 3 and 4	5,000 m

Table D-6. Composition of grout, wcr = water cement ratio.

Type of grout	Material	Quantity kg/m³
Wcr 0.5	Cement	1,198
	Plastisizer (SP40)	24
	Water	599
Wcr 1.0	Cement	749
	Plastisizer (SP40)	15
	Water	749
Wcr 2.0	Cement	427
	Plastisizer (SP40)	8
	Water	854
Wcr 2.0 (stable grout for grouting class 7)	Low pH-grout given by SKB	Low pH-grout given by SKB
Low pH-grout given by SKB	Cement (Ultrafin 16)	299
	Silica slurry (Grout aid)	419
	Plastisizer (SP40)	11
	Water	599

Table D-7. Assumptions for the estimation of quantities (assumption made by the designer).

Issue	Assumption
Length of grout holes	20 m
Length of grout holes when grouting the shafts from the ground surface	100 m
Diameter of grout holes	64 mm
Overlap between grouting fans	5 m
Part of rock mass between deterministically determined deformation zones that is grouted	2%
Number of grout holes in deposition tunnels, one grouting round	20
Number of grout holes in main tunnels, one grouting round	35
Number of grout holes in transport tunnels in deposition area , one grouting round	30
Number of grout holes in ramp, one grouting round	25
Number of ungroutable holes in deposition tunnels	40%
Number of ungroutable holes in ramp and shafts	50%
Number of ungroutable holes in other facility parts	70%

Tables D-8 to D-15 provide summaries of quantities of grouting as follows:

- the entire deep repository,
- tunnels and rock caverns at deposition depth (excluding shafts and access ramp),
- deposition tunnels.

The summaries are provided for the different sealing levels and points in time (excavation stages).

D.3.1 Quantities – early point in time

Sealing level 10^{-9} m/s

Table D-8. Summary of grouting for the entire deep repository, sealing level 10^{-9} m/s, early point in time.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Grouting holes, nr, min	641	nr	0.10	nr/m
Grouting holes, nr, max	694	nr	0.11	nr/m
Grouting holes, m, min	13,464	m	2.12	m/m
Grouting holes ,m, max	14,515	m	2.28	m/m
Volume of grouting holes, min	43	m^3	0.01	m^3/m
Volume of grouting holes, max	47	m^3	0.01	m^3/m
Grout volume excluding volume in grout hole, min	18	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	109	m^3	0.02	m^3/m
Cement for grout excluding hole filling, min	9	ton	0.001	ton/m
Cement for grout excluding hole filling, max	56	ton	0.01	ton/m
Silica slurry for grout excluding hole filling, min	0.2	ton	0.00003	ton/m
Silica slurry for grout excluding , max	2	ton	0.00026	ton/m
SP40 for grout excluding hole filling, min	0.1	ton	0.00002	ton/m
SP40 for grout excluding hole filling, max	0.9	ton	0.00014	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	0.7	m^3	0.00011	m^3/m
Volume of grout for hole filling, min	43	m^3	0.01	m^3/m
Volume of grout for hole filling, max	47	m^3	0.01	m^3/m
Cement for hole filling, min	34	ton	0.005	ton/m
Cement for hole filling, max	36	ton	0.01	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	0.7	ton	0.00010	ton/m
SP40 for hole filling, max	0.7	ton	0.00011	ton/m
SP40 for hole filling, min	0.5	m^3	0.00008	m^3/m
SP40 for hole filling, max	0.6	m^3	0.00009	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	18	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	109	m^3	0.02	m^3/m
Cement for grout excluding hole filling, min	5	ton	0.001	ton/m
Cement for grout excluding hole filling, max	32	ton	0.01	ton/m
Silica slurry for grout excluding hole filling, min	8	ton	0.001	ton/m
Silica slurry for grout excluding , max	46	ton	0.01	ton/m
SP40 for grout excluding hole filling, min	0.2	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	1.2	ton	0.00019	ton/m
SP40 for grout excluding hole filling, min	0.2	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	0.9	m^3	0.00014	m3/m
Volume of grout for hole filling, min	43	m^3	0.01	m^3/m
Volume of grout for hole filling, max	47	m^3	0.01	m^3/m
Cement for hole filling, min	13	ton	0.002	ton/m
Cement for hole filling, max	32	ton	0.005	ton/m
Silica slurry for hole filling, min	18	ton	0.0029	ton/m
Silica slurry for hole filling, max	20	ton	0.0031	ton/m
SP40 for hole filling, min	0.5	ton	0.00007	ton/m
SP40 for hole filling, max	0.5	ton	0.00008	ton/m
SP40 for hole filling, min	0.4	m^3	0.00006	m^3/m
SP40 for hole filling, max	0.4	m^3	0.00006	m^3/m

Table D-9. Summary of grouting at repository depth, sealing level 10^{-9} m/s, early point in time.

Item	Quantity	Unit	Per metre of tunnel and rock cavern at repository depth	Unit
Grouting holes, nr, min	581	nr	0.10	nr/m
Grouting holes, nr, max	633	nr	0.10	nr/m
Grouting holes, m, min	11,611	m	1.91	m/m
Grouting holes ,m, max	12,662	m	2.09	m/m
Volume of grouting holes, min	37	m^3	0.01	m^3/m
Volume of grouting holes, max	41	m^3	0.01	m^3/m
Grout volume excluding volume in grout hole, min	16	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	88	m^3	0.01	m^3/m
Cement for grout excluding hole filling, min	7	ton	0.001	ton/m
Cement for grout excluding hole filling, max	38	ton	0.01	ton/m
Silica slurry for grout excluding hole filling, min	0	ton	0.00	ton/m
Silica slurry for grout excluding , max	0	ton	0.00	ton/m
SP40 for grout excluding hole filling, min	0.1	ton	0.00002	ton/m
SP40 for grout excluding hole filling, max	0.7	ton	0.00012	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	0.5	m^3	0.00009	m^3/m
Volume of grout for hole filling, min	37	m^3	0.01	m^3/m
Volume of grout for hole filling, max	41	m^3	0.01	m^3/m
Cement for hole filling, min	29	ton	0.005	ton/m
Cement for hole filling, max	32	ton	0.01	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	0.6	ton	0.00009	ton/m
SP40 for hole filling, max	0.6	ton	0.00010	ton/m
SP40 for hole filling, min	0.4	m^3	0.00007	m^3/m
SP40 for hole filling, max	0.5	m^3	0.00008	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	16	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	88	m^3	0.015	m^3/m
Cement for grout excluding hole filling, min	5	ton	0.001	ton/m
Cement for grout excluding hole filling, max	26	ton	0.004	ton/m
Silica slurry for grout excluding hole filling, min	7	ton	0.001	ton/m
Silica slurry for grout excluding , max	37	ton	0.006	ton/m
SP40 for grout excluding hole filling, min	0.2	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	1.0	ton	0.00016	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	0.7	m^3	0.00012	m^3/m
Volume of grout for hole filling, min	37	m^3	0.01	m^3/m
Volume of grout for hole filling, max	41	m^3	0.01	m^3/m
Cement for hole filling, min	11	ton	0.002	ton/m
Cement for hole filling, max	26	ton	0.004	ton/m
Silica slurry for hole filling, min	16	ton	0.0026	ton/m
Silica slurry for hole filling, max	17	ton	0.0028	ton/m
SP40 for hole filling, min	0.4	ton	0.00007	ton/m
SP40 for hole filling, max	0.4	ton	0.00007	ton/m
SP40 for hole filling, min	0.3	m^3	0.00005	m^3/m
SP40 for hole filling, max	0.3	m^3	0.00006	m^3/m

Table D-10. Summary of grouting in deposition tunnels, sealing level 10^{-9} m/s, early point in time.

Item	Quantity	Unit	Per metre deposition tunnel	Unit
Grouting holes, nr, min	486	nr	0.10	nr/m
Grouting holes, nr, max	523	nr	0.11	nr/m
Grouting holes, m, min	9,723	m	2.05	m/m
Grouting holes ,m, max	10,454	m	2.20	m/m
Volume of grouting holes, min	31.3	m^3	0.0066	m^3/m
Volume of grouting holes, max	33.6	m^3	0.0071	m^3/m
Grout volume excluding volume in grout hole, min	15	m^3	0.0031	m^3/m
Grout volume excluding volume in grout hole, max	78	m^3	0.0165	m^3/m
Cement for grout excluding hole filling, min	6	ton	0.0013	ton/m
Cement for grout excluding hole filling, max	33	ton	0.0070	ton/m
Silica slurry for grout excluding hole filling, min	0.0	ton	0.00000	ton/m
Silica slurry for grout excluding , max	0.0	ton	0.00000	ton/m
SP40 for grout excluding hole filling, min	0.1	ton	0.00002	ton/m
SP40 for grout excluding hole filling, max	0.6	ton	0.00013	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	0.5	m^3	0.00010	m^3/m
Volume of grout for hole filling, min	31	m^3	0.00658	m^3/m
Volume of grout for hole filling, max	34	m^3	0.00708	m^3/m
Cement for hole filling, min	23	ton	0.00484	ton/m
Cement for hole filling, max	25	ton	0.00520	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	0.5	ton	0.00009	ton/m
SP40 for hole filling, max	0.5	ton	0.00010	ton/m
SP40 for hole filling, min	0.3	m^3	0.00007	m^3/m
SP40 for hole filling, max	0.4	m^3	0.00008	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	15	m^3	0.00307	m^3/m
Grout volume excluding volume in grout hole, max	78	m^3	0.01650	m^3/m
Cement for grout excluding hole filling, min	4	ton	0.00092	ton/m
Cement for grout excluding hole filling, max	23	ton	0.00493	ton/m
Silica slurry for grout excluding hole filling, min	6	ton	0.00129	ton/m
Silica slurry for grout excluding , max	33	ton	0.00692	ton/m
SP40 for grout excluding hole filling, min	0.2	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	0.9	ton	0.00018	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.00003	m^3/m
SP40 for grout excluding hole filling, max	0.7	m^3	0.00014	m^3/m
Volume of grout for hole filling, min	31	m^3	0.00658	m^3/m
Volume of grout for hole filling, max	34	m^3	0.00708	m^3/m
Cement for hole filling, min	9	ton	0.00197	ton/m
Cement for hole filling, max	23	ton	0.00493	ton/m
Silica slurry for hole filling, min	13	ton	0.00276	ton/m
Silica slurry for hole filling, max	14	ton	0.00296	ton/m
SP40 for hole filling, min	0.3	ton	0.00007	ton/m
SP40 for hole filling, max	0.4	ton	0.00008	ton/m
SP40 for hole filling, min	0.3	m^3	0.00006	m^3/m
SP40 for hole filling, max	0.3	m^3	0.00006	m^3/m

Sealing level 10^{-7} m/s

The quantities given in Table D-11 comprise the required grouting of the subhorizontal zones, which will be passed when excavating the ramp and shafts. One or two grouting fans might be needed in the transport tunnel when passing the deformation zone ZFMNE0060. The quantities that will be consumed when grouting the zone ZFMNE0060, are however estimated to be small compared to the volumes grouted in the subhorizontal zones. Thus these quantities are not included in the summary.

Table D-11. Summary of the required grouting of the subhorizontal zones, sealing level 10^{-7} m/s, early point in time.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Grouting holes, nr, min	31	nr	0.0049	nr/m
Grouting holes, nr, max	31	nr	0.0049	nr/m
Grouting holes, m, min	1,268	m	0.20	m/m
Grouting holes ,m, max	1,268	m	0.20	m/m
Volume of grouting holes, min	4.1	m^3	0.0006	m^3/m
Volume of grouting holes, max	4.1	m^3	0.0006	m^3/m
Grout volume excluding volume in grout hole, min	2	m^3	0.0003	m^3/m
Grout volume excluding volume in grout hole, max	17	m^3	0.0027	m^3/m
Cement for grout excluding hole filling, min	2	ton	0.0002	ton/m
Cement for grout excluding hole filling, max	17	ton	0.0026	ton/m
Silica slurry for grout excluding hole filling, min	0.2	ton	0.00003	ton/m
Silica slurry for grout excluding , max	2	ton	0.00026	ton/m
SP40 for grout excluding hole filling, min	0.01	ton	0.000002	ton/m
SP40 for grout excluding hole filling, max	0.14	ton	0.00002	ton/m
SP40 for grout excluding hole filling, min	0.01	m^3	0.000002	m^3/m
SP40 for grout excluding hole filling, max	0.10	m^3	0.00002	m^3/m
Volume of grout for hole filling, min	4	m^3	0.0006	m^3/m
Volume of grout for hole filling, max	4	m^3	0.0006	m^3/m
Cement for hole filling, min	3.3	ton	0.001	ton/m
Cement for hole filling, max	3.3	ton	0.001	ton/m
Silica slurry for hole filling, min	0	ton	0.00	ton/m
Silica slurry for hole filling, max	0.00	ton	0.00	ton/m
SP40 for hole filling, min	0.07	ton	0.00001	ton/m
SP40 for hole filling, max	0.07	ton	0.00001	ton/m
SP40 for hole filling, min	0.05	m^3	0.00001	m^3/m
SP40 for hole filling, max	0.05	m^3	0.00001	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	2	m^3	0.0003	m^3/m
Grout volume excluding volume in grout hole, max	17	m^3	0.0027	m^3/m
Cement for grout excluding hole filling, min	0.5	ton	0.0001	ton/m
Cement for grout excluding hole filling, max	5.1	ton	0.0008	ton/m
Silica slurry for grout excluding hole filling, min	0.7	ton	0.0001	ton/m
Silica slurry for grout excluding , max	7.1	ton	0.0011	ton/m
SP40 for grout excluding hole filling, min	0.02	ton	0.000003	ton/m
SP40 for grout excluding hole filling, max	0.19	ton	0.00003	ton/m
SP40 for grout excluding hole filling, min	0.01	m^3	0.000002	m^3/m
SP40 for grout excluding hole filling, max	0.14	m^3	0.00002	m^3/m
Volume of grout for hole filling, min	4.1	m^3	0.0006	m^3/m
Volume of grout for hole filling, max	4.1	m^3	0.0006	m^3/m
Cement for hole filling, min	1.2	ton	0.0002	ton/m
Cement for hole filling, max	5.1	ton	0.0008	ton/m
Silica slurry for hole filling, min	1.7	ton	0.0003	ton/m
Silica slurry for hole filling, max	1.7	ton	0.0003	ton/m
SP40 for hole filling, min	0.045	ton	0.000007	ton/m
SP40 for hole filling, max	0.045	ton	0.000007	ton/m
SP40 for hole filling, min	0.034	m^3	0.000005	m^3/m
SP40 for hole filling, max	0.034	m^3	0.000005	m^3/m

D.3.2 Quantities – late point in time

Sealing level 10^{-9} m/s

Table D-12. Summary of grouting for the entire deep repository, sealing level 10^{-9} m/s, late point in time.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Grouting holes, nr, min	6,412	nr	0.10	nr/m
Grouting holes, nr, max	6,937	nr	0.11	nr/m
Grouting holes, m, min	134,639	m	2.12	m/m
Grouting holes ,m, max	145,146	m	2.28	m/m
Volume of grouting holes, min	433	m^3	0.007	m^3/m
Volume of grouting holes, max	467	m^3	0.007	m^3/m
Grout volume excluding volume in grout hole, min	184	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	1,087	m^3	0.017	m^3/m
Cement for grout excluding hole filling, min	87	ton	0.001	ton/m
Cement for grout excluding hole filling, max	557	ton	0.009	ton/m
Silica slurry for grout excluding hole filling, min	1.7	ton	0.00003	ton/m
Silica slurry for grout excluding , max	17	ton	0.00026	ton/m
SP40 for grout excluding hole filling, min	1.5	ton	0.000023	ton/m
SP40 for grout excluding hole filling, max	8.7	ton	0.00014	ton/m
SP40 for grout excluding hole filling, min	1.1	m^3	0.000018	m^3/m
SP40 for grout excluding hole filling, max	6.7	m^3	0.00011	m^3/m
Volume of grout for hole filling, min	433	m^3	0.007	m^3/m
Volume of grout for hole filling, max	467	m^3	0.007	m^3/m
Cement for hole filling, min	337	ton	0.005	ton/m
Cement for hole filling, max	364	ton	0.006	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	6.6	ton	0.00010	ton/m
SP40 for hole filling, max	7.2	ton	0.00011	ton/m
SP40 for hole filling, min	5.1	m^3	0.00008	m^3/m
SP40 for hole filling, max	5.5	m^3	0.00009	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	184	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	1,087	m^3	0.017	m^3/m
Cement for grout excluding hole filling, min	55	ton	0.001	ton/m
Cement for grout excluding hole filling, max	325	ton	0.005	ton/m
Silica slurry for grout excluding hole filling, min	77	ton	0.001	ton/m
Silica slurry for grout excluding , max	455	ton	0.007	ton/m
SP40 for grout excluding hole filling, min	2.0	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	12.0	ton	0.00019	ton/m
SP40 for grout excluding hole filling, min	1.6	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	9.2	m^3	0.00014	m^3/m
Volume of grout for hole filling, min	433	m^3	0.007	m^3/m
Volume of grout for hole filling, max	467	m^3	0.007	m^3/m
Cement for hole filling, min	129	ton	0.002	ton/m
Cement for hole filling, max	325	ton	0.005	ton/m
Silica slurry for hole filling, min	181	ton	0.003	ton/m
Silica slurry for hole filling, max	196	ton	0.003	ton/m
SP40 for hole filling, min	4.8	ton	0.00007	ton/m
SP40 for hole filling, max	5.1	ton	0.00008	ton/m
SP40 for hole filling, min	3.7	m^3	0.00006	m^3/m
SP40 for hole filling, max	3.9	m^3	0.00006	m^3/m

Table D-13. Summary of grouting at repository depth, sealing level 10^{-9} m/s, late point in time.

Item	Quantity	Unit	Per metre of tunnel and rock cavern at repository depth	Unit
Grouting holes, nr, min	5,806	nr	0.10	nr/m
Grouting holes, nr, max	6,331	nr	0.10	nr/m
Grouting holes, m, min	116,114	m	1.91	m/m
Grouting holes ,m, max	126,621	m	2.09	m/m
Volume of grouting holes, min	373	m^3	0.01	m^3/m
Volume of grouting holes, max	407	m^3	0.01	m^3/m
Grout volume excluding volume in grout hole, min	160	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	880	m^3	0.01	m^3/m
Cement for grout excluding hole filling, min	68	ton	0.001	ton/m
Cement for grout excluding hole filling, max	376	ton	0.006	ton/m
Silica slurry for grout excluding hole filling, min	0	ton	0.0000	ton/m
Silica slurry for grout excluding , max	0	ton	0.0000	ton/m
SP40 for grout excluding hole filling, min	1	ton	0.00002	ton/m
SP40 for grout excluding hole filling, max	7	ton	0.00012	ton/m
SP40 for grout excluding hole filling, min	1	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	5	m^3	0.00009	m^3/m
Volume of grout for hole filling, min	373	m^3	0.01	m^3/m
Volume of grout for hole filling, max	407	m^3	0.01	m^3/m
Cement for hole filling, min	289	ton	0.005	ton/m
Cement for hole filling, max	316	ton	0.01	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	5.7	ton	0.00009	ton/m
SP40 for hole filling, max	6.2	ton	0.00010	ton/m
SP40 for hole filling, min	4.4	m^3	0.00007	m^3/m
SP40 for hole filling, max	4.8	m^3	0.00008	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	160	m^3	0.0026	m^3/m
Grout volume excluding volume in grout hole, max	880	m^3	0.0145	m^3/m
Cement for grout excluding hole filling, min	48	ton	0.0008	ton/m
Cement for grout excluding hole filling, max	263	ton	0.0043	ton/m
Silica slurry for grout excluding hole filling, min	67	ton	0.0011	ton/m
Silica slurry for grout excluding , max	369	ton	0.0061	ton/m
SP40 for grout excluding hole filling, min	1.8	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	9.7	ton	0.0002	ton/m
SP40 for grout excluding hole filling, min	1.4	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	7.4	m^3	0.0001	m^3/m
Volume of grout for hole filling, min	373	m^3	0.0062	m^3/m
Volume of grout for hole filling, max	407	m^3	0.0067	m^3/m
Cement for hole filling, min	112	ton	0.0018	ton/m
Cement for hole filling, max	263	ton	0.0043	ton/m
Silica slurry for hole filling, min	156	ton	0.0026	ton/m
Silica slurry for hole filling, max	171	ton	0.0028	ton/m
SP40 for hole filling, min	4.1	ton	0.00007	ton/m
SP40 for hole filling, max	4.5	ton	0.00007	ton/m
SP40 for hole filling, min	3.2	m^3	0.00005	m^3/m
SP40 for hole filling, max	3.4	m^3	0.00006	m^3/m

Table D-14. Summary of grouting in deposition tunnels, sealing level 10^{-9} m/s, late point in time.

Item	Quantity	Unit	Per metre deposition tunnel	Unit
Grouting holes, nr, min	4,861	nr	0.10	nr/m
Grouting holes, nr, max	5,227	nr	0.11	nr/m
Grouting holes, m, min	97,230	m	2.05	m/m
Grouting holes ,m, max	104,537	m	2.20	m/m
Volume of grouting holes, min	313	m^3	0.0066	m^3/m
Volume of grouting holes, max	336	m^3	0.0071	m^3/m
Grout volume excluding volume in grout hole, min	146	m^3	0.003	m^3/m
Grout volume excluding volume in grout hole, max	784	m^3	0.017	m^3/m
Cement for grout excluding hole filling, min	62	ton	0.001	ton/m
Cement for grout excluding hole filling, max	335	ton	0.007	ton/m
Silica slurry for grout excluding hole filling, min	0	ton	0	ton/m
Silica slurry for grout excluding , max	0	ton	0	ton/m
SP40 for grout excluding hole filling, min	1.2	ton	0.00002	ton/m
SP40 for grout excluding hole filling, max	6.3	ton	0.00013	ton/m
SP40 for grout excluding hole filling, min	0.9	m^3	0.00002	m^3/m
SP40 for grout excluding hole filling, max	4.8	m^3	0.00010	m^3/m
Volume of grout for hole filling, min	312.6	m^3	0.00658	m^3/m
Volume of grout for hole filling, max	336.1	m^3	0.00708	m^3/m
Cement for hole filling, min	229.9	ton	0.00484	ton/m
Cement for hole filling, max	247.2	ton	0.00520	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	4.5	ton	0.00009	ton/m
SP40 for hole filling, max	4.8	ton	0.00010	ton/m
SP40 for hole filling, min	3.5	m^3	0.00007	m3/m
SP40 for hole filling, max	3.7	m^3	0.00008	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	146	m^3	0.00307	m^3/m
Grout volume excluding volume in grout hole, max	784	m^3	0.01650	m^3/m
Cement for grout excluding hole filling, min	43.6	ton	0.00092	ton/m
Cement for grout excluding hole filling, max	234.4	ton	0.00493	ton/m
Silica slurry for grout excluding hole filling, min	61.1	ton	0.00129	ton/m
Silica slurry for grout excluding , max	328.5	ton	0.00692	ton/m
SP40 for grout excluding hole filling, min	1.6	ton	0.00003	ton/m
SP40 for grout excluding hole filling, max	8.6	ton	0.00018	ton/m
SP40 for grout excluding hole filling, min	1.2	m^3	0.00003	m^3/m
SP40 for grout excluding hole filling, max	6.6	m^3	0.00014	m^3/m
Volume of grout for hole filling, min	312.6	m^3	0.00658	m^3/m
Volume of grout for hole filling, max	336.1	m^3	0.00708	m^3/m
Cement for hole filling, min	93.5	ton	0.00197	ton/m
Cement for hole filling, max	234.4	ton	0.00493	ton/m
Silica slurry for hole filling, min	131.0	ton	0.00276	ton/m
Silica slurry for hole filling, max	140.8	ton	0.00296	ton/m
SP40 for hole filling, min	3.4	ton	0.00007	ton/m
SP40 for hole filling, max	3.7	ton	0.00008	ton/m
SP40 for hole filling, min	2.6	m^3	0.00006	m^3/m
SP40 for hole filling, max	2.8	m^3	0.00006	m^3/m

Sealing level 10^{-7} m/s

The quantities given in Table D-15 comprise the required grouting of the subhorizontal zones, which will be passed when excavating the ramp and shafts. One or two grouting fans might be needed in the transport tunnel when passing the deformation zone ZFMNE0060. The quantities that will be consumed when grouting the zone ZFMNE0060, are however estimated to be small compared to the volumes grouted in the subhorizontal zones. Thus these quantities are not included in the summary.

Table D-15. Summary of the required grouting of the subhorizontal zones, sealing level 10^{-7} m/s, late point in time.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Grouting holes, nr, min	314	nr	0.005	nr/m
Grouting holes, nr, max	314	nr	0.005	nr/m
Grouting holes, m, min	12,680	m	0.20	m/m
Grouting holes ,m, max	12,680	m	0.20	m/m
Volume of grouting holes, min	41	m^3	0.001	m^3/m
Volume of grouting holes, max	41	m^3	0.001	m^3/m
Grout volume excluding volume in grout hole, min	16	m^3	0.0003	m^3/m
Grout volume excluding volume in grout hole, max	170	m^3	0.003	m^3/m
Cement for grout excluding hole filling, min	16	ton	0.0002	ton/m
Cement for grout excluding hole filling, max	165	ton	0.003	ton/m
Silica slurry for grout excluding hole filling, min	2	ton	0.00003	ton/m
Silica slurry for grout excluding , max	17	ton	0.00026	ton/m
SP40 for grout excluding hole filling, min	0.1	ton	0.000002	ton/m
SP40 for grout excluding hole filling, max	1.4	ton	0.00002	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.000002	m^3/m
SP40 for grout excluding hole filling, max	1.0	m^3	0.00002	m^3/m
Volume of grout for hole filling, min	41	m^3	0.0006	m^3/m
Volume of grout for hole filling, max	41	m^3	0.0006	m^3/m
Cement for hole filling, min	33	ton	0.0005	ton/m
Cement for hole filling, max	33	ton	0.0005	ton/m
Silica slurry for hole filling, min	0	ton	0	ton/m
Silica slurry for hole filling, max	0	ton	0	ton/m
SP40 for hole filling, min	0.65	ton	0.000010	ton/m
SP40 for hole filling, max	0.65	ton	0.000010	ton/m
SP40 for hole filling, min	0.50	m^3	0.000008	m^3/m
SP40 for hole filling, max	0.50	m^3	0.000008	m^3/m
Alternative grout supplied by SKB				
Grout volume excluding volume in grout hole, min	16	m^3	0.00025	m^3/m
Grout volume excluding volume in grout hole, max	170	m^3	0.003	m^3/m
Cement for grout excluding hole filling, min	5	ton	0.00008	ton/m
Cement for grout excluding hole filling, max	51	ton	0.001	ton/m
Silica slurry for grout excluding hole filling, min	7	ton	0.00011	ton/m
Silica slurry for grout excluding , max	71	ton	0.001	ton/m
SP40 for grout excluding hole filling, min	0.2	ton	0.000003	ton/m
SP40 for grout excluding hole filling, max	2	ton	0.00003	ton/m
SP40 for grout excluding hole filling, min	0.1	m^3	0.000002	m^3/m
SP40 for grout excluding hole filling, max	1	m^3	0.00002	m^3/m
Volume of grout for hole filling, min	41	m^3	0.001	m^3/m
Volume of grout for hole filling, max	41	m^3	0.001	m^3/m
Cement for hole filling, min	12	ton	0.0002	ton/m
Cement for hole filling, max	51	ton	0.001	ton/m
Silica slurry for hole filling, min	17	ton	0.00027	ton/m
Silica slurry for hole filling, max	17	ton	0.00027	ton/m
SP40 for hole filling, min	0.45	ton	0.000007	ton/m
SP40 for hole filling, max	0.45	ton	0.000007	ton/m
SP40 for hole filling, min	0.34	m^3	0.000005	m^3/m
SP40 for hole filling, max	0.34	m^3	0.000005	m^3/m

Appendix E

Estimation of rock support quantity

E.1 Premises and assumptions

The geometries of different parts of the facility are presented in Table E-1. The perimeter length of roof and walls corresponds to the theoretical boundaries, which means that the area of shotcrete will be calculated in accordance with MER2002 Anläggning /Svensk Byggtjänst 2002/, which normally is applied for Swedish tunnelling projects.

Table E-2 sets out the compositions of shotcrete, grout and plugs provided by SKB.

Other important assumptions for the estimation of quantities of rock support are listed in Table E-3.

Table E-1. Geometries of extensive parts of the facility, which are used for the estimation of quantities.

Part of the facility	Geometry	Value
Main tunnel	Roof	11.3 m
	Walls	10.4 m
	Length, layout 1 and 2	5,030 m
	Length, layout 3 and 4	4,923 m
	Number of intersections with deposition tunnels	115
Transport tunnels in deposition area	Roof	8.5 m
	Walls	10.4 m
	Length, layout 1 and 2	2,676 m
	Length, layout 3 and 4	2,869 m
Deposition tunnels	Roof	6.2 m
	Walls	8 m
	Length, layout 1 and 2	47,503 m
	Length, layout 3 and 4	48,618 m
	Number of concrete plugs, layout 1 and 2	187
	Number of concrete plugs, layout 3 and 4	190
Ramp	Roof	7.2 m
	Walls	8.4 m
	Length, layout 1 and 2	4,000 m
	Length, layout 3 and 4	5,000 m

Table E-2. Compositions of shotcrete, grout for anchoring of rock bolts and concrete plugs (provided by SKB).

Rock support construction	Material	Quantity kg/m³
Shotcrete for rock support	Cement	306
	Silica fume	204
	Aggregates	1,500
	Plasticiser (SP40)	Approximately 7
	Accelerator	15 (no alkali in accelerator, assumed by design team)
	Water	214
	Steel fibres	70
Grout for anchoring of rock bolts	Cement (white)	596
	Silica fume	255
	Water	696
	Plasticiser (SP40)	2
Concrete plugs	Cement	240
	Silica fume	160
	Aggregates (sand + 4–15 mm)	710+1,160
	Water	160
	Plasticiser (SP40)	6

Table E-3. Assumptions for the estimation of quantities (assumptions made by the designer).

Issue	Assumption
Intensity of spot bolting in deposition tunnels	0.25–0.5 bolts/m
Intensity of spot bolting in rock caverns	1–2 bolts/m
Intensity of spot bolting in other tunnels	0.5–1 bolts/m
Bolts, weight	4 kg/m bolt (25 mm diameter, including all bolt components, e.g. faceplate)
Factor for calculating actual shotcrete volume from theoretical volume (allowing for irregularities of rock surface and rebound)	1.1–1.3
Number of supplementary rock bolts per intersection between deposition and main tunnels	200
Supplementary area of shotcrete per intersection between deposition and main tunnels	400 m²
Specific weight of steel mesh	1.7 kg/m²
Specific weight of plastic mesh	0.8 kg/m²
Rock bolts for fixing mesh, length	0.5 m
Rock bolts for fixing mesh, intensity	0.5 bolts/m²
Rock bolts for fixing mesh, weight	2 kg/m bolt (16 mm diameter)

E.2 Estimation of quantities

Tables E-4 to E-11 present summaries of quantities of rock support as follows:

- the entire deep repository,
- tunnels and rock caverns at deposition depth (excluding shafts and access ramp),
- deposition tunnels,
- quantities for concrete plugs.

Quantities are given for estimations of both minimum (min) and maximum (max) values.

Furthermore the estimation of quantities:

- is based on a minimum rock support with either shotcrete or mesh,
- is presented for both steel mesh and plastic mesh,
- does not include the possible requirement for mesh in deposition tunnels.

It should be noted that the ventilation shafts have not been included in the estimated quantities per metre for the entire repository, since no support was estimated in the present design step.

Table E-4. Summary of support requirements for the entire deep repository, Layouts 1 and 2.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Rock bolts, nr, min	48,607	nr	0.78	nr/m
Rock bolts, nr, max	68,884	nr	1.11	nr/m
Rock bolt steel, weight, min	539	ton	0.009	ton/m
Rock bolt steel, weight, max	751	ton	0.012	ton/m
Rock bolt grout, volume, min	135	m^3	0.0022	m^3/m
Rock bolt grout, volume, max	188	m^3	0.0030	m^3/m
Rock bolt grout, cement, min	80	ton	0.0013	ton/m
Rock bolt grout, cement, max	112	ton	0.0018	ton/m
Rock bolt grout, silica fume, min	34	ton	0.0006	ton/m
Rock bolt grout, silica fume, max	48	ton	0.0008	ton/m
Rock bolt grout, SP40, min	0.27	ton	0.000004	ton/m
Rock bolt grout, SP40, max	0.38	ton	0.000006	ton/m
Rock bolt grout, SP40, min	0.21	m^3	0.000003	m^3/m
Rock bolt grout, SP40, max	0.29	m^3	0.000005	m^3/m
Shotcrete, area	187,647	m^2	3.03	m^2/m
Shotcrete, volume, min	10,375	m^3	0.17	m^3/m
Shotcrete, volume, max	12,262	m^3	0.20	m^3/m
Shotcrete, cement, min	3,175	ton	0.051	ton/m
Shotcrete, cement, max	3,752	ton	0.060	ton/m
Shotcrete, silica fume, min	2,117	ton	0.034	ton/m
Shotcrete, silica fume, max	2,501	ton	0.040	ton/m
Shotcrete, SP40, min	73	ton	0.0012	ton/m
Shotcrete, SP40, max	86	ton	0.0014	ton/m

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Shotcrete, SP40, min	56	m³	0.0009	m³/m
Shotcrete, SP40, max	66	m³	0.0011	m³/m
Shotcrete, steel fibres, min	726	ton	0.012	ton/m
Shotcrete, steel fibres, max	858	ton	0.014	ton/m
Shotcrete, accelerator, min	156	ton	0.003	ton/m
Shotcrete, accelerator, max	184	ton	0.003	ton/m
Shotcrete, accelerator, min	120	m³	0.002	m³/m
Shotcrete, accelerator, max	141	m³	0.002	m³/m
Mesh instead of shotcrete (minimum support)				
Mesh, area	171,975	m²	2.81	m²/m
Mesh, weight (steel)	292	ton	0.005	ton/m
Mesh, weight (plastic)	138	ton	0.002	ton/m
Fixing bolt for mesh, nr	85,988	nr	1.41	nr/m
Fixing bolt for mesh, weight	86	ton	0.001	ton/m

Table E-5. Summary of support requirements at repository depth, Layouts 1 and 2.

Item	Quantity	Unit	Per metre of tunnel and rock cavern at repository depth	Unit
Rock bolts, nr, min	46,182	nr	0.81	nr/m
Rock bolts, nr, max	64,034	nr	1.12	nr/m
Rock bolt steel, weight, min	511	ton	0.009	ton/m
Rock bolt steel, weight, max	695	ton	0.012	ton/m
Rock bolt grout, volume, min	128	m³	0.0022	m³/m
Rock bolt grout, volume, max	174	m³	0.0030	m³/m
Rock bolt grout, cement, min	76	ton	0.0013	ton/m
Rock bolt grout, cement, max	104	ton	0.0018	ton/m
Rock bolt grout, silica fume, min	33	ton	0.0006	ton/m
Rock bolt grout, silica fume, max	44	ton	0.0008	ton/m
Rock bolt grout, SP40, min	0.26	ton	0.000004	ton/m
Rock bolt grout, SP40, max	0.35	ton	0.000006	ton/m
Rock bolt grout, SP40, min	0.20	m³	0.000003	m³/m
Rock bolt grout, SP40, max	0.27	m³	0.000005	m³/m
Shotcrete, area	144,168	m²	2.52	m²/m
Shotcrete, volume, min	7,984	m³	0.14	m³/m
Shotcrete, volume, max	9,435	m³	0.17	m³/m
Shotcrete, cement, min	2,443	ton	0.04	ton/m
Shotcrete, cement, max	2,887	ton	0.05	ton/m
Shotcrete, silica fume, min	1,629	ton	0.03	ton/m
Shotcrete, silica fume, max	1,925	ton	0.03	ton/m
Shotcrete, SP40, min	56	ton	0.0010	ton/m
Shotcrete, SP40, max	66	ton	0.0012	ton/m
Shotcrete, SP40, min	43	m³	0.0008	m³/m

Item	Quantity	Unit	Per metre of tunnel and rock cavern at repository depth	Unit
Shotcrete, SP40, max	51	m³	0.0009	m³/m
Shotcrete, steel fibres, min	559	ton	0.010	ton/m
Shotcrete, steel fibres, max	660	ton	0.012	ton/m
Shotcrete, accelerator, min	120	ton	0.002	ton/m
Shotcrete, accelerator, max	142	ton	0.002	ton/m
Shotcrete, accelerator, min	92	m³	0.002	m³/m
Shotcrete, accelerator, max	109	m³	0.002	m³/m
Mesh instead of shotcrete (minimum support)				
Mesh, area	143,175	m²	2.50	m²/m
Mesh, weight (steel)	243	ton	0.004	ton/m
Mesh, weight (plastic)	115	ton	0.002	ton/m
Fixing bolt for mesh, nr	71,588	nr	1.25	nr/m
Fixing bolt for mesh, weight	72	ton	0.001	ton/m

Table E-6. Summary of support requirements in deposition tunnels, Layouts 1 and 2.

Item	Quantity	Unit	Per metre deposition tunnel	Unit
Rock bolts, nr, min	17,769	nr	0.37	nr/m
Rock bolts, nr, max	30,317	nr	0.64	nr/m
Rock bolt steel, weight, min	171	ton	0.004	ton/m
Rock bolt steel, weight, max	291	ton	0.01	ton/m
Rock bolt grout, volume, min	43	m³	0.00090	m³/m
Rock bolt grout, volume, max	73	m³	0.00153	m³/m
Rock bolt grout, cement, min	25	ton	0.00054	ton/m
Rock bolt grout, cement, max	43	ton	0.00091	ton/m
Rock bolt grout, silica fume, min	11	ton	0.00023	ton/m
Rock bolt grout, silica fume, max	19	ton	0.00039	ton/m
Rock bolt grout, SP40, min	0.09	ton	0.0000018	ton/m
Rock bolt grout, SP40, max	0.15	ton	0.0000031	ton/m
Rock bolt grout, SP40, min	0.07	m³	0.0000014	m³/m
Rock bolt grout, SP40, max	0.11	m³	0.0000024	m³/m

Table E-7. Summary of concrete plugs, Layouts 1 and 2.

Item	Quantity	Unit
Concrete, volume	26,180	m³
Steel reinforcement	1,571	ton
Cement	6,283	ton
Silica fume	4,189	ton
SP40	157	ton
SP40	121	m³

Table E-8. Summary of support requirements for the entire deep repository, Layouts 3 and 4.

Item	Quantity	Unit	Per metre of tunnel, rock cavern and shaft	Unit
Rock bolts, nr, min	48,992	nr	0.76	nr/m
Rock bolts, nr, max	70,212	nr	1.09	nr/m
Rock bolt steel, weight, min	544	ton	0.01	ton/m
Rock bolt steel, weight, max	766	ton	0.01	ton/m
Rock bolt grout, volume, min	136	m^3	0.0021	m^3/m
Rock bolt grout, volume, max	191	m^3	0.0030	m^3/m
Rock bolt grout, cement, min	81	ton	0.0013	ton/m
Rock bolt grout, cement, max	114	ton	0.0018	ton/m
Rock bolt grout, silica fume, min	35	ton	0.0005	ton/m
Rock bolt grout, silica fume, max	49	ton	0.0008	ton/m
Rock bolt grout, SP40, min	0.3	ton	0.000004	ton/m
Rock bolt grout, SP40, max	0.4	ton	0.000006	ton/m
Rock bolt grout, SP40, min	0.2	m^3	0.000003	m^3/m
Rock bolt grout, SP40, max	0.3	m^3	0.000005	m^3/m
Shotcrete, area	198,649	m^2	3.08	m^2/m
Shotcrete, volume, min	10,972	m^3	0.17	m^3/m
Shotcrete, volume, max	12,967	m^3	0.20	m^3/m
Shotcrete, cement, min	3,357	ton	0.05	ton/m
Shotcrete, cement, max	3,968	ton	0.06	ton/m
Shotcrete, silica fume, min	2,238	ton	0.03	ton/m
Shotcrete, silica fume, max	2,645	ton	0.04	ton/m
Shotcrete, SP40, min	77	ton	0.0012	ton/m
Shotcrete, SP40, max	91	ton	0.0014	ton/m
Shotcrete, SP40, min	59	m^3	0.0009	m^3/m
Shotcrete, SP40, max	70	m^3	0.0011	m^3/m
Shotcrete, steel fibres, min	768	ton	0.01	ton/m
Shotcrete, steel fibres, max	908	ton	0.01	ton/m
Shotcrete, accelerator, min	165	ton	0.0026	ton/m
Shotcrete, accelerator, max	195	ton	0.0030	ton/m
Shotcrete, accelerator, min	127	m^3	0.0020	m^3/m
Shotcrete, accelerator, max	150	m^3	0.0023	m^3/m
Mesh instead of shotcrete (minimum support)				
Mesh, area	179,675	m^2	2.79	m^2/m
Mesh, weight (steel)	265	ton	0.004	ton/m
Mesh, weight (plastic)	125	ton	0.002	ton/m
Fixing bolt for mesh, nr	101,841	nr	1.58	nr/m
Fixing bolt for mesh, weight	119	ton	0.002	ton/m

Table E-9. Summary of support requirements at repository depth, Layouts 3 and 4.

Item	Quantity	Unit	Per metre of tunnel and rock cavern at repository depth	Unit
Rock bolts, nr, min	45,967	nr	0.79	nr/m
Rock bolts, nr, max	64,162	nr	1.10	nr/m
Rock bolt steel, weight, min	509	ton	0.01	ton/m
Rock bolt steel, weight, max	696	ton	0.01	ton/m
Rock bolt grout, volume, min	127	m^3	0.0022	m^3/m
Rock bolt grout, volume, max	174	m^3	0.0030	m^3/m
Rock bolt grout, cement, min	76	ton	0.0013	ton/m
Rock bolt grout, cement, max	104	ton	0.0018	ton/m
Rock bolt grout, silica fume, min	32	ton	0.0006	ton/m
Rock bolt grout, silica fume, max	44	ton	0.0008	ton/m
Rock bolt grout, SP40, min	0.3	ton	0.000004	ton/m
Rock bolt grout, SP40, max	0.3	ton	0.000006	ton/m
Rock bolt grout, SP40, min	0.2	m^3	0.000003	m^3/m
Rock bolt grout, SP40, max	0.3	m^3	0.000005	m^3/m
Shotcrete area	144,516	m^2	2.48	m^2/m
Shotcrete volume, min	7,995	m^3	0.14	m^3/m
Shotcrete volume, max	9,448	m^3	0.16	m^3/m
Shotcrete, cement, min	2,446	ton	0.04	ton/m
Shotcrete, cement, max	2,891	ton	0.05	ton/m
Shotcrete, silica fume, min	1,631	ton	0.03	ton/m
Shotcrete, silica fume, max	1,927	ton	0.03	ton/m
Shotcrete, SP40, min	56	ton	0.0010	ton/m
Shotcrete, SP40, max	66	ton	0.0011	ton/m
Shotcrete, SP40, min	43	m^3	0.0007	m^3/m
Shotcrete, SP40, max	51	m^3	0.0009	m^3/m
Shotcrete, steel fibres, min	560	ton	0.01	ton/m
Shotcrete, steel fibres, max	661	ton	0.01	ton/m
Shotcrete, accelerator, min	120	ton	0.0021	ton/m
Shotcrete, accelerator, max	142	ton	0.0024	ton/m
Shotcrete, accelerator, min	92	m^3	0.0016	m^3/m
Shotcrete, accelerator, max	109	m^3	0.0019	m^3/m
Mesh instead of shotcrete (minimum support)				
Mesh, area	143,175	m^2	2.45	m^2/m
Mesh, weight (steel)	243	ton	0.004	ton/m
Mesh, weight (plastic)	115	ton	0.002	ton/m
Fixing bolt for mesh, nr	71,588	nr	1.23	nr/m
Fixing bolt for mesh, weight	72	ton	0.001	ton/m

Table E-10. Summary of support requirements in deposition tunnels, Layouts 3 and 4.

Item	Quantity	Unit	Per metre of deposition tunnel	Unit
Rock bolts, nr, min	17,561	nr	0.36	nr/m
Rock bolts, nr, max	30,439	nr	0.63	nr/m
Rock bolt steel, weight, min	169	ton	0.003	ton/m
Rock bolt steel, weight, max	292	ton	0.01	ton/m
Rock bolt grout, volume, min	42	m^3	0.00089	m^3/m
Rock bolt grout, volume, max	73	m^3	0.00154	m^3/m
Rock bolt grout, cement, min	25	ton	0.00053	ton/m
Rock bolt grout, cement, max	44	ton	0.00092	ton/m
Rock bolt grout, silica fume, min	11	ton	0.00023	ton/m
Rock bolt grout, silica fume, max	19	ton	0.00039	ton/m
Rock bolt grout, SP40, min	0.08	ton	0.0000018	ton/m
Rock bolt grout, SP40, max	0.15	ton	0.0000031	ton/m
Rock bolt grout, SP40, min	0.06	m^3	0.0000014	m^3/m
Rock bolt grout, SP40, max	0.11	m^3	0.0000024	m^3/m

Table E-11. Summary of concrete plugs, Layouts 3 and 4.

Item	Quantity	Unit
Concrete, volume	26,600	m^3
Steel reinforcement	1,596	ton
Cement	8,140	ton
Silica fume	5,426	ton
SP40	186	ton
SP40	143	m^3