R-06-36

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November 2006

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Summary

Objectives

This report summarizes the results from the work with design step D1 for a deep repository located in Laxemar. The work was based on the Site Descriptive Model Laxemar v 1.2 (SDM v 1.2).

According to current plans for the Swedish nuclear programme the minimum number of canister positions required in the repository is determined to be 4,500. However, to enable possible tentative extensions of the nuclear programme, an extra 1,500 positions were added. Thus the design was made for 6,000 positions.

The guidelines for the design work for the deep repository was described in "Underground Design Premises" (UDP) /SKB 2004a/. According to he UDP the main objectives of the design step D1 are:

- 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.

Some deviations from the guidelines in the UDP were initiated during the execution of the work due to results from other studies and R&D work. These deviations are summarised and explained in Chapter 2 of the report.

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

The possible location for a tentative final repository has been defined by SKB to be within the Laxemar subarea. Laxemar subarea is, together with Simpevarp subarea, the area covered by the site investigation in Oskarshamn. This report only deals with studies regarding the Laxemar area. The extent of the Laxemar area can be seen from Figure 3-2 of the report and the interfaces are as follows:

- To the east, Deformation Zone ZSMNE005A (Äspö shear zone) and the outer limit of national interest for a final repository.
- To the north, Deformation Zone ZSMEW002A (Mederhult zone).
- To the west, Deformation Zone ZSMNS001C.
- To the south, the interface corresponds with the southern limit of national interest for a final repository /SKI 2004/.

The bedrock within the area in question consists of fine-grained crystalline basement. The various rock types have a similar composition and differ in the first instance with regard to grain size and colour. For the design work in Laxemar, the following rock domains are considered:

- Rock domain A: Mainly Ävrö granite.
- Rock domain B: Mainly fine-grained dioritoid.
- Rock domain BA: A mixture of Ävrö granite and fine-grained diorite.
- Rock domain D: Mainly quartz monzodiorite.
- Rock domain M: A large proportion of diorite/gabbro in Ävrö granite and quartz monzodiorite.

The existence and extent of the rock domains vary with depth. The depths that have been investigated are between 400 and 700 m.

The preliminary assessment made in Chapter 3 clearly demonstrates that the repository can be accommodated within the Laxemar area. The factor P varies between 2.76 and 2.91 where a P value of 1 means sufficient capacity to accommodate the repository. For this study medium and high confidence zones where taken into account.

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 of repository depth.

For design step D1 tunnel geometries and dimensions were recommended to be in accordance with Layout E /SKB 2002/, which also states a distance between deposition tunnels of 40. The minimum canister spacing varies from 6.7 to 7.6 m, at a depth of 400 m, and from 7.8 m to 9.2 m, at a depth of 700 m. The depth variations in spacing are an effect of variations in thermal properties for the rock domains.

The analyses of optimal orientation with respect to water seepage to the deposition tunnel and deposition holes indicate an optimal orientation of the deposition tunnels is N132°. However, the differences between different orientations are very small.

The risk of spalling is negligible down to 500 m depth, irrespective of tunnel orientation. At a depth of 600 m the probability of spalling is 40% for the least favourable tunnel orientation, /Martin 2005/, although the volume of overbreak is small. Furthermore, the least favourable tunnel direction, with increasing depth, is perpendicular to the main stress.

The optimal orientation of the deposition tunnels with respect to unstable wedges is not well pronounced, but the analysis indicates a value of approximately N12° for all depths.

The performed analysis of loss of deposition holes depending on elongated fractures have only been assigned one value, regardless of depth, the value is 16%. Deposition hole loss due to unacceptably high water inflows indicate a loss of between 1.5 and 6% for the criterion $q > 10$ l/min, depending on the rock domain.

The results of the wedge failure analyses indicate that the loss could be about 5% if we choose the volume criterion with wedges larger than 0.15 m^3. A review of the results from drilled deposition holes in Äspö indicates a zero per cent loss, which affects the assessment of the loss proportion for future design.

The study by /Martin 2005/, indicates that at a depth of 500 m, the probability for stress-induced spalling in deposition holes is 5%, but the volume of the overbreak is minimal. At a depth of 600 m, the probability of stress-induced spalling is significantly higher, 40%, but also here it is considered to be a question of small volumes of loose rock.

After considering all design analyses, it is recommended that the repository should be located as close to the surface as possible, i.e. at a depth of 400 m. However, there are factors of importance for the long-term safety that are not considered in the UDP /SKB 2004a/. Since the benefits of placing the repository at –400 m compared to the initial reference level of –500 m are marginal, the reference level of –500 m has been maintained for the purposes of the current Laxemar D1 layout.

Layout studies

In Chapter 5 the layout studies are reported, and two alternative layouts for each repository level at 500 and 600 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 two alternatives at 500 m indicate the most favourable conditions. They both give smaller loss of deposition holes as well as a considerably smaller volume of excavated rock. Furthermore, the higher loss of deposition holes at 600 m, which could be as much as 40%, means that in reality all the additional holes would have to be used in these alternatives.

One of the two alternatives at 500 m is located with the central area situated in the south western part (West) of the Laxemar area, whereas the other is situated in the central part (Central). The area in the central part is considered to be more advantageous, primarily with respect to the conditions for the surface facilities, and has therefore been chosen as the basic layout.

For the basic layout the total volume, included deposition area, central area, ramp and shafts, is approximately 2.5 million m3 (excluding deposition holes), including 72 km of tunnels and 7,500 available canister positions (including allowance for a loss of deposition holes of 20%).

Identification of passages through deformation zones

Studies of identified passages through deformation zones are presented in Chapter 6. Totally six passages and one reserve passage have been studied.

For the majority of the passages there are no major problems expected during excavation through the deformation zones. However there is a risk of potentially high water inflow in some of the passages and the consequence is careful excavation with extensive grouting work. Many alternative excavations, including rock support, and grouting methods are possible and should be in readiness.

Seepage and hydrogeological situation around the repository

The study of seepage and hydrogeological situation is reported in Chapter 7.

The results from the calculations of seepage to the repository show a strong variation depending on calculation method, grouting level and construction step. The results from the analytical calculations of seepage range from 0.2 l/s–29 l/s depending on construction step. For the numerical simulations, the corresponding values are 19–37 l/s.

The drawdown area, as calculated using the numerical model, will be significant. For both grouting to a hydraulic conductivity of 10^{-7} and 10^{-9} m/s, an area of about 10 km^2 will get a groundwater table that is depressed by 0.3 m or more.

The results from the numerical simulations show small possibilities for inflow of saline groundwater.

Assessment of rock grouting need

Assessment of the rock grouting need is reported in Chapter 8. The total grout quantity injected into the rock mass, included plugged volume, is estimated to be 1,700 to 2,950 m^3 for grouting level 1 ($K = 10^{-7}$ m/s) and 4,950 to 9,650 m^3 for grouting level 2 ($K = 10^{-9}$ m/s). The deposition tunnels need 1,200 to 1,900 m^3 for grouting level 1 and 3,600 to 6,900 m^3 for grouting level 2, all including plugged volume.

The estimates and calculations of grouting quantities are very uncertain and are based on a number of assumptions and subjective assessments, which are of great importance for the forecast quantities. In addition, the planned facility is large and complex, which means that individual uncertainties may together be of great importance.

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 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. A preliminary assessment has been made of required support quantities in the repository.

The total quantity of bolts in the complete facility is calculated to be between 145,000 and 189,000 pcs, of which approximately 102,000 to 133,000 pcs are in deposition tunnels. The total amount of fibre reinforced shotcrete is between 12,000 and 19,000 m^3. Only 400 to 2,000 m^3 fibre reinforced shotecrete is calculated in deposition tunnels, instead wire mesh is proposed as rock support. The wire mesh is estimated in deposition tunnels to be between 219,000 and 293,000 m^2. A small amount of approximately 20 m^3 unreinforced shotcrete is calculated in the other tunnels/rock caverns.

The required quantity of rock support will depend on the extent of stress-induced spalling in the facility, even though it has in general been assumed to be less than 5% of the overall tunnel length. If no form of stress-induced spalling occurs, the quantity of shotcrete could probably be significantly reduced. The number of bolts would also be reduced, though to a somewhat lesser extent.

The concrete, i.e. both the shotcrete and the bolt mortar, will have a low pH, according to SKB's safety analysis, in order to avoid an excessively alkaline environment in the rock mass around the final repository.

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 assigned area". A model considering variations in different factors, which influence the available area for the repository, was developed and an analysis was carried out using Monte Carlo simulations.

The most important results obtained from the calculations are:

- The 6,000 canisters can be accommodated within the studied area at depth of 500 m (the probability is 100%).
- The average area needed to host 6,000 canisters at a depth of 500 m is 2.63 km^2, including central area.

The four deposition areas used for the basic layout holds enough space to accommodate the repository even in cases where a larger area is needed for deposition for different reasons. The parameters that affect the area needed for deposition are the required distance between deposition holes and the percentage loss of deposition holes. Those are also the parameters that have a considerable impact in the sensitivity analysis.

The sizes of the four depositional areas used for the basic layout are affected by four parameters in the Monte Carlo simulation. These parameters are:

- whether the medium confidence deformation zone ZSMNS046A exists or not
- the position of two of the limiting lines for the area, which are directly depending on the dip of the deformation zones ZSMNS001C and ZSMEW002A
- whether respect distance (100 m) or margin for excavation (20 m) should be applied to the deformation zones ZSMEW007A and ZSMNW932A, i.e. whether the zones are longer than 3 km or not
- whether the low confidence deformation zones ZSMNW170A, ZSMNE043A and ZSMNE138A exists at a depth of 500 m or not

Sammanfattning

Inledning

Denna rapport beskriver bergprojekteringen, avseende ett djupförvar i Laxemar, som utförts under projekteringssteg D1 och baseras på Platsbeskrivning Laxemar v1.2 (SDM v1.2).

Enligt gällande planer för det svenska kärnkraftprogrammet är det minsta antalet kapselpositioner i slutförvaret bedömt till 4 500 stycken. Med hänsyn till rådande osäkerheter i möjlig förlängning av kärnkraftsverkens driftsperiod, har SKB beslutat att denna studie skall förutsätta att 6 000 kapslar kommer att placeras i slutförvaret.

SKB har utarbetat en handledning ”Underground Design Premises” (UDP) /SKB 2004a/ för projektering av slutförvaret. I den anges de huvudsakliga målsättningarna för projekteringssteg D1:

- bedöma om slutförvaret ryms inom det studerade området,
- identifiera platsspecifika anläggningskritiska parametrar,
- testa och utvärdera den projekteringsmetodik som beskrivs i /SKB 2004a/,
- ge återkoppling till:
  - projekteringsorganisationen avseende behovet av ytterligare studier,
  - platsundersöknings- och modelleringsorganisationen avseende behov av ytterligare undersökningar,
  - organisationen för säkerhetsanalys.

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

Möjliga platser och preliminär bedömning att rymma slutförvaret

Möjlig placering av ett tänkt djupförvar har definierats av SKB att vara inom Laxemar området. Laxemarområdet utgör tillsammans med Simpevarpsområdet de områdena i Oskarshamn som platsundersökningarna sker i. Denna rapport behandlar endast studier för Laxemarområdet. Utbredningen av Laxemarområdet visas i figur 3-2 i denna rapport och begränsas av följande gränslinjer:

- I öster, av deformationszon ZSMNE005A (Äspö skjuvzon) och delvis av nationella intressegränsen för slutförvar /SKI 2004/.
- I norr, av deformationszon ZSMEW002A (Mederhult zonen).
- I väst, deformationszon ZSMNS001C.
- I syd, helt av nationella intressegränsen för slutförvar /SKI 2004/.

Områdets bergmassa består av finkornig kristallint urberg. De varierande bergarterna inom området har likartade sammansättning och olikheterna finns på nivån kornstorlek och i färg. För Laxemars projektering, har enligt /SKB 2006/ följande bergdomäner beaktas:

- Bergdomän A: huvudsakligen Ävrö granit.
- Bergdomän B: huvudsakligen finkornig diorit.
- Bergdomän BA: blandning av Ävrö granit och finkornig diorit.
- Bergdomän D: huvudsakligen kvartsmonzodiorit.
- Bergdomän M: en stor andel diorite/gabbro i Ävrö graniten eller kvartsmonzodiorit.

Bergdomänernas omfattning och utbredning varierar med djupet. Djupen som har studerats är mellan 400 och 700 m.

Den preliminära bedömningen som görs i kapitel 3 visar att Laxemarområdet har en tydlig möjlighet att rymma djupförvaret. Faktorn P varierar mellan 2,76 och 2,91 med hänsyn tagen till deformationszoner med både medel och hög konfidens samt inom alla studerade djup. Ett P-värde över 1,0 anger att platsen har tillräcklig kapacitet.

Utformning av deponeringsområden

Utformningen av deponeringsområdena redovisas i kapitel 4 och omfattar analyser av layout-egenskaper för alla tunnlar, deponeringshål, tunnlarnas orientering, analys av uppskattat bortfall av deponeringshål på grund av använda designförutsättningar /SKB 2004a/ samt en rekommendation av lämpligt förvarsdjup.

För projekteringssteg D1 har tunnelgeometrier och dimensioner rekommenderats i enlighet med Layout E /SKB 2002/, vilken även anger ett avstånd mellan deponeringstunnlar på 40 m. Minsta tillåtna kapselavstånden varierar mellan 6,7 och 7,6 m, vid djupet 400 m, samt mellan 7,8 m och 9,2 m, vid djupet 700 m. Variationerna inom djupen är en effekt av olika värden på värmeledningsförmågan i de olika bergdomänerna.

Analyserna för optimal tunnelorientering med hänsyn till vatteninläckage i depositionstunnlar och -hål ger endast en minimal fördelaktig orientering. Den fördelaktiga tunnelorienteringen är dock N132°.

Sannolikheten för spjälkning av bergmassan är försumbar ner till 500 m djup, oavsett tunnel-orientering. Vid djupet 600 m och för den mest ogynsamma tunnelorienteringen är sannolik-heten för spjälkning 40 %, enligt /Martin 2005/, dock är volymen på de enskilda utfallen små. Hur som helst, den mest ogynnsamma tunnelorienteringen, med djupet, är vinkelrät mot största huvudspänning.

Någon optimal tunnelorientering med avseende på kilutfall är också minimal, men analysen ger en fördelaktig tunnelorientering på ca N12° oberoende av djup.

Den utförda analysen av SKB avseende bortfallet av deponeringshål på grund av långa sprickor som träffar hålen har endast redovisats som ett värde, till 16 % oberoende av djup. Motsvarande bortfall på grund av för högt vatteninflöde i hålen, med maximum kriteriet 10 l/min, varierar mellan 1,5 och 6 %, beroende på bergdomän.

Analysen av kilutfall i deponeringshål ger en bortfallsandel på 5 %, vid det givna volyms-kriteriet 0,15 m^3. Vid en granskning av kilutfallet i borrade deponeringshål i Äspö Laboratoriet, som finns i närheten av Laxemarområdet, var det naturliga utfallet noll. Denna erfarenhet har beaktats vid bedömningen av bortfall på grund av kilutfall.

Studien av /Martin 2005/ visar på en sannolikhet för spänningsinducerade spjälkning i depone-ringshålen är 5 %, vid djupet 500 m, men volymen på de enskilda utfallen är små. Vid djupet 600 m är sannolikheten för spänningsinducerade spjälkningar betydligt högre, 40 %, men även dessa enskilda volymutfall är små.

Vid en sammanvägning av samtliga designförutsättningar framgår det klart och tydligt att slutförvaret skall placeras så ytligt som möjligt, dvs på djupet 400 m. Det finns emellertid betydande faktorer för långtidssäkerheten som inte beaktats i UDP:n /SKB 2004a/. Eftersom fördelarna med att placera slutförvaret på djupet 400 m i jämförelse med den ursprungliga referensnivån 500 m är marginella, har referensnivån 500 m behållits i det fortsatta layoutarbetet för Laxemar D1.

Layoutstudier

I kapitel 5 redovisas layoutstudierna, två alternativ för varje tänkbart förvarsdjup 500 och 600 m. Layoutstudierna baseras på analyser och resultat från föregående kapitel samt kravet på en kapacitet för 6 000 kapslar med beaktande av bortfallet av deponeringshål. Layouten möjliggör deponering av ca 200–400 kapslar för den inledande driften, vilka inkluderas i kapacitetskravet på 6 000 kapslar.

De två alternativen på djupet 500 m är de två mest fördelaktiga. För de två alternativen på 500 m är bortfallet uppskattat till 20 % jämfört med 600 m alternativen där bortfallet uppskattas till 40 %. Detta innebär en betydligt mindre volym bergschakt för 500 m alternativen.

I det ena alternativet på 500 m ligger centralområdet placerad i syd-västra delen av Laxemar-området och i det andra alternativet i centrala delen av området. Det centrala alternativet bedöms vara det mest fördelaktiga, främst med hänsyn till fördelar för ovanjordsanläggningen, och har därför valts som baslayouten.

Den totala volymen bergschakt, inklusive deponeringsområden, central området, ramp och vertikalschakt, för baslayouten uppskattas till ca 2,5 miljoner kubik (exkluderat deponeringshålen) vilket inkluderar 72 km tunnlar och 7 500 deponeringshål.

Identifiering av passager genom deformationszoner

Analyser av passager genom deformationszoner presenteras i kapitel 6. Totalt har sex passager och en reservpassage analyserats.

För huvuddelen av passagerna förväntas inga problem vid tunneldrivningen genom deformationszonerna. I enstaka passager kan höga vatteninflöden förväntas med konsekvensen att tunneldrivningen måste anpassas till zonen och att omfattande injekteringsinsatser kan förväntas. Många alternativa drivningssätt, inklusive bergförstärkning, och injekteringsmetoder finns möjliga att utnyttja och skall finnas i beredskap inför passagerna.

Inläckage och hydrogeologiska situation kring förvaret

I kapitel 7 redovisas analyserna av inläckaget och den hydrogeologiska situationen runt slutförvaret.

Analyserna av inläckaget in till förvaret visar på en stor variation i resultat beroende på beräkningsmetod, tätningsnivå samt utbyggnadsskede. Resultaten från de analytiska beräkningarna ger ett inläckage mellan 0,2 och 29 l/s beroende på utbyggnadsskede. Motsvarande resultat från de numeriska beräkningarna är 19–37 l/s.

Grundvattensänkningens influensområde, baserat på den numeriska beräkningen, kommer att bli betydande. För de båda tätningsnivåerna, med ett tätningskriterium 10^{-7} m/s respektive 10^{-9} m/s, fås ett område på ca 10 km^2, för en grundvattensänkning på 0,3 m eller mer.

Den numeriska analysen visar på en liten möjligt saltvatteninträngning i slutförvaret.

Uppskattning av tätningsinsatsen

Uppskattningen av tätningsinsatsen redovisas i kapitel 8. Den totala injekteringsmängden som injekteras i bergmassan, inklusive all hålvolym, uppskattas till 1 700–2 950 m^3 för tätningsnivå 1 (K=10^{-7} m/s) och 4 950–9 650 m^3 för tätningsnivå 2 (K=10^{-9} m/s). Deponeringstunnlarna dominerar tätningsbehovet med 1 200–1 900 m^3 för tätningsnivå 1 ($K=10^{-7}$ m/s) och 3 600–6 900 m^3 för tätningsnivå 2 ($K=10^{-9}$ m/s), inklusive all hålfyllnad i tunnlarna.

Den analyserade och uppskattade mängden injektering innehåller stora osäkerheter med ett antal villkor och subjektiva uppskattning, vilka har stor betydelse för den prognostisera mängden. Dessutom är anläggningen stor och komplex så att enstaka avvikelser i beräkningsförutsättningarna får stor betydelse på den totala mängduppskattningen.

Det är viktigt att begränsa pH värdet i bergmassan runt förvaret, med KBS-3-konceptet, för att inte 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 alternativ bruk i den framtagna injekteringsmetodiken.

Uppskattning av bergförstärkningsinsats

Uppskattningen av bergförstärkningsinsats redovisas i kapitel 9. En preliminär uppskattning av nödvändiga förstärkningsmängder i förvaret har gjorts. Den total mängden bultar i hela anläggningen är beräknad till mellan 145 000 och 189 000 stycken, varav ca 102 000–133 000 stycken i deponeringstunnlar. Den totala mängden fiberarmerad sprutbetong är mellan 12 000 och 19 000 m^3. Endast mellan 400 och 2 000 m^3 av denna fiberarmerad sprutbetong är beräknad i deponeringstunnlar. I stället kommer nätning i huvudsak att användas i deponeringstunnlarna. Mängden nätning i deponeringstunnlar uppskattas till mellan 219 000 och 293 000 m^2. En mindre mängd, ca 20 m^3, oarmerad sprutbetong är beräknad för andra tunnlar och bergrum.

Den erforderliga mängden bergförstärkning påverkas av till stor del av i vilken omfattning de spänningsinducerade spjälkningsbrott får, det förutsätts i mängdberäkningarna att spjälkningsbrott sker i mindre än 5 % av tunnellängden. Om ingen typ av spjälkningsbrott inträffar, kommer mängden sprutbetong sannolik att reduceras i betydande grad. Även mängden bultar kommer troligtvis att reduceras, dock ej i samma utsträckning som sprutbetong.

Betongen, dvs både sprutbetong och bultbruk, skall ha ett lågt pH, enligt säkerhetsanalyserna, för att unvika en för hög alkalisk miljö i bergmassan runt förvaret.

Teknisk riskbedömning

En teknisk riskbedömning har gjorts och är redovisad i kapitel 10. Det huvudsakliga syftet med riskbedömningen var att svara på frågan ”Kan slutförvaret rymmas inom anvisat område?”. En modell som beaktar variationer för ett antal parametrar, vilka påverkar det tillängliga området för slutförvaret, har utvecklats. Analysen utfördes med hjälp av Monte Carlo simulering.

De mest betydande resultaten som erhållits från beräkningarna är:

- 6 000 kapslar ryms inom det studerade området på djupet 500 m (sannolikheten är 100 %).
- Den genomsnittliga area som behövs för att rymma 6 000 kapslar på ett djup av 500 m är 2,63 km^3, inkluderat centralområdet.

De fyra depositionsytorna som utnyttjas i baslayouten är tillräckliga för att rymma förvaret även då osäkerheterna i indata parametrar beaktas vid analys av nödvändig deponeringsarea. De parametrar som främst påverkar nödvändig deponeringsarea är avståndet mellan deponeringshålen och andelen bortfall av deponeringshål. Dessa två parametrar har, enligt känslighetsanalysen på Monte Carlo simuleringen, även en betydande inverkan på resultat av hela riskbedömningen.

Total storleken av de fyra deponeringsytorna som utnyttjas i baslayouten påverkas enligt Monte Carlo simuleringen främst av fyra parametrar:

- Om deformationszon ZSMNS046A, med en medel konfidensgrad, förekommer på förvarsdjupet eller ej.
- Två av begränsningslinjernas läge på förvarsdjupet 500 m, dvs lutningen på deformationszon ZSMNS001C och ZSMEW002A.
- Om respektavstånd (100 m) eller byggavstånd (20 m) skall användas för deformationszon ZSMEW007A och ZSMNW932A, dvs är dessa zoners längd mer än 3 km eller inte.
- Om deformationszon ZSMNW170A, ZSMNE043A och ZSMNE138A, med en låg konfidensgrad, förekommer på förvarsdjupet eller ej.

Contents

1 Introduction

1.1 Objectives

SKB is currently planning for the construction of a final repository for disposal of spent nuclear fuel and radioactive waste 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 Tyrens AB and NGI (Norwegian Geotechnical Institute), to meet the goals for design step D1 of a final repository at the Laxemar 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. From these guidelines the following basic objectives for the Layout D1 design can be summarized:

The main objectives of rock engineering during design step D1 should be to:

- 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 needs 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 the surface facility
  - the location and extent of the underground facility
  - 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 stages 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 two key issues:

- To identify suitable areas 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 reporting is:

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

The site investigation data are submitted in consecutive parts (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 investigation 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 SDM v 1.2. 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 D1 design step.

The working strategy for the design team to partly use reports that are not quality controlled and reviewed 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 SDM v 1.2 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.

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

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 UDP is divided in several design tasks for various technical issues (cf 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 applied 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 i nvestigations, 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. The following individuals from Tyrens and NGI have contributed to the design work:

Tyrens:

- Barbro Karlsson: Layout and CAD-operator (2D/3D)
- Bengt Hansson/Martin Bergström*: Project manager and co-ordinator Tyrens
- Thomas Janson: Rock engineer, design and technical responsible
- Jakob Magnusson: Hydrogeology and design
- Martin Bergvall: Sensitivity analysis and hydrogeology
- Thomas Andersson: Rewier
- and co-workers

* Bengt Hansson between May 2005 to Dec 2005 and Martin Bergström between Jan 2006 to June 2006.

NGI:

- Fabrice Cuisiat and Elin Skurtveit: DFN-analyses
- Roger Olsson: Rock engineer and co-ordinator NGI
- Eystein Grimstad: Rock support

- Tore Valstad and Panayiotis Chryssanthakis: Rewiers
- and co-workers.

Graham Ainscough: Translation to English, Chapter 3 to 6 and 8 to 10.

The design work has been carried out according to systems for quality assurance from Tyrens AB.

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 v 1.2	Preliminary site description Laxemar area – version 1.2. /SKB 2006a/.
SI	Site investigation phase. The site investigation lasts until the construction and detailed characterization phase and includes the time taken by the authorities to process the siting application with respect to the Environmental Code and the Law of Nuclear Activities.
UDP	The document "Underground Design Premises, Edition D1/1" /SKB 2004a/.

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 that embraces the site investigation phase, up to submission of a siting application.
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 Figure 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.
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.

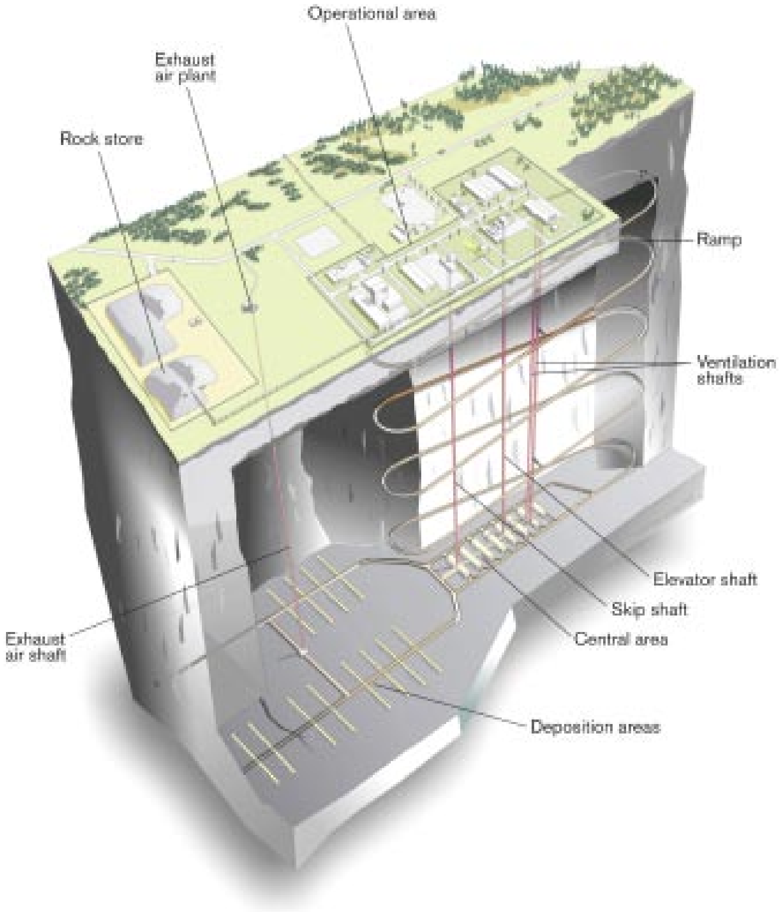


Figure 1-1. *3D illustrations of surface and underground facilities.*

Canister	Circular 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.

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.

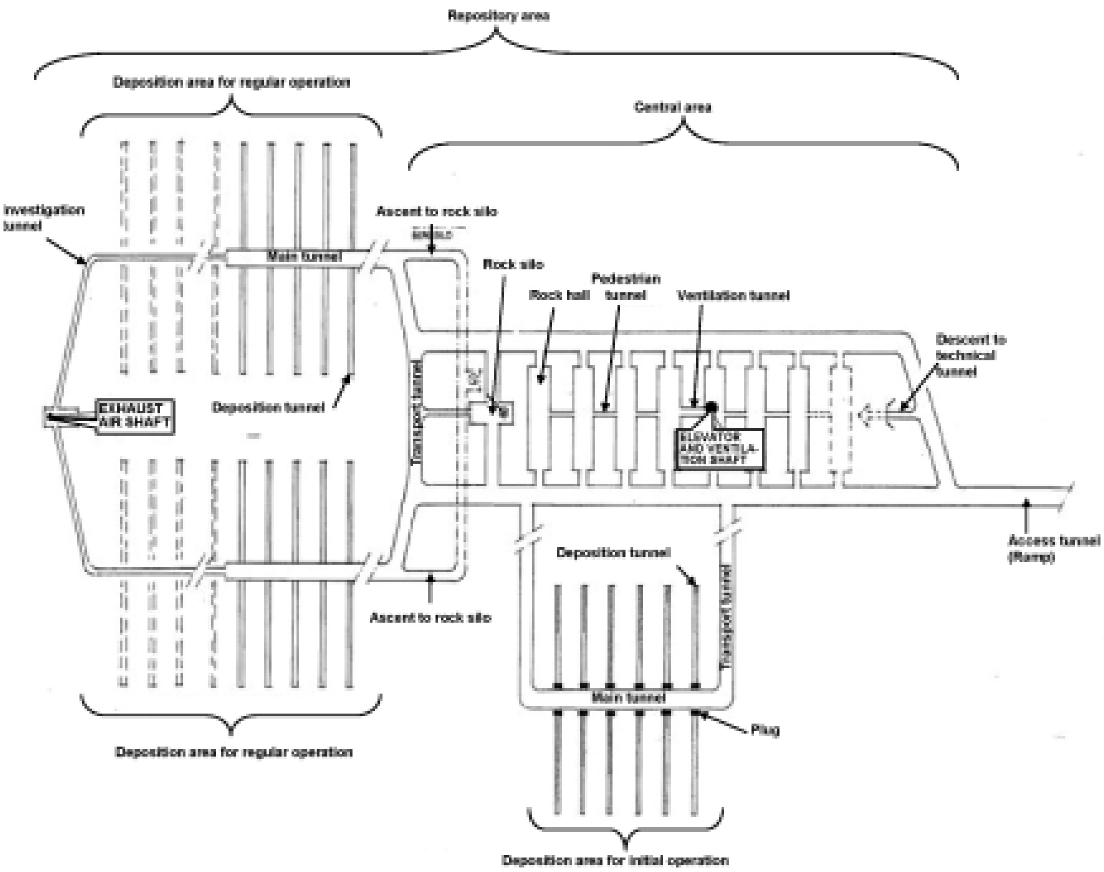


Figure 1-2. *Schematic plan showing certain parts and underground openings.*

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 /see Andersson 2003/.
Respect Distance (RD)	The minimum permissible distance between a deposition hole and a zone with a trace length of 3000 m or more, due to anticipated future seismic events on canister integrity /SKB 2004b/.
Margin for Excavation (MFE)	The minimum distance between a particular deformation zone and a deposition tunnel or cavern excavation 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 applied in this study is in detail described in the UDP (Underground Design Premises) /SKB 2004a/, and below the 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?

The work of design task A and B are presented in Chapter 3.

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 as regards maximum permissible temperature on the canister surface?

   C3. What orientation may be suitable for deposition tunnels as regards water seepage and stability in deposition tunnels and deposition holes?

   C4. What number of deposition holes may be unusable as regards the minimum permissible distance to stochastically determined fractures, excessive water inflow and 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?

The works of design task C1 to C5 are presented in Chapter 4.

D. How can the other underground openings, especially the central area's rock caverns, be designed as regards stability, and the equipment and activities they have to accommodate?

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

The works of design task E is presented in Chapter 5 and partly design task D.

F. What deformation zones might be intersected with proposed layouts and what difficulties could be expected to arise?

The works of design task F is presented in Chapter 6.

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?

The works of design task G is presented in Chapter 7.

H. How much grouting might be required?

The works of design task H is presented in Chapter 8.

I. How much rock support might be required?

The works of design task I is presented in Chapter 9.

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?

The works of design task K is presented in Chapter 10.

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.

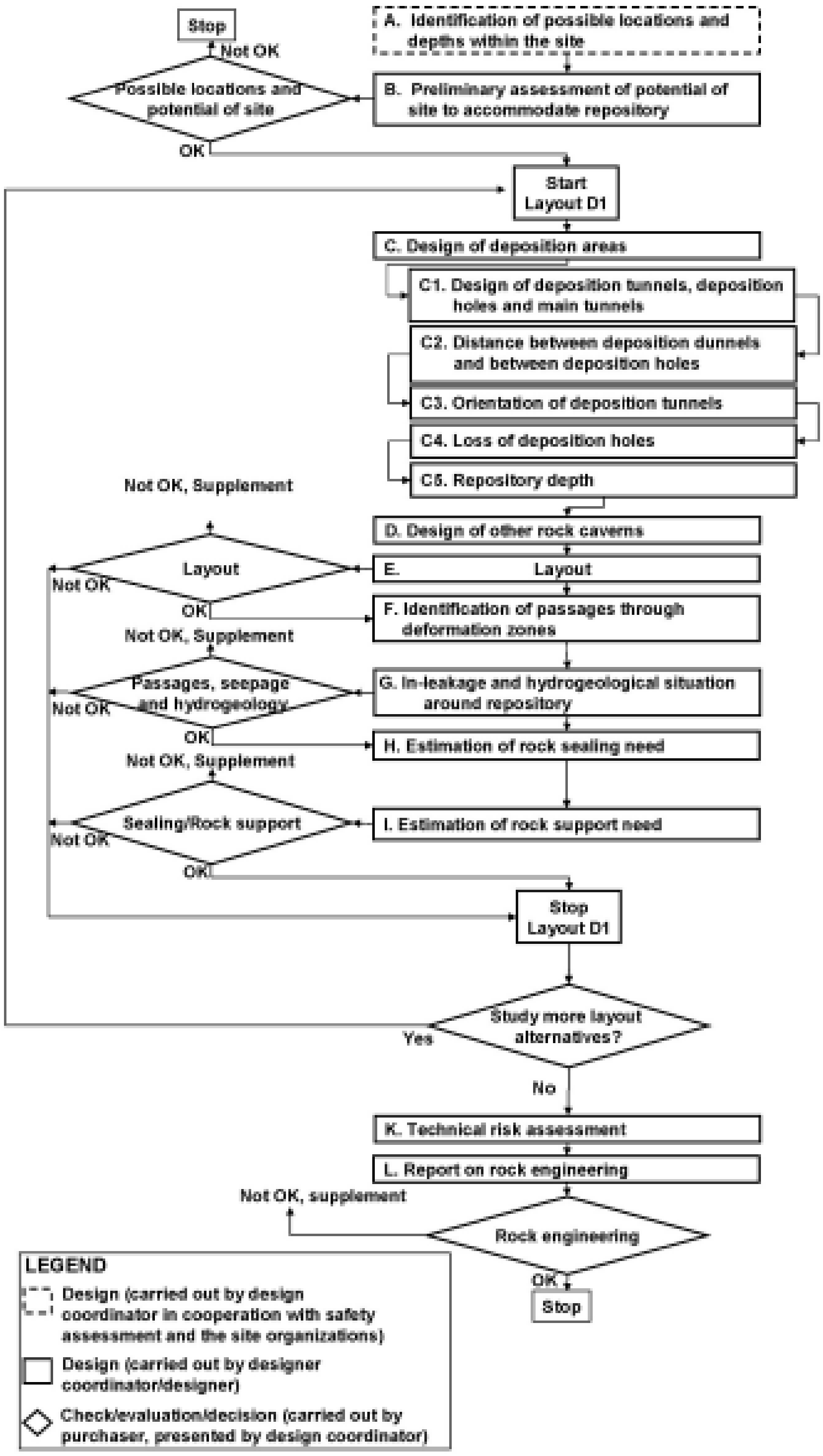


Figure 2-1. *Design methodology, logical framework.*

2.2 Site specific key issues

Prior to the commencement of the design task the following issues were identified by SKB as site specific key issues for the Laxemar site, having a strong influence on the accessible area for deposition and the layout of the repository:

- High number of deterministically determined fracture zones.
- High number of stochastically determined fracture/fracture zones with radius 50 m < r < 600 m.
- A high variability in hydraulic conductivity.
- A low and fluctuating thermal conductivity.

Site specific key issues are further identified and analysed in the individual analyses, 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 v 1.2 shall be the basis for the Layout D1 design /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 SDM v 1.2 results would not be conservative. The influence on respective design task concerning new data not applied in the 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 v 1.2 /SKB 2006a/.

Design task	Chapter in this report	Preliminary data used	Final SDM v 1.2 data	Estimation of influence from new data	Analysis rectified Yes/No
B, E and K	3.2, 5.3.1 and 10	ZSMNE138B LAX1.2-LOC-DZ, Deformation zone model, Preliminary delivery (april 2005).	Zone was not included in final version.	Negligible effect.	No
C2 and E: Distance between deposition holes	4.2	The thermal conductivity for Rock Domain M is based on TPS (Transient Plane Source method) measurements.	The thermal conductivity for Rock Domain M is based on density logging.	Considered in Chapter 10	Yes
C4: Potential wedge breakout in deposition holes	4.4	The analysis is based on preliminary DFN (June 2005)	New DFN-data in final version.	Neglectable effect	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 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.

For the design of the repository, Chapter 5, the easternmost part of the studied area was excluded. This concerned area is everything located east of "Kustvägen" which is the road striking from north to south in the east part of Laxemar subarea (Figure 3-1). This area was excluded due to environmental issues on the surface. The excluded area is also shown in Figure 5-4 to Figure 5-7.

Loss of deposition holes due to stochastically determined (elongated) fractures/fracture zones is as described in Section 4.4 calculated according to /Hedin 2005/. The analytical calculations were prepared by SKB, and the design team was instructed to apply a loss rate of 16% 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/.

Numerical simulations of the seepage to the repository and the hydrogeological situation around were reported in /Svensson 2006/.

During completion of this design report, finding in parallel ongoing studies, Preliminary assessment of long term safety for KBS-3 repositories at Forsmark and Laxemar /SKB 2006b/, 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 allowed for a 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.

It was decided by SKB that deterministically determined deformation zones with a low level of confidence should not be included in the design. The effect of the low confidence zones are however studied in a separate sensitivity analysis in Chapter 5.

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 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/ is not meaningful, or ought to be carried out differently. The most important 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/ and section below	Deviation from /SKB 2004a/	Justification
C2: Distance between deposition holes	4.2	Sensitivity analysis, see Section 5.4.2.	Addition in sensitivity analyses; percentile 2.5% and 97.5% and initial temp difference 1.0 to 1.5°C	Instruction from SKB modeling, due to too high uncertainties in the distribution of the thermal conductivity.
C3 and C4: Orientation of deposition tunnels and loss of deposition holes, DFN analysis	4.3	The analyses should include the rock domains and depths with the range 400–700 m, see Section 5.4.3 and 5.4.4	The analyses were done for two domains at one depth (500 m)	Only one level was analyzed as the hydraulic conductivity of the hydro-DFN has no depth dependency below 300 m
C3: Orientation of deposition tunnels, DFN analysis	4.3	P33 shall be reported as mean on the 20 m scale, see Section 5.4.3	One value was used for the entire deposition tunnel.	After discussion with SKB
C4: Orientation of deposition tunnels, DFN analysis	4.4	A zone around the actual tunnel with a suggested volume of 300 m × 400 m × 500 m should be used, see Section 5.4.4.	The model volume is divided into two areas, one inner and one outer.	All joints are generated in the inner and larger joints in the outer volume. This gives a more flexible calculation and does not affect the results.
C3 and C4: Optimal orientation of deposition tunnels and loss of deposition holes with regard to potential instable wedges	4.3 and 4.4	The unstable rock wedges formed between the fractures and the tunnel contour, see Section 5.4.3 and 5.4.4.	The analysis is done without consideration to where the rock wedge is located.	It was not possible to perform the analysis as described in UDP when using the tools prescribed.
C3 and C4: Optimal orientation of deposition tunnels with regard to potential instable wedges	4.3 and 4.4	Recommendations regarding method and tool to be used for the analysis of the optimal orientation for the deposition tunnels with respect to the volume of instable wedges, see Section 5.4.3.	As it was not possible to analyze all the stochastic generated fractures crossing a deposition tunnel, 50 random stochastic fractures were analyzed. It was assumed that these fractures formed wedges together with the tunnel contour.	It was not possible to perform the analysis as described in UDP when using the tools prescribed.
C4: Loss of deposition holes due to fractures/fracture zones > 100 m	4.4	Fractures with 100 m < radius < 500 m should be used and have a permissible distance to deposition holes, see Section 5.4.4.	An analytical method as proposed in /Hedin 2005/ were used, with 50 m < radius < 600 m and no permissible distance.	SKB instruction
C4: Loss of deposition holes due to seepage.	4.4	Calculations according to point 1, see Section 5.4.4.3, shall be carried out for holes that have been approved	As point 1 was not analysed in this work, it has not been taken into consideration. The calculation is based on the total number of deposition holes.	Point 1 was analyzed by SKB /Hedin 2005/
C4: Loss of deposition holes due to seepage.	4.4	The analyses shall be carried out for drawdown, see Section 5.4.4.	No analyses were done regarding drawdown	After discussion with SKB
C4: Loss of deposition holes due to seepage.	4.4	Sensitivity analyses for a seepage of 1.0 l/min, see Section 5.4.4.	Sensitivity analysis were done for both 1.0 l/min and 0.1 l/min	Parallel studies /SKB 2006c/ show that piping and erosion in the buffer are possible at this flow value (0.1 l/min).

Design task	Chapter in this report	Premises according to /SKB 2004a/ and section below	Deviation from /SKB 2004a/	Justification
C4: Loss of deposition holes due to instable wedge	4.4	–	Comparison have also been made with Äspö experince	SKB instruction
E: Layout design	5	The orientation of deposition tunnels shall be chosen according to Chapter 5.4.3 in this report	This was not done as the differences between different orientations were regarded so small that strictly geometrical considerations could be used for the orientation instead.	Conferred with SKB
E: Layout design	5	Sensitivity analysis should be done for six parameters, see Section 5.6.	Sensitivity analysis was only done for four parameters.	Conferred with SKB
H: Estimation of rock grouting needed	8	Description of grouting method and mortar mixtures including additives, see Section 5.9	No description has been produced, but estimation of low pH cement volume is done and handed over to SKB:s safety analysis	Conferred with SKB
H: Estimation of rock grouting needed	8	Estimation of rock grouting for different construction stages, number of grouting screens and total number of drilling meters, see Section 5.9	No estimation has been done.	Conferred with SKB, give in this stage to detailed analysis
H: Estimation of rock grouting needed	8	The porosity and transmissivity distribution in 20 m scale, see Section 5.9 Hydrogeological data should be in mean values, see Section 5.9	Data from Chapter 4 and 7 and SDM v 1.2, 100 m scale The values in SDM v 1.2 is given as median values.	Conferred with SKB According to SDM v 1.2
I: Estimation of the need for rock support	9	–	Estimation of low pH cement volume is done and handed over to SKB: s safety analysis	Conferred with SKB
I: Estimation of the need for rock support		Requirements on materials for shotcrete according to BV Tunnel, see Section 5.10.	Requirements on materials for shotcrete according to Norsk Betongforening were also analysed.	Comparison with a international requirements

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

The investigated site, Laxemar, is situated within what has been defined as the Simpevarp area by SKB. This area is divided into two subareas. The Simpevarp subarea and the Laxemar subarea (see Figure 3-1). This report only deal with studies regarding the Laxemar subarea, and from now on it will only be referred to as Laxemar.

3.1 Possible location

The possible location for a tentative final repository has been defined by SKB to be within the Laxemar area. Laxemar is, together with Simpevarp, the area covered by the site investigation in Oskarshamn, see Figure 3-1.

The investigations in Laxemar started at the beginning of 2004 and have so far comprised geological mapping, geophysical surveys and deep core drilling. These site investigations, have contributed to clarify the basic geological conditions in the area. In order for the site to qualify as a final repository, a number of safety criteria connected with the properties of the bedrock must be satisfied. The documentation of performed investigations and the preconditions of the site are compiled in SDM v 1.2 /SKB 2006a/.

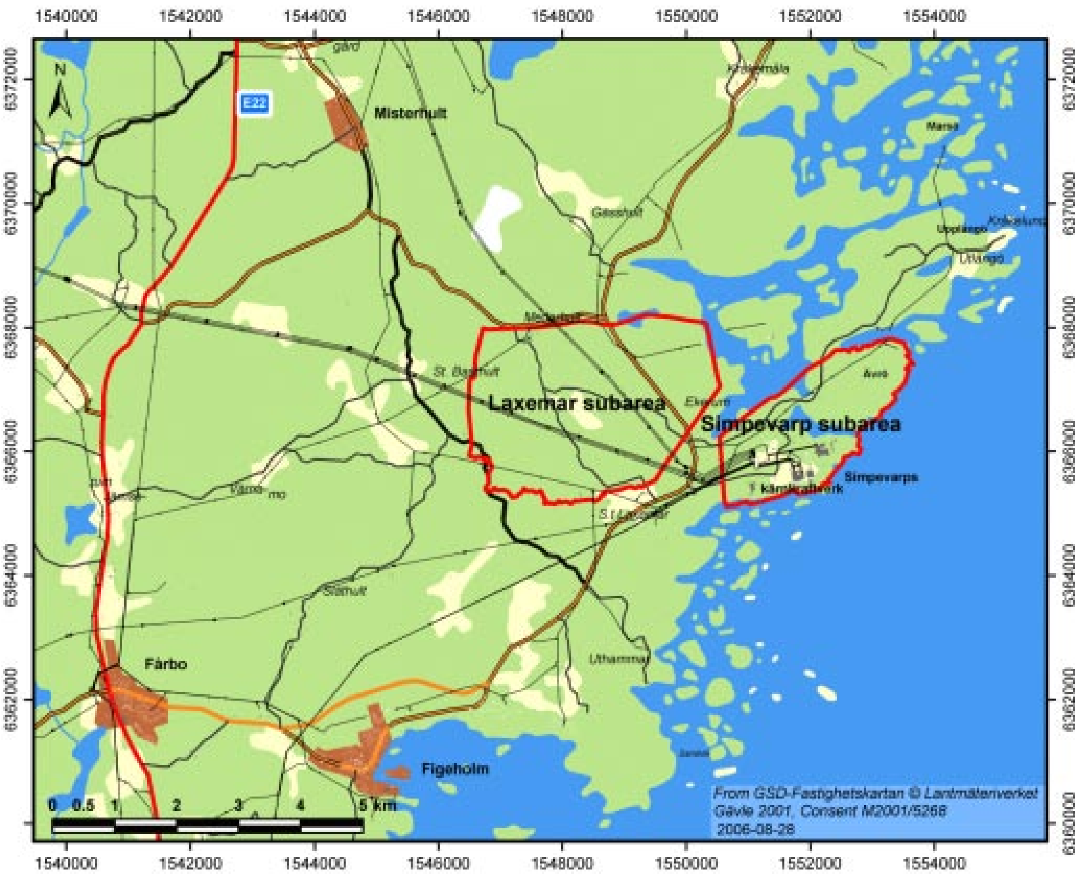


Figure 3-1. Location of the Laxemar- and Simpevarp subareas.

Geology of the area

During the course of the investigations, a number of deformation zones have been identified. The zones consist of fractured or deformed rock mass. Some of the zones have been selected as limits for the rock volumes where a final repository can be constructed together with restrictions of national interest for a final repository /SKI 2004/. The extent of the area can be seen from Figure 3-2 and the interfaces are as follows:

- To the east, Deformation Zone ZSMNE005A (Äspö shear zone) and the outer limit of national interest for a final repository.
- To the north, Deformation Zone ZSMEW002A (Mederhult zone)
- To the west, Deformation Zone ZSMNS001C
- To the south, the interface corresponds with the southern limit of national interest for a final repository /SKI 2004/.

The bedrock within the area in question consists of fine-grained crystalline basement. The various rock types have a similar composition and differ in the first instance with regard to grain size and colour. A number of 14 rock domains have been identified in the local scale in the SDM v 1.2 /SKB 2006a/. For the design work in Laxemar, the following rock domains are considered:

- RSMA (Rock domain A): Mainly Ävrö granite.
- RSMB (Rock domain B): Mainly fine-grained dioritoid.

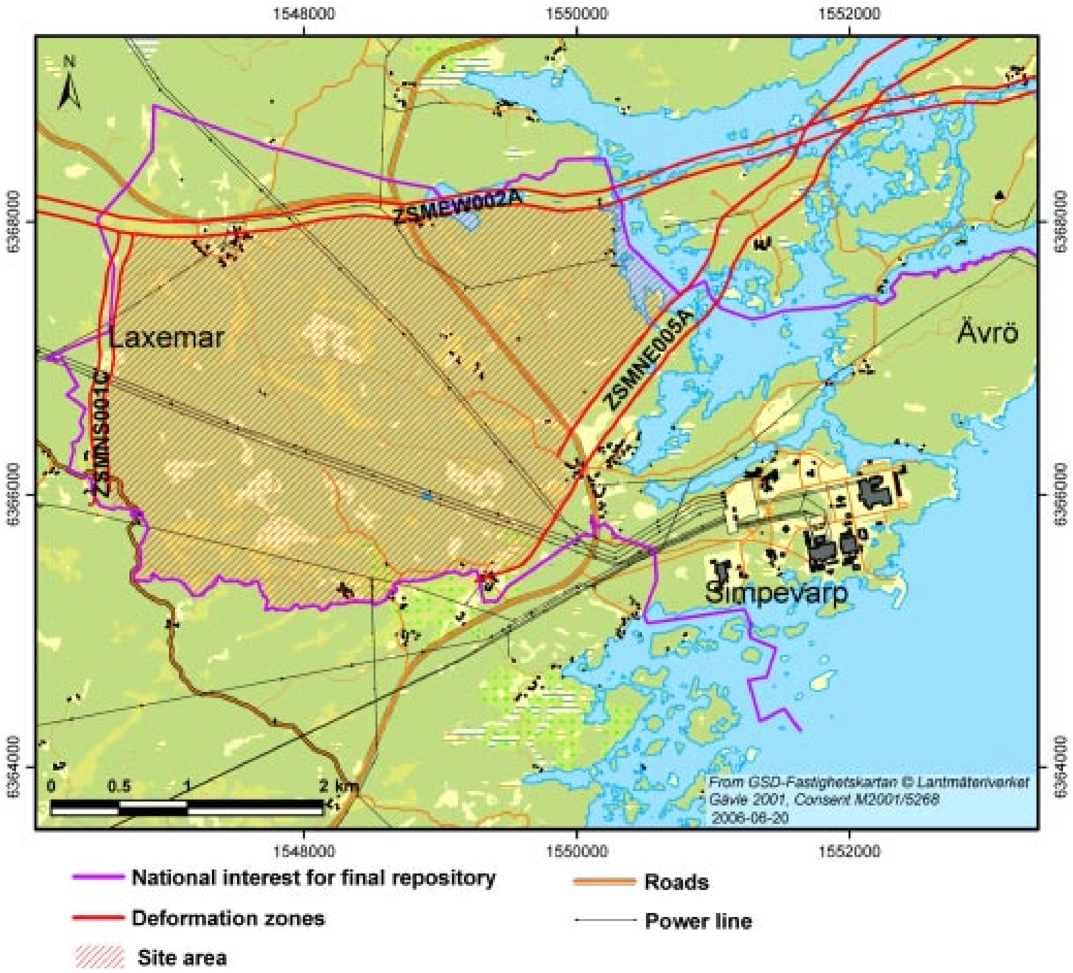


Figure 3-2. Location of the investigated area, Laxemar in Oskarshamn.

- RSMBA (Rock domain BA): A mixture of Ävrö granite and fine-grained diorite.
- RSMD (Rock domain D): Mainly quartz monzodiorite.
- RSMM (Rock domain M): A large proportion of diorite/gabbro in Ävrö granite and quartz monzodiorite.

The existence and extent of the rock domains vary with depth. The depths that have been investigated are 400 m, 500 m, 600 m and 700 m. Figure 3-3 to Figure 3-6 show the presence of rock domains at the different depths.

3.2 Preliminary assessment of potential of site to accommodate repository

In order for the site to be of interest for a final repository, the available deposition areas within the site must be sufficiently large. Available deposition areas are limited primarily by deformation zones. The deposition tunnels may not cross deformation zones that have been modelled and are more than 1 km long, so-called deterministically interpreted deformation zones /SKB 2006a/, /SKB 2004a/. For zones with more than 3 km length, a respect distance has been defined, which means that no deposition tunnel should be constructed within this distance. For zones with less than 3 km length, a margin for excavation (MFE) will be used and no deposition tunnels should be constructed within this margin.

The potential for the site to accommodate a predetermined number of canisters has been calculated by comparing the available area to the area needed to accommodate the required deposition holes.

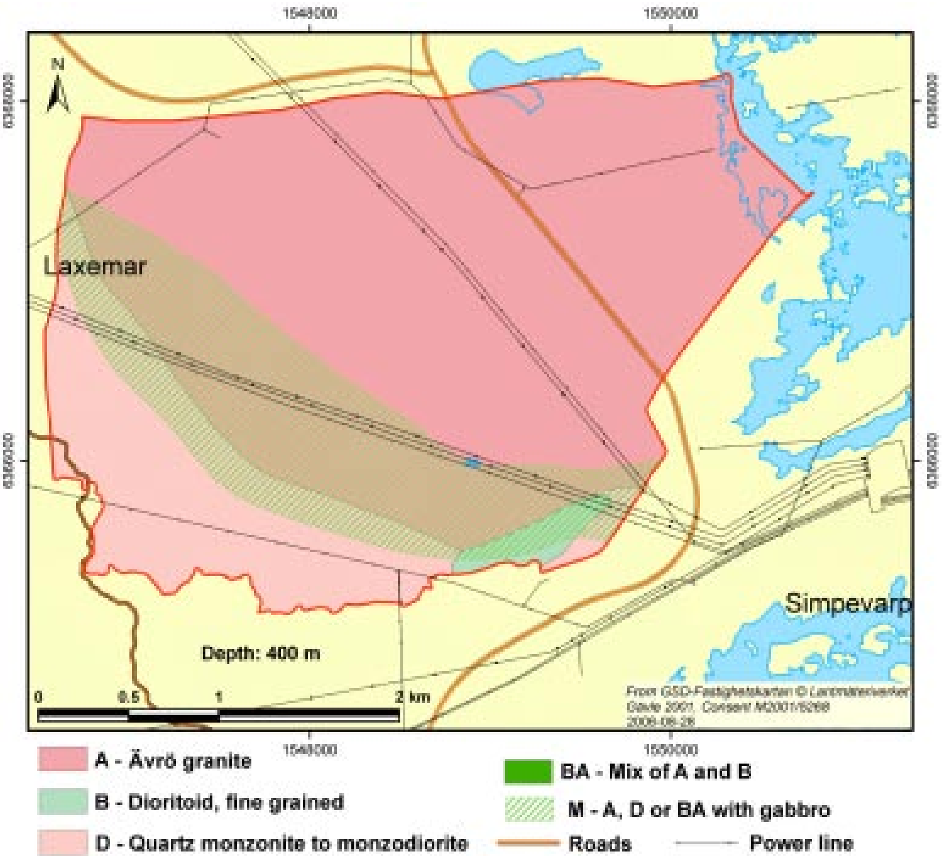


Figure 3-3. Extent of rock domains at a depth of 400 m. A = RSMA, B = RSMB, D = RSMD, BA = RSMBA and M = RSMM /SKB 2006a/.

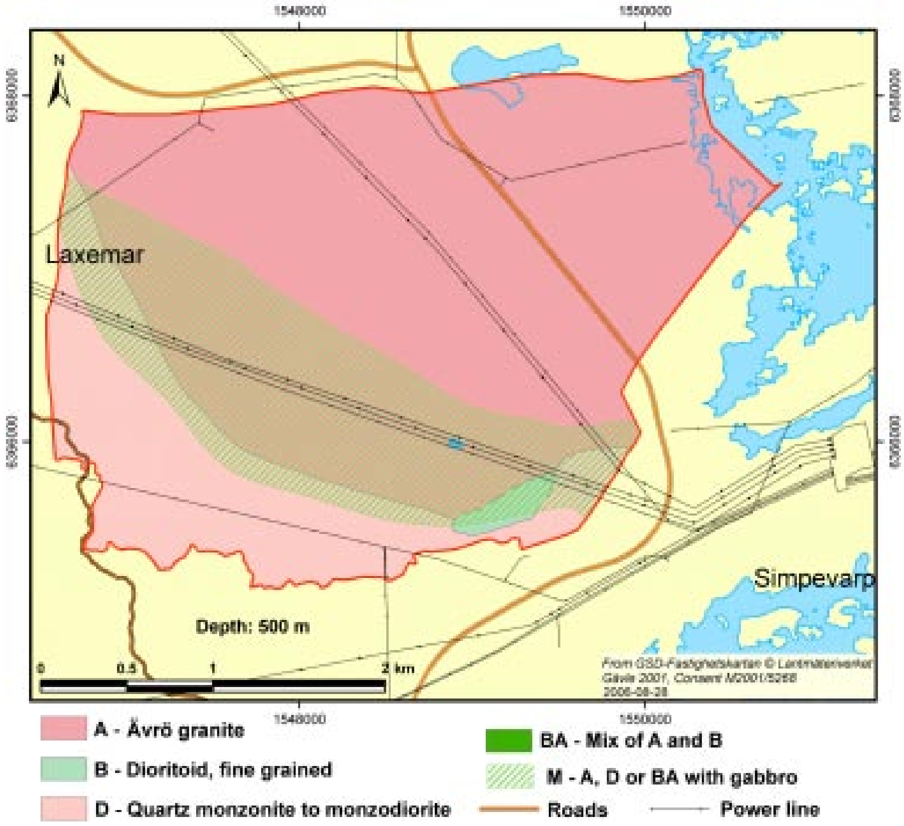


Figure 3-4. *Extent of rock domains at a depth of 500 m. A = RSMA, B = RSMB, D = RSMD, BA = RSMBA and M = RSMM /SKB 2006a/.*

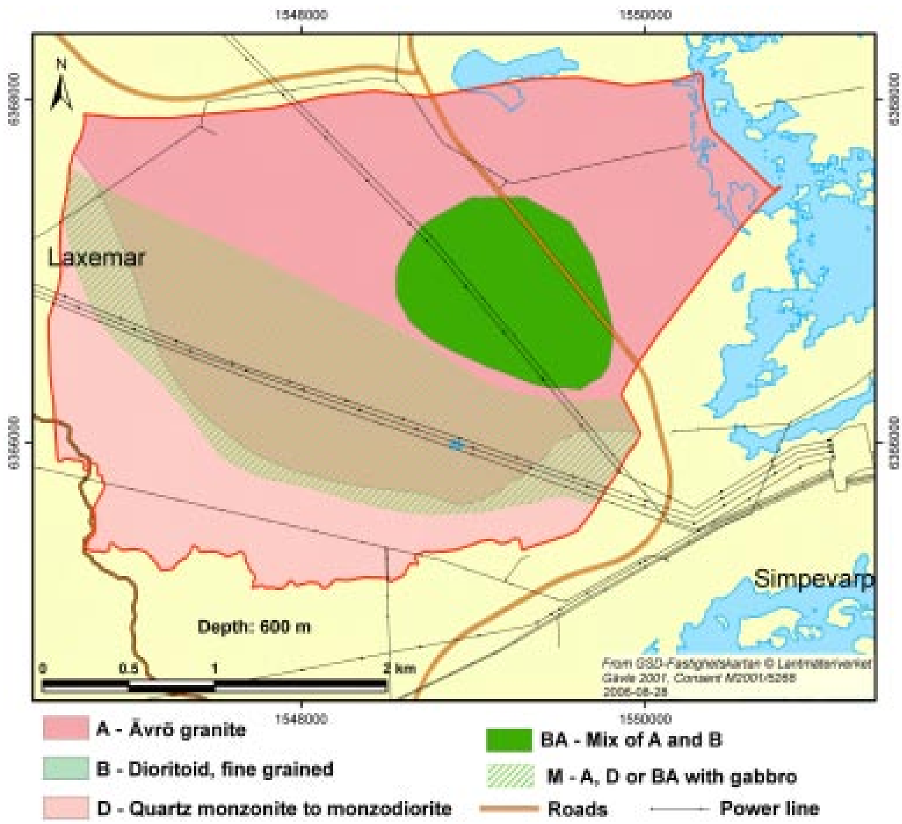


Figure 3-5. *Extent of rock domains at a depth of 600 m. A = RSMA, B = RSMB, D = RSMD, BA = RSMBA and M = RSMM /SKB 2006a/.*

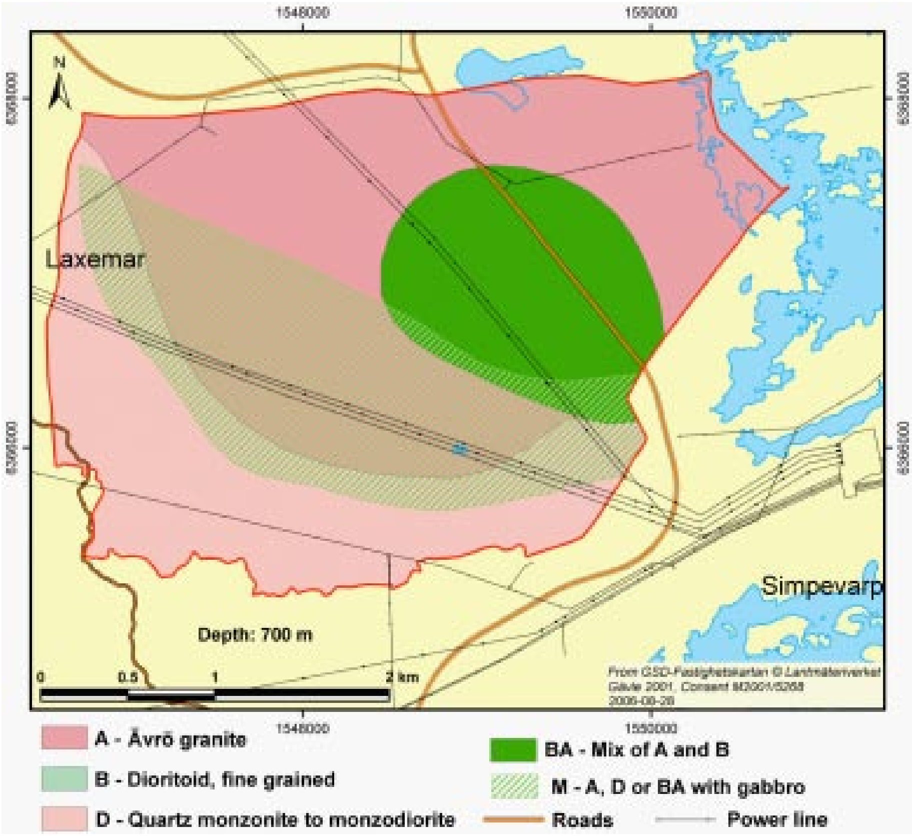


Figure 3-6. Extent of rock domains at a depth of 700 m. A = RSMA, B = RSMB, D = RSMD, BA = RSMBA and M = RSMM /SKB 2006a/.

3.2.1 Input data and assumptions

When calculating the potential of the site to accommodate the repository, consideration has been given to /SKB 2004a/:

1. the loss of deposition area as a result of preliminary respect distance to deterministically interpreted fractured zones,
2. the assumed loss of deposition holes (25%).

Distance to deformation zones

According to /SKB 2004b/, deformation zones with more than 3 km length shall have a respect distance within which no deposition tunnels should be constructed. This distance is 100 m from the centre of the zone. As a result of this it may be necessary to add an extra margin outside the zone if it is wider than 200 m. Thus there are two possible distances for zones with more than 3 km length.

For zones with less than 3 km length, no distances have been defined by SKB but there should be a margin for excavation (MFE) between deposition tunnels and zones. Some zones are not given any width in /SKB 2006a/ and for these zones an extra 10 m of margin were added because of the uncertainty of the width.

As a whole four different types of distances have been defined in this study to be used in the design depending on length and width of the zones. The four types are defined below:

1. Respect distance of 100 m from the centre of the zone applies to zones with more than 3 km length.
2. MFE of 20 m from each side of the zone applies to zones with more than 3 km length and more than 200 m width.
3. MFE of 20 m from each side of the zone applies to zones with less than 3 km length.
4. MFE of 20 m+10 m of uncertainty distance from each side of the zone applies to zones with less than 3 km length and of unknown width.

The assessments have been made for the levels 400, 500, 600 and 700 m. The total deposition area shall hold 4,500 canisters + 1,500 in reserve to account for the uncertainty that exists concerning nuclear power plant operational life-times.

Potential, P, is calculated on the following equation:

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

where:

- K = assumed percentage preliminary loss of deposition holes.
- N = preliminary number of canisters to be disposed, 4,500 pcs + 1,500 for the uncertainty of the final number.
- A_T = total available deposition area per depth.
- A_S = preliminary requisite specific area per deposition hole, assumed to be 240 m^2.

A calculated P-value < 1 means that there is a lack of space and P > 1 means the opposite, that the area has a surplus capacity and can hold more canisters.

When locating the deposition tunnels in the deposition areas, no deterministically interpreted deformation zones may be crossed. The crossing of main, transport and similar tunnels through these zones shall also be avoided as far as possible.

3.2.2 Execution

The approach for assessing the potential of the site for accommodating the repository is subdivided into two parts.

1. The first part entails superimposing on plan maps of the deterministically interpreted deformation zones and their respect distances, as well as MFE and distance for uncertainty. After this, the available deposition subarea can be determined for the respective deposition area and depth level.
2. In the second part of the work, a calculation is made of the potential for the required number of canisters to be accommodated in accordance with the specified equation (Equation 3-1).

Deformation zones

The zones that have been defined as deterministic are those that are more than 1,000 m in length /SKB 2000/.

The deformation zones are in the site description divided into three classes: those with a high confidence level, medium confidence level and low confidence level. It was decided by SKB that the low confidence deformation zones were not to be included in the design The uncertainty around the importance of the low deformation zones will be considered in Chapter 10.

The deformation zones with a high confidence level have been confirmed by means of investigations and boreholes. There is also information available on their properties to a varying extent. In all, 13 deterministically interpreted deformation zones with a high confidence level have been identified, see Table 3-1.

Deformation zones with a medium confidence level usually appear as linear structures, lineament, in terrain but otherwise have unknown characteristics. The medium deformation zones have been assumed to be completely vertical and their length corresponds to the length of the lineament in the terrain. A total of 18 deterministically interpreted deformation zones with a medium confidence level have been identified, see Table 3-2.

Interpreted zones with assigned low confidence are only supported by indirect sources of information such as lineament indications of lesser strength, either from topography, magnetics or electromagnetic methods. A total of 6 deterministically interpreted deformation zones with a low confidence level have been identified, see Table 3-3.

Table 3-1. Deterministically interpreted deformation zones with a high confidence level /SKB 2006a/.

Designation	Length (km)	Width (m)	Span-width (m)	Strike/dip
ZSMEW002A	17.8 (± 5)	20–200	20–200	090/65
ZSMEW007A	3.3 (± 0.2)	50	20–60	278/43
ZSMEW013A	4.4 (2.5–4.4)	45	20–50	085/90
ZSMEW900A	1.7 (1–2)	20	± 10	100/70
ZSMNE005A	10.5 (± 0.2)	250	50–300	060/90
ZSMNE040A	1.4 k (± 0.1)	20	5–20	030/90
ZSMNS001C	2.2	100	± 50	010/90
ZSMNS059A	5.3 (± 0.2)	50	20–60	000/90
ZSMNW042A	3.4 (± 0.1)	80	30–80	105/90
ZSMNW042B	0.8	Line	Line	–
ZSMNW929A	1.9 (± 0.1)	50	20–50	113/79
ZSMNW931A	3.9 (± 0.2)	50	50–100	165/90
ZSMNW932A	2.8 (± 0.2)	0	0–20	120/90

Table 3-2. Deterministically interpreted deformation zones with a medium confidence level. /Wahlgren et al. 2005/.

Designation	Length (km)	Designation	Length (km)
ZSMEW014A	1.2	ZSMNS046A	3.6
ZSMEW039A	2.2	ZSMNS054A	1.2
ZSMEW120A	1.2	ZSMNW047A	1.1
ZSMNE041A	1.1	ZSMNW048A	1.8
ZSMNE058A	2.1	ZSMNW051A	1.7
ZSMNE063A	1.4	ZSMNW052A	1.1
ZSMNE094A	2.9	ZSMNW119A	2.8
ZSMNE107A	1.6	ZSMNW932C	1.0
ZSMNE108A	1.5	ZSMNW932D	0.6

Table 3-3. Deterministically interpreted deformation zones with a low confidence level. /Wahlgren et al. 2005/.

Designation	Length (km)
ZSMNE043A	1.7
ZSMNE045A	1.5
ZSMNE138A	2.2
ZSMNE138B*	0.8*
ZSMNW170A	2.2
ZSMNW932B	0.4

*See Table 2-1.

The deformation zones with low confidence are not included in the design and thus not shown in Figure 3-7 to Figure 3-10.

Each zone with its type of distance, as defined in Section 3.2.1, is presented in Table 3-4.

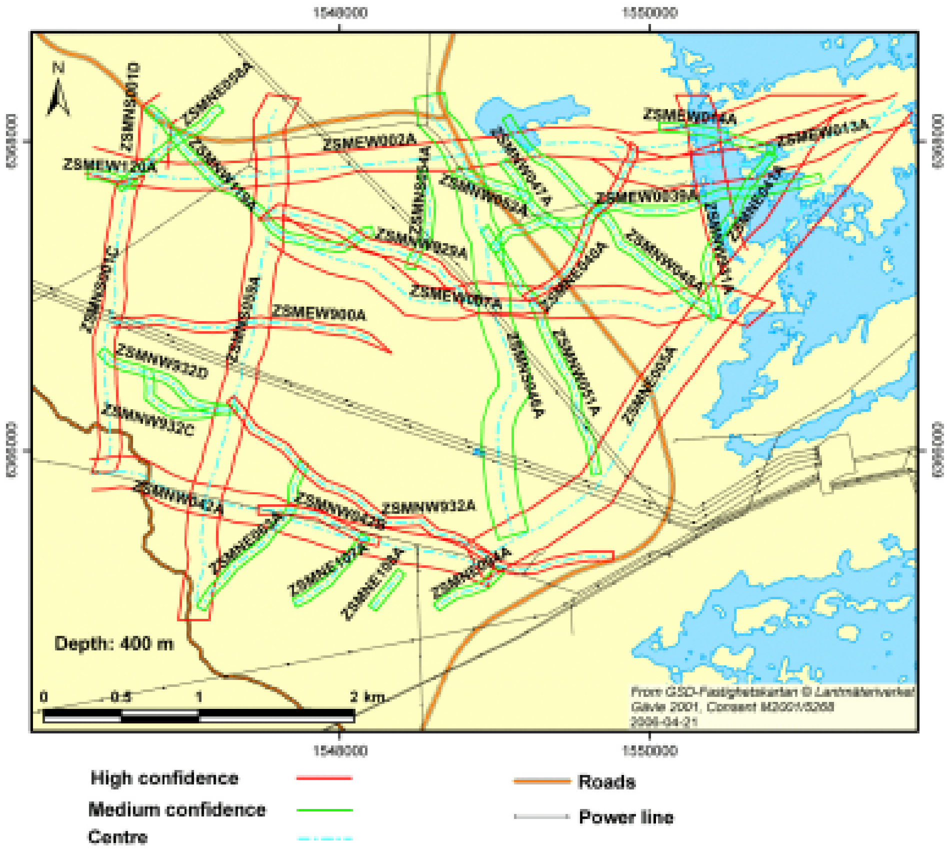


Figure 3-7. Deformation zones and their distances, depth level 400 m.

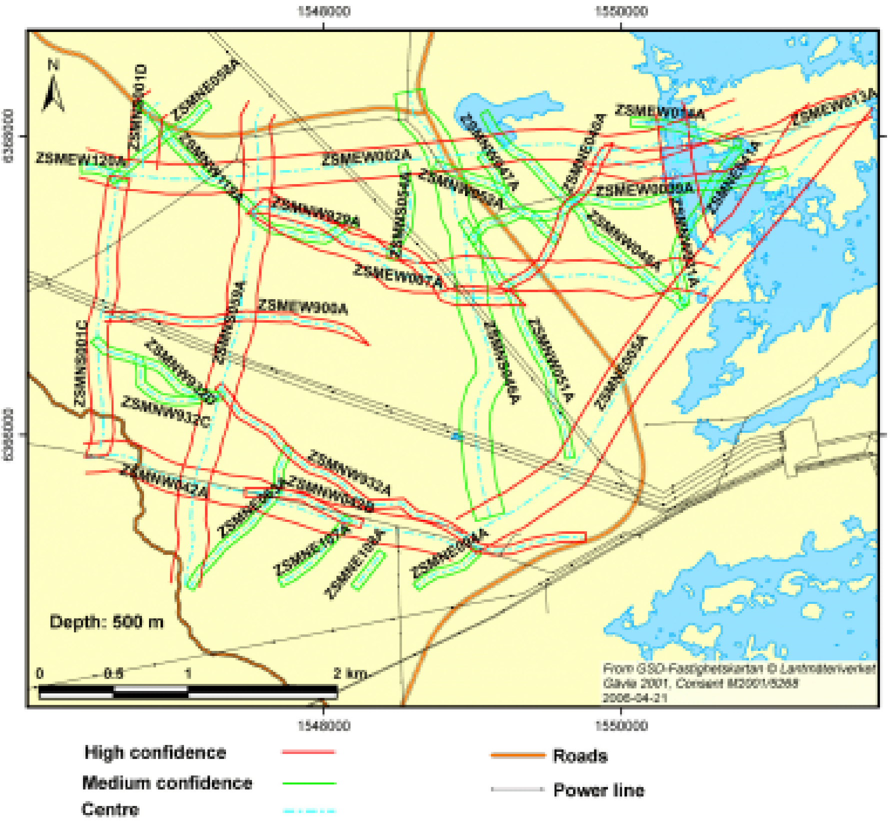


__Figure 3-8.__ Deformation zones and their distances, depth level 500 m.

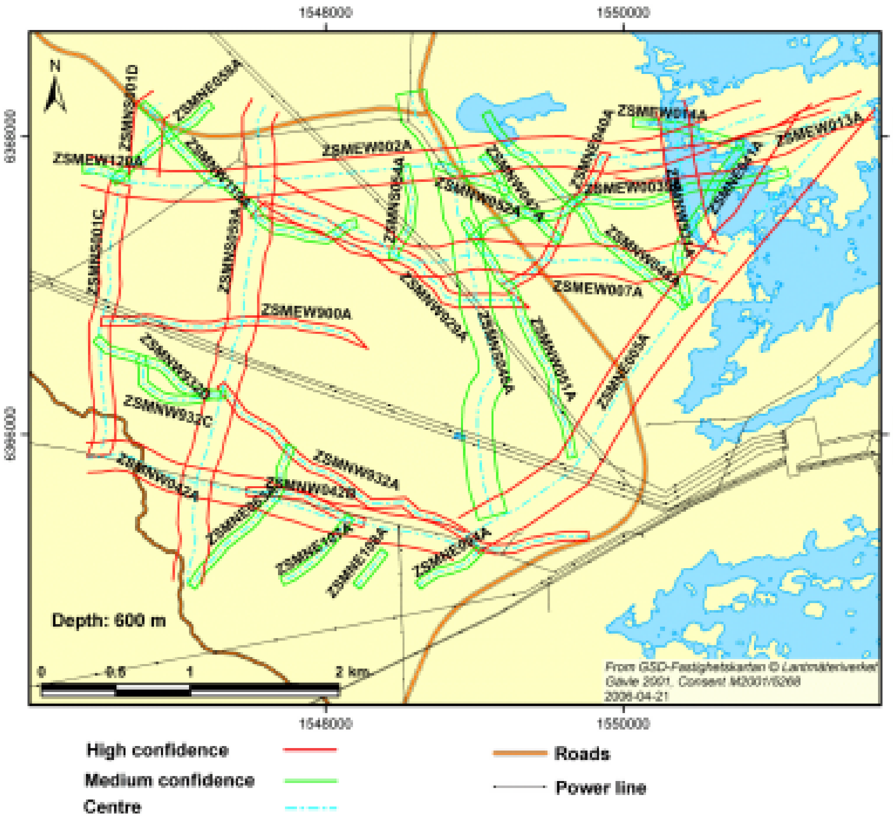


__Figure 3-9.__ Deformation zones and their distances, depth level 600 m.

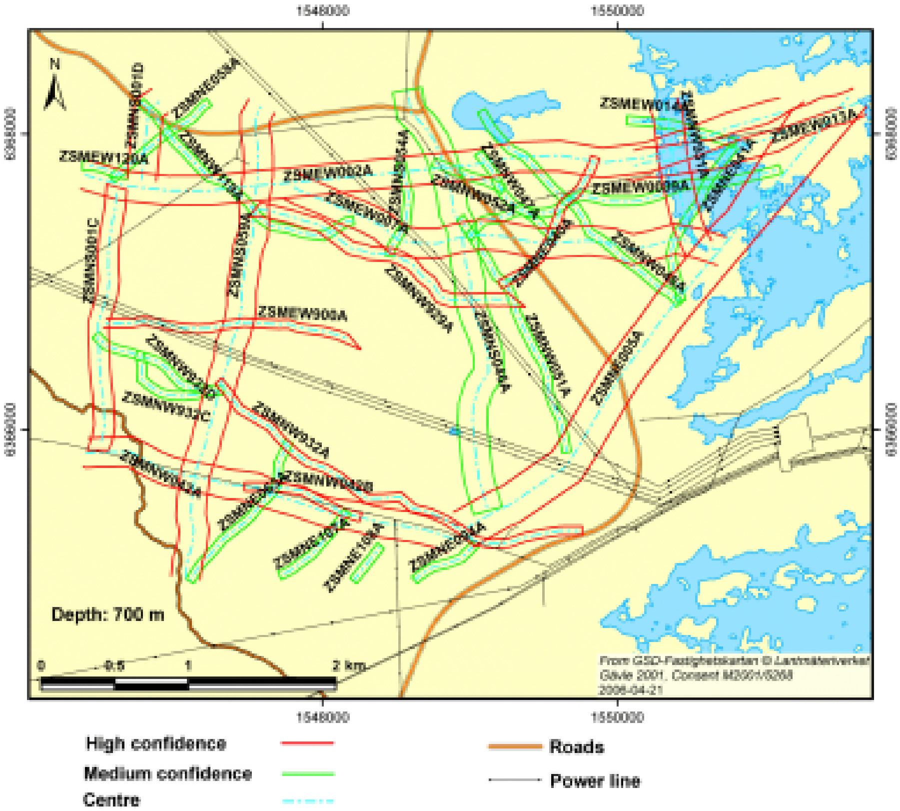


Figure 3-10. Deformation zones and their distances, depth level 700 m.

Table 3-4. Type of distance (m) for the deformation zones respectively.

Type 1 (100 m) (Respect distance)	Type 2 (20 m) (Margin for excavation)	Type 3 (20 m) (Margin for excavation)	Type 4 (30 m) (Margin for excavation + distance for uncertainty)
ZSMEW002A* (high)	ZSMNE005A (high)	ZSMEW900A (high)	ZSMNW042B (high)
ZSMEW007A (high)	ZSMEW002A* (high)	ZSMNE040A (high)	ZSMNW932A (high)
ZSMEW013A (high)		ZSMNS001C (high)	+ the 17 other zones with a medium confidence level
ZSMNS059A (high)		ZSMNW929A (high)	
ZSMNW042A (high)			
ZSMNW931A (high)			
ZSMNS046A (medium)			

* The width of ZSMEW002A is not constant. It varies between 20 and 200 m in the RVS-model.

3.2.3 Results

For assessing the potential of the site to accommodate the repository, the deformation zones and their distances have been superimposed on maps, see Figure 3-7 to Figure 3-10 for the deep levels of 400, 500, 600 and 700 m /SKB 2006a/.Defermation zones with a low level of confidence are not included as decided by SKB.

The available deposition areas are divided into a number of smaller-areas, which from now on are referred to as DA (deposition areas) + a serial number, e.g. DA25. In all, between 26 and 29 deposition areas have been identified at each depth level. Each DA varies in size with depth depending on the strike and dip of the different deformation zones. The deposition areas of the various depth levels are shown in Figure 3-11 to Figure 3-14. The specified deposition areas refer to the total geometrical areas.

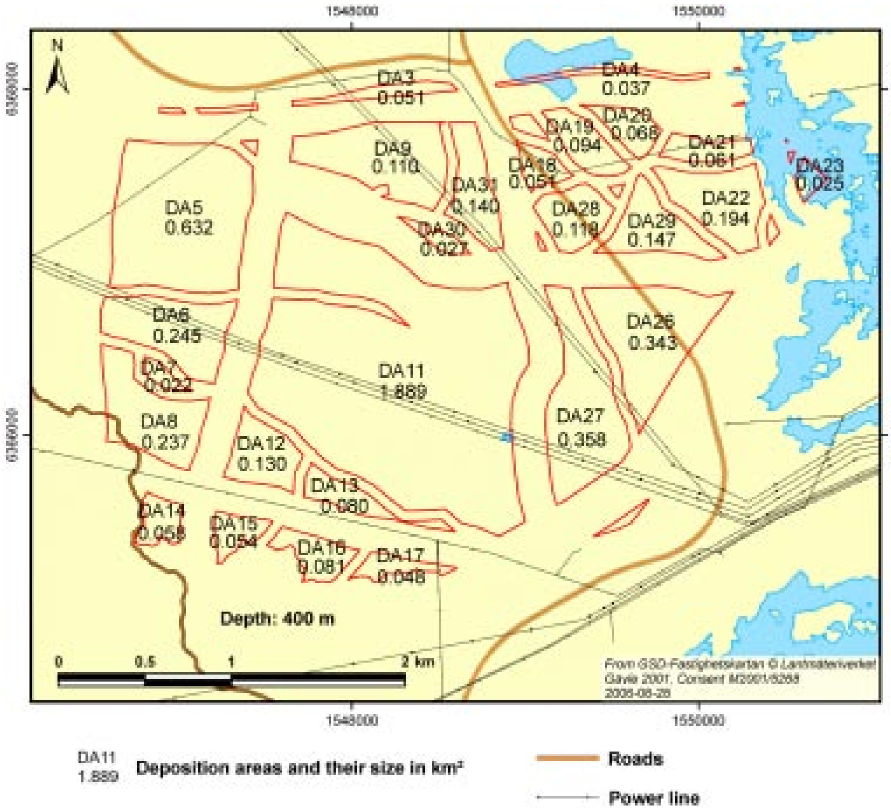


Figure 3-11. *Deposition areas and their size, depth level 400 m.*

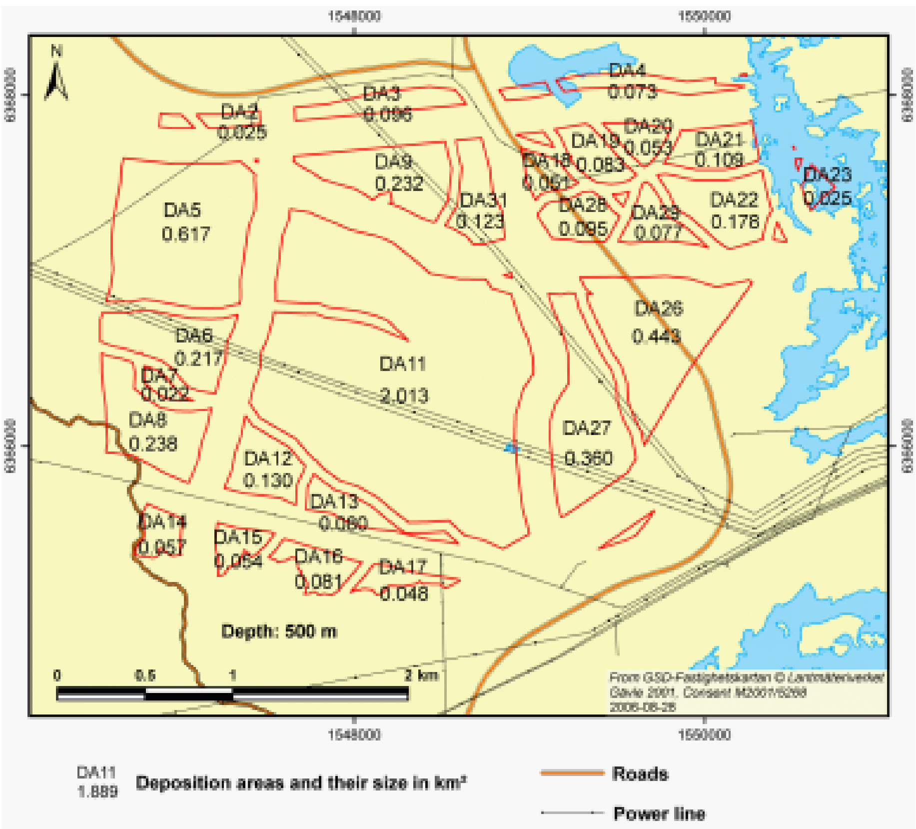


Figure 3-12. *Deposition areas and their size, depth level 500 m.*

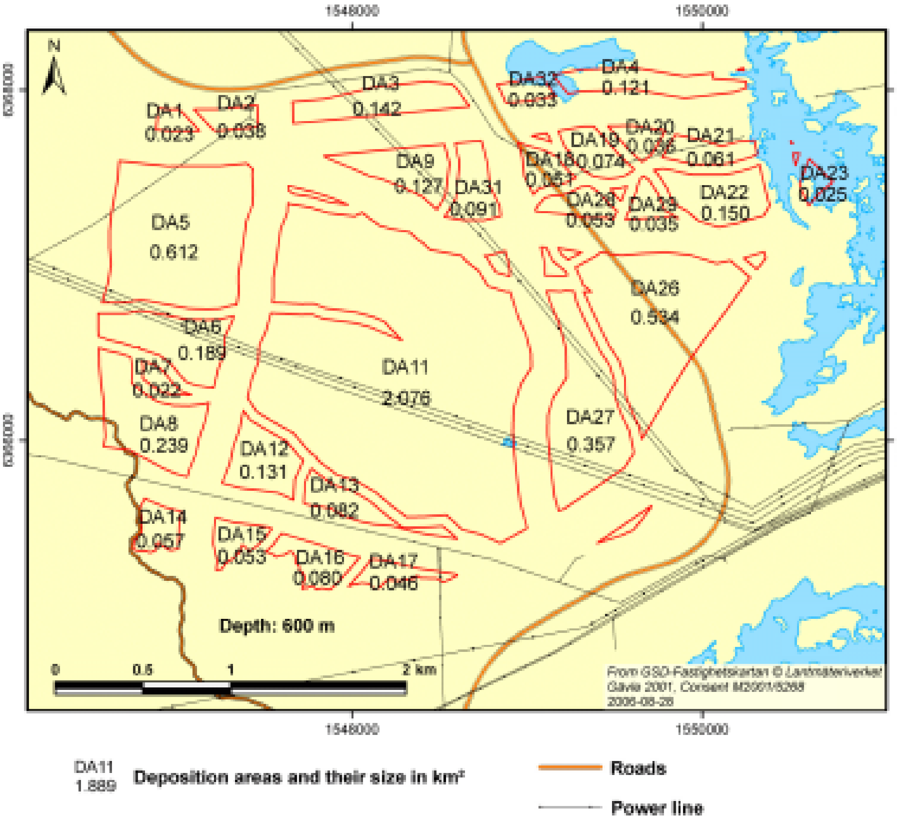


Figure 3-13. *Deposition areas and their size, depth level 600 m.*

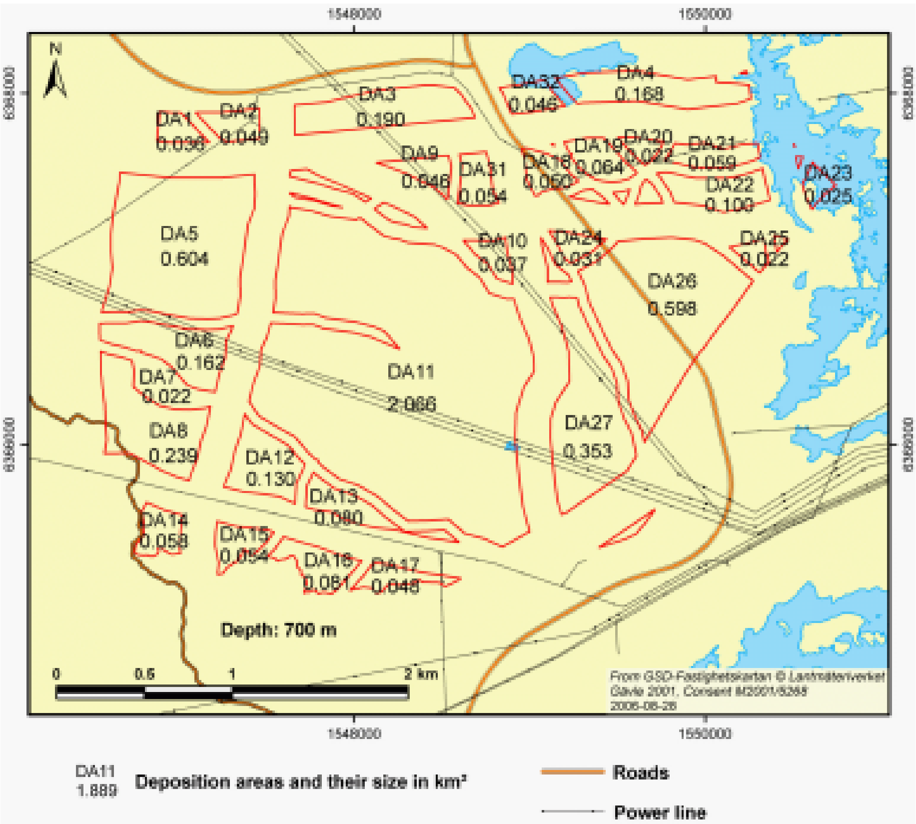


Figure 3-14. *Deposition areas and their size, depth level 700 m.*

Any available areas that are less than 0.2 km^2 in size have been rejected since they fail to meet the demand that they accommodate at least 5 deposition tunnels with a minimum length of 100 m /SKB 2004a/.

The available deposition areas, with a an area > 0.2 km^2, and their sizes have been analysed. Three alternative deposition areas, with a varying number of deposition subareas, have been calculated. Table 3-5 shows the deposition areas and their interfaces/deposition surfaces.

The sizes of the alternative deposition areas vary with depth level. Table 3-6 presents the subareas for each respective depth level (400, 500, 600 and 700 m).

Calculated potential

The proposed total available deposition area A_T varies depending on the limitations and depth. Therefore, the potential for the different alternative deposition areas has been calculated; see Equation 3-1, Section 3.2.1. A value of over 1 means that the area has surplus capacity and can accommodate more canisters than the demand stipulates. Table 3-7 shows the calculated potentials for each alternative and depth level respectively.

The results show that all the alternative deposition areas and depth levels can accommodate the repository with sufficient margins.

Table 3-5. Alternative deposition areas and their limitations.

Deposition area	Limitations
I	All deposition areas within Laxemar
II	In the north ZSME007A and in the south ZSMNW042A, i.e. DA05 to DA08, DA10 to DA13 and DA24 to DA27
III	The four largest deposition areas, i.e. DA05, DA11, DA26 and DA27

Table 3-6. Proposed deposition areas and their size at different depths.

Alternative deposition area	Size, km^2 400 m	500 m	600 m	700 m
I	5.30	5.58	5.54	5.49
II	3.94	4.12	4.24	4.34
III	3.22	3.43	3.58	3.62

Table 3-7. Calculated potential for the different deposition areas.

Alternative deposition area	Potential 400 m	500 m	600 m	700 m
I	2.76	2.91	2.88	2.86
II	2.05	2.15	2.21	2.26
III	1.68	1.79	1.86	1.89

3.2.4 Conclusions and recommendations

The total available deposition areas for the repository within the entire Laxemar area are 5.30, 5.58, 5.54 and 5.49 km^2 corresponding to depths of 400, 500, 600 and 700 m. These numbers have been obtained in consideration to deterministically interpreted deformation zones and their distances (respect, MFE and uncertainty) to the deposition areas. There is therefore only a marginal difference in total available deposition area between the depths. This means that none of the depth levels can be rejected on the grounds of the deformation zones.

Three alternative deposition areas per depth, with varying limitations, have been analysed. The analysis has been performed in order to assess the preliminary potential of the site to accommodate the repository regarding to deformation zones and an assumed loss of deposition holes (25%). The potential of the alternative sites varies between 1.68 and 2.91 depending on alternative and depth level. All alternatives and depth levels provide sufficient margins for accommodating the repository. One conclusion from this is that the repository can be situated in many alternative locations within the Laxemar area.

4 Design of deposition areas

4.1 Design of tunnel geometries

The proposed rock cavern geometries are illustrated in repository description Layout E /SKB 2002/. Figure 4-1 shows the cross-section geometries of main tunnels, transport tunnels and deposition tunnels, according to Layout E. However, the cross-section of the deposition tunnels has been modified by SKB to 4.9 m × 5.4 m, which deviates from Layout E /SKB 2002/ where the dimensions were 5.5 m×5.5 m respectively.

The minimum required distance between deposition tunnel and main tunnel is 20 m, see Figure 4-2, and the distance between the deposition hole periphery and the tunnel face shall be at least 8 m, see Figure 4-3, as per /SKB 2004a/.

Furthermore, the length of deposition tunnels shall be at least 100 m and not more than 300 m.

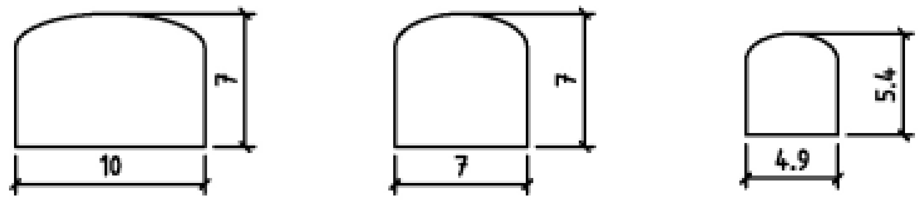


Figure 4-1. *Cross-sectional dimensions of main tunnels, transport tunnels and deposition tunnels.*

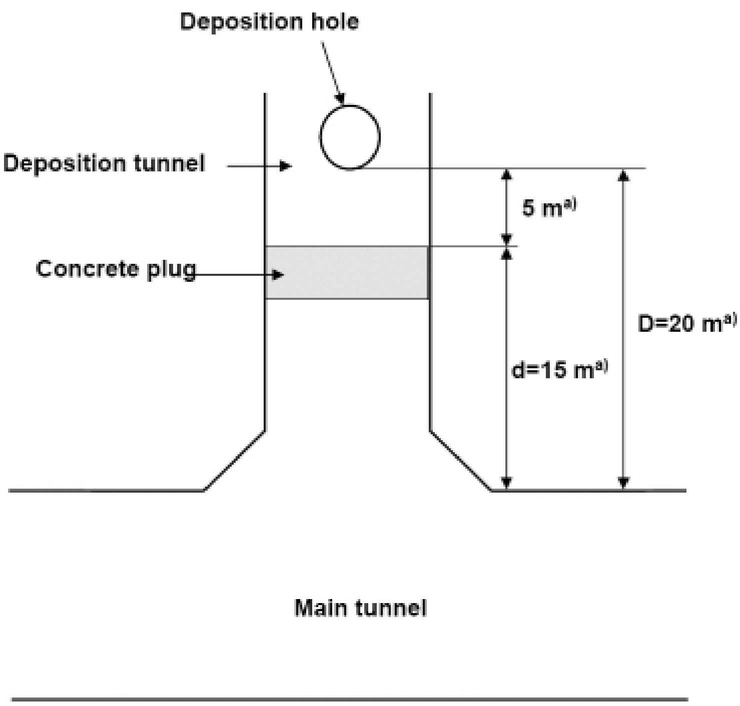


Figure 4-2. *Outline plan of main tunnel, deposition tunnel and deposition hole /SKB 2004a/.*

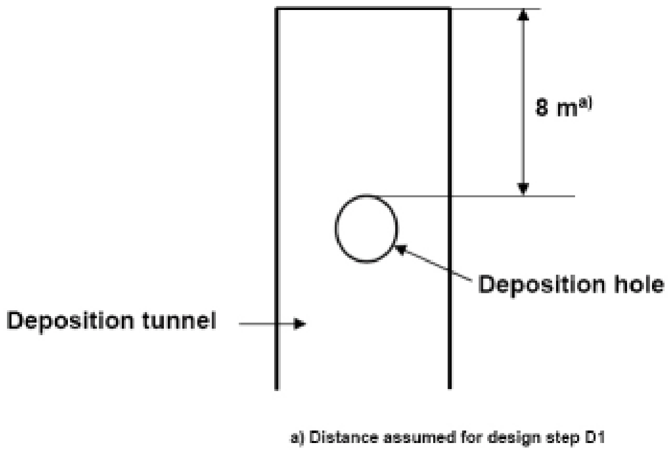


Figure 4-3. Outline plan of deposition tunnel face and nearest deposition hole /SKB 2004a/.

4.2 Distance between deposition tunnels and deposition holes

4.2.1 Input data and assumptions

The determination of the distance between deposition tunnels and between deposition holes shall be made by considering the following:

- Thermal properties of the rock mass.
- Initial temperature at the repository depth.
- The buffer and its thermal properties.

The thermal properties of the rock mass are described in terms of thermal conductivity (W/m, K) and thermal capacity (MJ/m³, K). In the SDM v 1.2 /SKB 2006a/ a presentation of the mean thermal conductivity value, standard deviation for certain domains and different percentiles for the domains in question is given, see Table 4-1.

Furthermore, the SDM v 1.2 also presents data for thermal capacity, see Table 4-2.

While determining the distance (c/c) between the desposition holes, should the thermal capacity of the rock mass has been assigned a value of 2.08 (MJ/m³, K), the initial encapsulation effect 1,700 W/canister and the thermal conductivity of the bentonite 1.0 W/m, K, according to /SKB 2004a/. Furthermore, a limit value temperature at the surface of 80°C is assumed after considering some air gaps and uncertainty in input data.

Data on the initial temperature at certain storage depths have been derived from the site description. Temperatures at different depths are presented in Table 4-3.

Table 4-1. Thermal conductivity (W/m, K) according to /SKB 2006a/.

Rock domain	Mean value	Standard deviation	2.5 percentile	97.5 percentile
A	2.82	0.29	2.32	3.39
BA	2.87	0.29	2.37	3.43
D	2.70	0.17	2.44	3.19
M, alt 1*	2.78	–	–	–
M, alt 2*	2.58	–	2.3	2.80

* M, alt 1 is based on direct thermal laboratory tests for the entire investigation area and the distribution of the thermal conductivities for respective rock types. M, alt 2, is based on the density logging of a borehole within Rock domain M.

Table 4-2. Thermal capacity (MJ/(m^3 K) according to /SKB 2006a/.

Rock domain	Mean value	Standard deviation	2.5 percentile	97.5 percentile
A	2.24	0.13	1.98	2.50
BA	2.23	0.12	1.99	2.48
D	2.29	0.12	2.06	2.52
M	2.25	0.13	1.99	2.47

Table 4-3. Temperature for the different depths, based on data from the SDM v 1.2.

Depth (m)	Temperature (°C), mean value	Temperature span (°C)
400	12.3	11.1–13.4
500	13.9	12.8–15.1
600	15.6	14.5–16.6
700	Approx. 17*	–

* The value is estimated from the diagram of temperature vs depth in SDM v 1.2.

4.2.2 Execution

The distance (c/c) between the deposition tunnels is defined to 40 m according to /SKB 2004a/.

The determination of the distance between the deposition holes shall be derived through application of the graph in Figure 4-4, where the input data of interest is thermal conductivity and the initial temperature.

The graph in Figure 4-4 is based on an initial temperature of 15°C and a limit value temperature of 80°C. The limit value temperature is adjusted linearly in consideration to the initial rock temperature, which is done by shifting the limit value temperature in parallel corresponding to the temperature difference between 15°C and the current initial temperature.

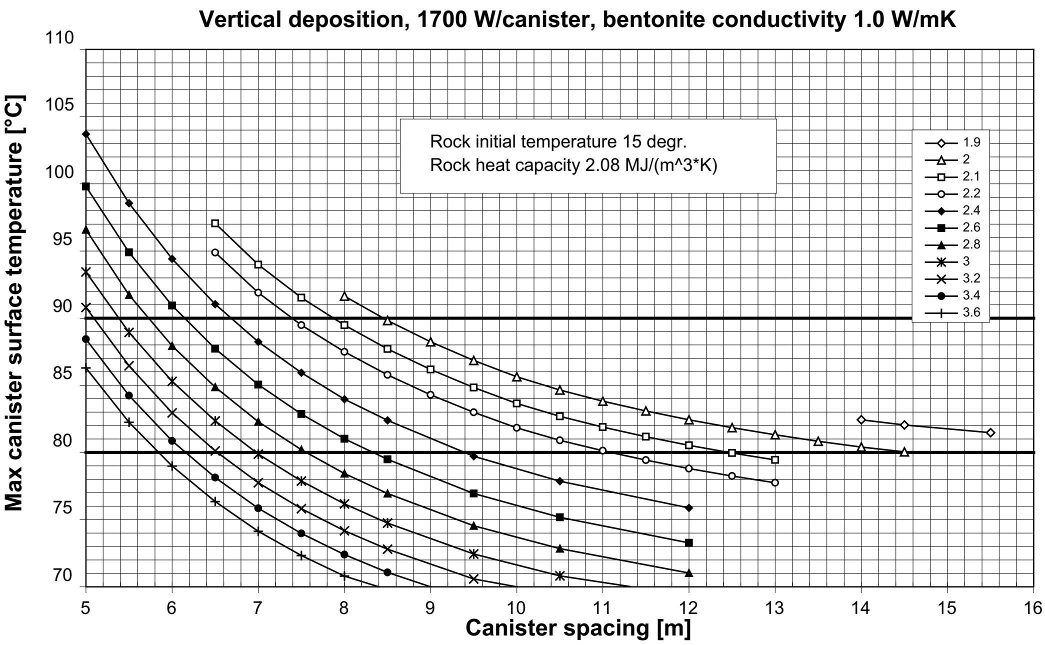


Figure 4-4. *Maximum temperature of canister surface as a function of distance (c/c) between deposition and different thermal conductivities (W/mK) in the rock /rewised after Hökmark and Fält 2003/.*

As a first step, the design distance between the deposition holes is determined by using the mean value of the thermal conductivity and the initial temperature. After that, a sensitivity analysis was made with respect to the uncertainty of the thermal conductivity and the initial temperature.

In the sensitivity analysis, hole distance is presented based on:

- deviation ± 5% from the mean value of the thermal conductivity /SKB 2004a/,
- 2.5 and 97.5 percentiles for thermal conductivity but with a limit value temperature of +83°C instead of +80°C (addition from SKB),
- The initial temperature distribution of 1.0 to 1.5°C for a storage depth of 500 and 600 m (addition from SKB).

The 2.5 and 97.5 percentiles for Rock domain M, alternative 1, is not shown in the SDM v 1.2 /SKB 2006a/ and has therefore not been analysed.

4.2.3 Results

Table 4-4 shows the distances (c/c) between deposition holes for respective rock domains, based on the mean value for thermal conductivity.

The results of the sensitivity analysis are presented in Table 4-5 until Table 4-7. The hole distances based on a deviation of ± 5% from the mean value for thermal conductivity are shown in Table 4-5.

Table 4-6 shows the derived distances between deposition holes for the different rock domains, based on the value for 2.5 and 97.5 percentiles.

The hole distance/spacing for the depths of 500 and 600 m vary between 0.7 and 1.0 m depending on uncertainties in determination of the initial temperature, see Table 4-7.

Table 4-4. Distance between deposition holes for the different rock domains, based on the mean value for thermal conductivity.

Depth (m)	Hole distance, c/c, (m)				
	A	BA	D	M, alt. 1	M, alt. 2
400	6.9	6.7	7.2	7.0	7.6
500	7.2	7.0	7.6	7.4	8.1
600	7.6	7.4	8.1	7.8	8.6
700	8.0	7.8	8.5	8.2	9.2

Table 4-5. Determination of distance between deposition holes, per domain, based on a deviation of ± 5% from the mean value for thermal conductivity.

Depth (m)	Hole distance, c/c, (m)									
	A		BA		D		M, alt. 1		M, alt. 2	
	– 5%	+ 5%	– 5%	+ 5%	– 5%	+ 5%	– 5%	+ 5%	– 5%	+ 5%
400	7.2	6.4	7.1	6.4	7.7	6.7	7.4	6.5	8.0	7.1
500	7.8	6.8	7.5	6.8	8.2	7.1	7.8	7.0	8.7	7.6
600	8.1	7.1	8.0	7.1	8.7	7.5	8.2	7.4	9.3	8.1
700	8.6	7.5	8.4	7.5	9.3	7.9	8.8	7.8	9.9	8.6

Table 4-6. Determination of distance between deposition holes for the various rock domains, based on the 2.5 and 97.5 percentiles, respectively, for thermal conductivity.

Depth (m)	Hole distance, c/c, (m) A		BA		D		M, alt. 2	
Percentile	2.5	97.5	2.5	97.5	2.5	97.5	2.5	97.5
400	7.9	5.4	7.7	5.3	7.3	5.6	8.0	6.3
500	8.5	5.6	8.3	5.5	7.7	5.9	8.5	6.7
600	9.1	5.8	8.8	5.7	8.2	6.1	9.1	7.0
700	9.7	6.0	9.4	5.9	8.7	6.4	9.7	7.3

Table 4-7. For the depths of 500 m and 600 and an initial temperature uncertainty of 1.5°C, the following variation in hole distance/spacing is obtained for each rock domain, based on the mean value for thermal conductivity.

Rock domain	Depth 500 m	Depth 600 m
A	6.9–7.6 m	7.2–8.0 m
BA	6.7–7.4 m	7.0–7.8 m
D	7.2–8.1 m	7.6–8.5 m
M, alt. 1	7.0–7.8 m	7.4–8.2 m
M, alt. 2	7.7–8.6 m	8.1–9.1 m

4.2.4 Discussion

The distance between the deposition holes varies according to rock domain and repository depth. On the basis of the mean value for thermal conductivity, a minimum distance of 6.7 m is obtained for Rock domain BA and 400 m repository depth, and a max distance of 9.2 m, for Rock domain M based on its thermal conductivity as per alternative 2 and repository depth 700 m.

The variation in thermal conductivity within the rock domains is significant, which means that there is an uncertainty in the determination of hole distance. During the sensitivity analysis of the hole distance, the uncertainty has been expressed through the following variations in input data:

- Deviation ± 5% from the mean value for thermal conductivity.
- Thermal conductivities of 2.5 and 97.5 percentiles.
- An uncertainty of 1.5°C in the initial temperature,

For Rock domains A and BA, the maximum hole distance based on 2.5 percentile is obtained. For Rock domains D and M, i.e. those with a somewhat lower thermal conductivity, the maximum hole distance is obtained based on the deviation from the mean value for thermal conductivity with –5%. None of the sensitivity criterion always gives therefore a maximum distance. This is because different uncertainty temperatures (17 or 20°C) are used as input data in the nomogram, see Figure 4-4, for determination of the hole distance in combination with the non-linear conditions between thermal conductivity and hole distance.

For the purpose of continuity in the design process, it is recommended that the hole distance which is based on the mean value of the thermal conductivity for respective rock domains to be used in the layout work. The consideration of uncertainty in the hole distance for the layout of the repository facility will be further analysed in Chapter 10.

4.3 Orientation of deposition tunnels

4.3.1 Input data and assumptions

The purpose of this section has been the recommendation of a suitable orientation for deposition tunnels with respect to:

1. Calculated quantity of assumed water seepage into deposition tunnels and deposition holes.
2. Risk of spalling in deposition tunnels.
3. Calculated volume of potentially unstable wedges in deposition tunnels and deposition holes .

Analysis of Points 1 and 3 has been carried out within the present project. Analysis of Point 2 has been carried out in a separate SKB study /Martin 2005/.

Description of input data execution and presentation of results are conducted separately as part of respective sub-analyses, i.e. Points 1 to 3.

Point 1: Tunnel orientation with respect to seepage into deposition tunnels and deposition holes

In order to be able to decide an optimal orientation for the deposition tunnels with respect to the seepage, both an analytical and a numerical (DFN) model have been used. The analyses have been carried out for Rock Domains A and M at a depth of 500 m. The fact that only one depth level has been studied was decided following discussions with SKB. The DFN-model is not depth dependent.

The analysis is calculated for a 300 m long deposition tunnel at a depth of 500 m. The calculation is made for stationary conditions and the skin-factor is set at zero.

The conductivity data is directionally dependent in accordance with HydroDFN in the SDM v 1.2 /SKB 2006a/ with a maximum transmissivity for fractures in an E-W direction. Data for hydraulic conductivity has been derived from the semi-correlated transmissivity model with a block size of 100 m at a depth below 300 m.

The numerical DFN-model is based on available data from HydroDFN in the SDM v 1.2. The model consists of 5 fracture sets that are described with respect to orientation, length, intensity (P_{32}) and transmissivity. There are also presented in HydroDFN model three different interpretations of fracture transmissivity from site data and a fracture length model.

Point 2: Stress-induced spalling in the deposition tunnels

The separate study of /Martin 2005/, shows a method in which the strength of the rock mass against stress-induced spalling can be set at 0.57 of the mean value for uniaxial compressor tests. The study also shows a probability-based attempt to evaluate the risk of stress-induced spalling in a facility. The analysis for Laxemar has been made for two stress domains, I and II. The domain that is of interest for continued design within Laxemar, is Domain I. Input data in the analyses is based on site data from the rock mechanical laboratory testing /Martin 2005/.

Point 3: Tunnel orientation with respect to volume of potentially unstable wedges in deposition tunnels and deposition holes

The in situ stress field that has been measured in the area /SKB 2006a/ has been used for the analysis. This gives a restraint of the wedges and increases the safety against wedge failure. The analyses have been conducted with initial in situ stresses that can be anticipated at depths of 500 m and 600 m.

It is assumed in the analysis that all fracture planes are continuous and flat, and have the same strength. This means rather conservative conditions. For the analysis purpose, the Mohr-Coulombs fracture criterion has been used with a residual friction angle of 32° and zero in cohesion.

4.3.2 Execution

Point 1: Tunnel orientation with respect to seepage into deposition tunnels and deposition holes

Analytical method:

The analytical calculation of seepage, q_s, has been carried out in accordance with Equation 4-1 in Figure 4-5.

An isotrope, equivalent conductivity, K_b, can be calculated on the basis of the vertical, K_z, and horizontal conductivities that are perpendicular to the tunnel (i.e. for those fractures that cut through the tunnel at an angle of 90°), as per Equation 4-2:

$$K_b = \sqrt{K_{\alpha+\pi/2} \cdot K_z} \qquad \text{Equation 4-2}$$

Where α is the angle between the tunnel and the maximum horizontal conductivity, K_{hmax}, and $K_{a+\pi/2}$ is the horizontal conductivity in direction a+π 2 from K_{hmax}.

Statistical analyses of seepage have been performed by using the Monte Carlo simulation, where 1,000 realisations of conductivity data have been made. The analyses were carried out by using Excel with the addition of NtRand Version 2.01 (Numerical Technologies@).

The tunnel orientation varies in relation to the largest conductivity direction within the relative angles 0° (parallel), 30°, 60°, 90°, 120° and 150°. The analyses have also been performed and presented in relation to the largest horizontal main stress direction (N132°).

Numerical method:

The analyses are carried out by using NAPSAC Version 9.0 software, with the following preconditions:

- Model size: 400 m × 300 m × 500 m (height × width × length).
- The tunnel is 300 m long and the cross section geometry has been slightly changed from the original horseshoe-shaped tunnel cross-section. The horseshoe-shaped geometry has been recalculated to an equivalent radius of 2.8 m for a circular opening.
- The tunnel is modelled without other openings in the vicinity.
- The boundary conditions are constant, i.e. application of 500 m hydrostatic pressure on all enclosed surfaces and atmospheric pressure in the inner part of the tunnel.
- Stationary flow model.

d = distance from centre of tunnel to original groundwater table (m)

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

r_w = representative tunnel radius (m)

ξ = natural skin factor (dimensionless)

$$q_s = \frac{2 \cdot \pi \cdot K_b \cdot d}{\ln\left[\frac{2 \cdot d}{r_w}\right] + \xi} \qquad \textit{Equation 4-1}$$

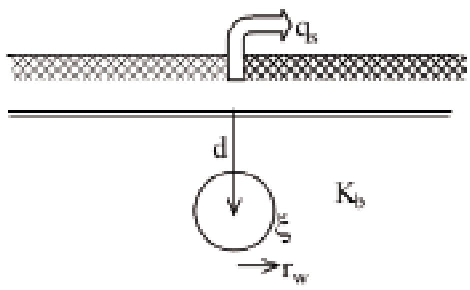


***Figure 4-5.** Relationship for the analytical calculation of inflow to deposition tunnels.*

For the calculation of P_{33}, total fracture volume per volume unit, a correlated fracture aperture model has been used /Hartley et al. 2005/:

$$e = a' T^{b'} \qquad \text{Equation 4-3}$$

where e is the fracture aperture, T is transmissivity the constants, a' = 0.46 and b' = 0.5. P_{33} is calculated for the entire length of the tunnel in an area of 20 m×30 m×500 m around the tunnel.

The direction/orientation of the deposition tunnel has been varied between 0°, 30°, 60°, 90°, 120° and 150° in relation to the largest horizontal in situ main stress at, N132° according to /SKB 2006a/.

Fifty Monte Carlo simulations have been carried out for each tunnel orientation in order to study the variation and uncertainty in the input data.

For modelling purposes, the model area has been divided into an inner and an outer region. In the inner region (H 30 m×B 20 m×L 500 m), i.e. surrounding the tunnel, all joints that are described in HydroDFN are generated. An example is shown in Figure 4-6.

In the outer region, only joints with a length of more than 10 m are generated. The consequence of this, is that the joint intensity, for joints larger than 10 m, has to be adjusted in the analysis in order to match P_{32} in the HydroDFN model.

Point 2: Stress-induced spalling in the deposition tunnels

Only the results of the separate study /Martin 2005/ are presented in this report.

Point 3: Tunnel orientation with respect to the volume of potentially unstable wedges in deposition tunnels and deposition holes

The analysis has been carried out by using the program *Unwedge* from Rockscience Ltd. The program is a 3D stability program for calculating the stability in rock wedges in underground facilities. It is based on the fact that there are always three distinct fractures/fracture planes that come from wedges, and can appear anywhere while excavating a tunnel/cavern

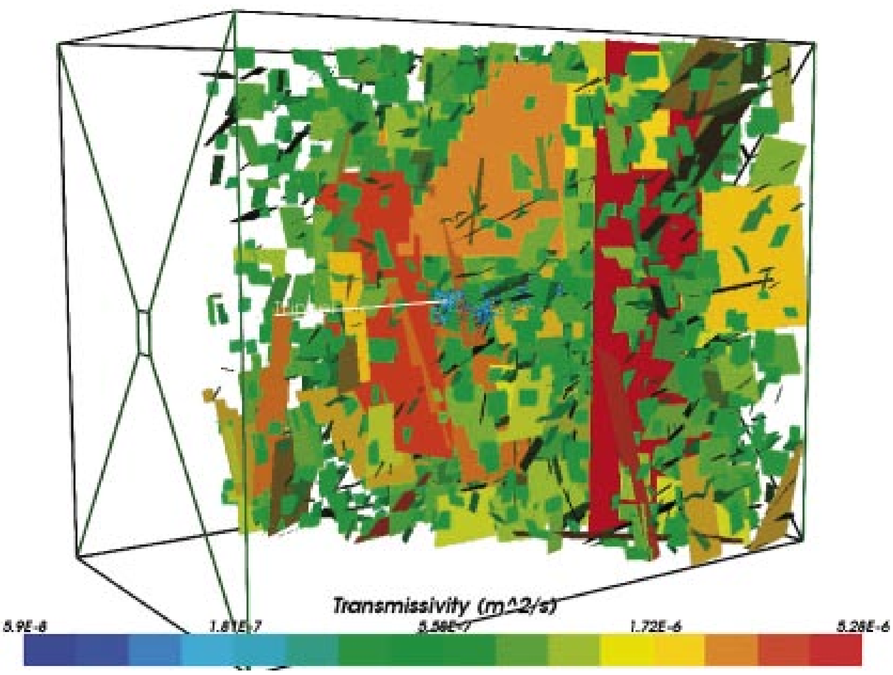


Figure 4-6. *Example of a section through a realisation from the DFN model. The blue joints in the centre are short fractures.*

in the rock mass. By using block theories, the program can determine the largest wedges that can occur for the three fracture planes in a tunnel roof/wall/floor. This means that the analyses are rather on the conservative side. *Unwedge* defines automaticly the tunnel length that is needed in order for all unstable tetraedric wedges to be formed by three fractures/fracture planes.

For each wedge analysis, five fracture planes have been specified: four sub-vertical and one sub-horizontal. Since it was not possible to analyse all stochastically-generated fractures that intersect the tunnel and deposition holes, it was decided to analyse a number of random stochastic fractures. By using the program NAPSAC, 50 random fractures were selected within each fracture set. These were taken together and analysed in the order they were generated.

Calculations have been made for all tetraedric wedges that can be combined within three of the five fracture planes. In the case in question, the number of conceivable combinations is 10. Since the analysis covers all wedges, unstable wedges have been defined as those with a safety factor of ≤ 1.0. Wedges that may occur at the the tunnel end, have not been included in the analysis. The term "total wedge volume" means that it is the cumulative volume of the largest wedges with the safety factor ≤ 1.0 for all conceivable combinations for the 50 random fractures within each fracture set.

The results of the calculations in the form of safety factors and wedge volumes have than been processed in Excel.

4.3.3 Results

A comparison of the results from the analytical calculations and the numerical (DFN) modelling show that the DFN model gives 3 to 4 times more seepage into the deposition tunnel than the analytical. The DFN model gave a little anisotropy in seepage for Rock domain A, with two weak min. values at orientations N72° and N132°, whereas the results for Rock domain M indicate a weak min value only at N132º. The analytical calculation shows only one minimum value in an E-W direction, see Figure 4-7 and Figure 4-8.

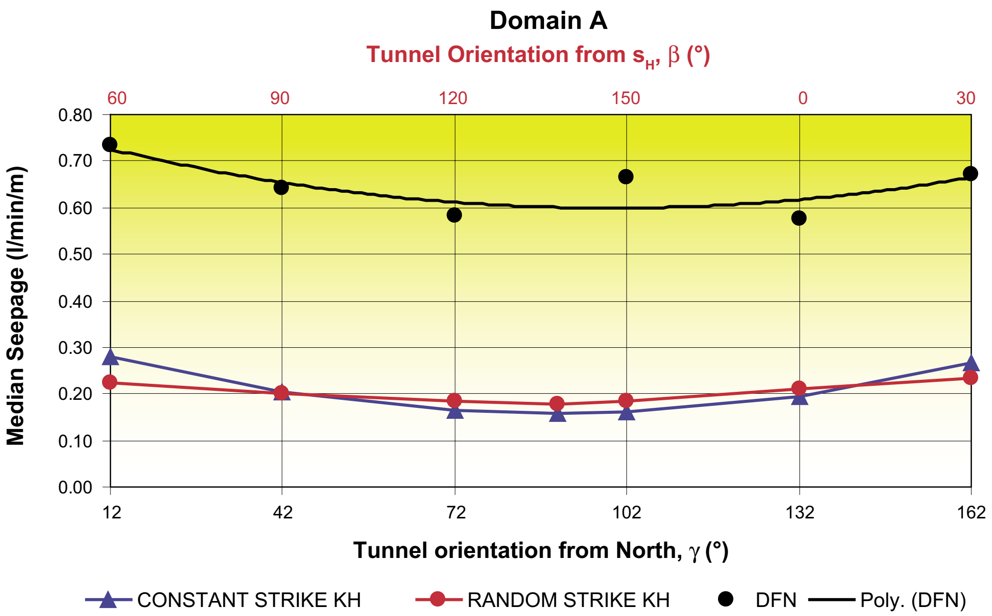


***Figure 4-7.** A comparison of median values from the analytical calculations (KH) and DFN modelling for Rock domain A.*

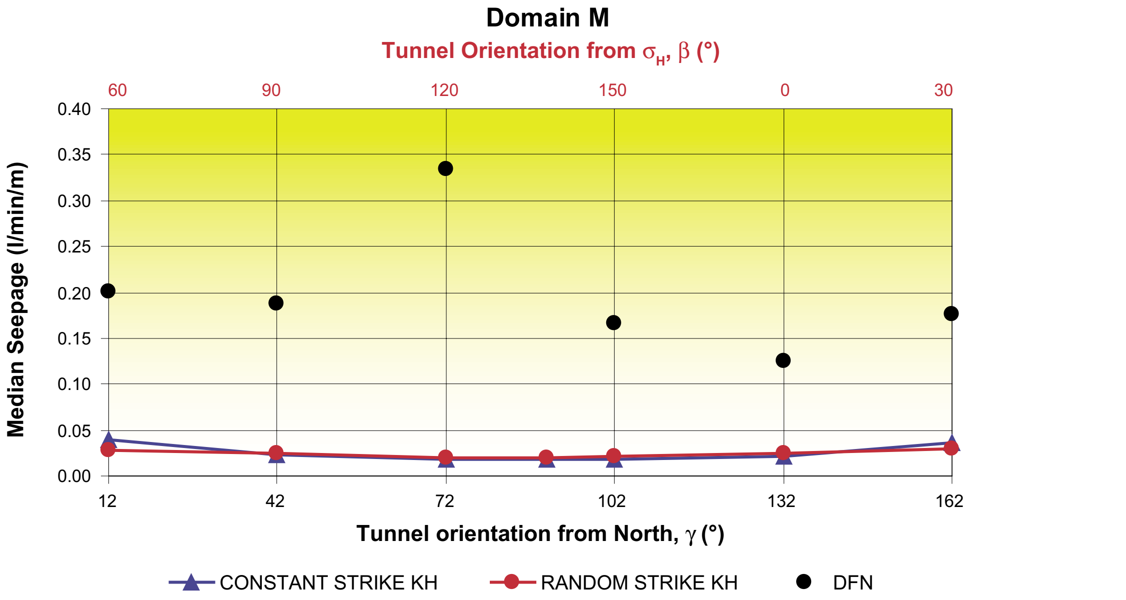


Figure 4-8. *A comparison of median values from the analytical calculations (KH) and DFN modelling for Rock domain M.*

The tunnel direction for the least amount of seepage, calculated from the analytical calculations, is derived from Hydro-DFN and is affected by the scaling method that is used in order to calculate block conductivity. The results of the DFN modelling are representative of fracture connections and flow anisotropy in the rock mass surrounding the tunnel. In view of this, the results of the DFN modelling are more relevant than the analytical calculations. The most optimal direction for the deposition tunnel is thus N132°, i.e. parallel to the largest horizontal main stress σ_H.

The distribution of seepage between deposition tunnels and deposition holes shows that the anisotropy in relation to the tunnel orientation is greatest in the deposition tunnel while seepage into the deposition holes varies to a lesser extent, see Figure 4-9 and Figure 4-10 for the 95% probability interval.

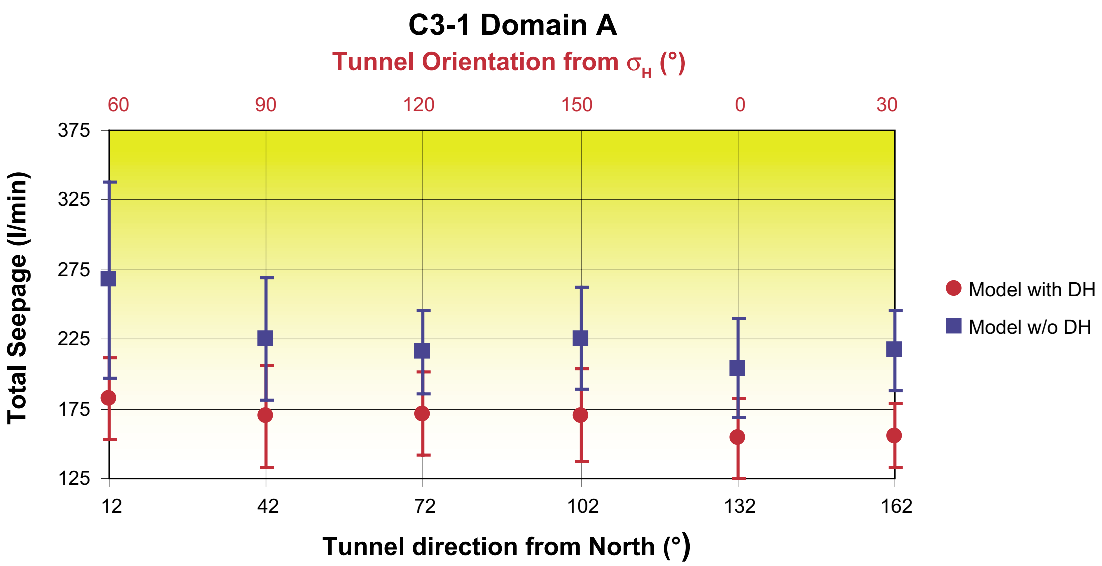


Figure 4-9. *Comparison of results from modelling with deposition holes and modelling without deposition holes, for Rock domain A and the 95% probability interval.*

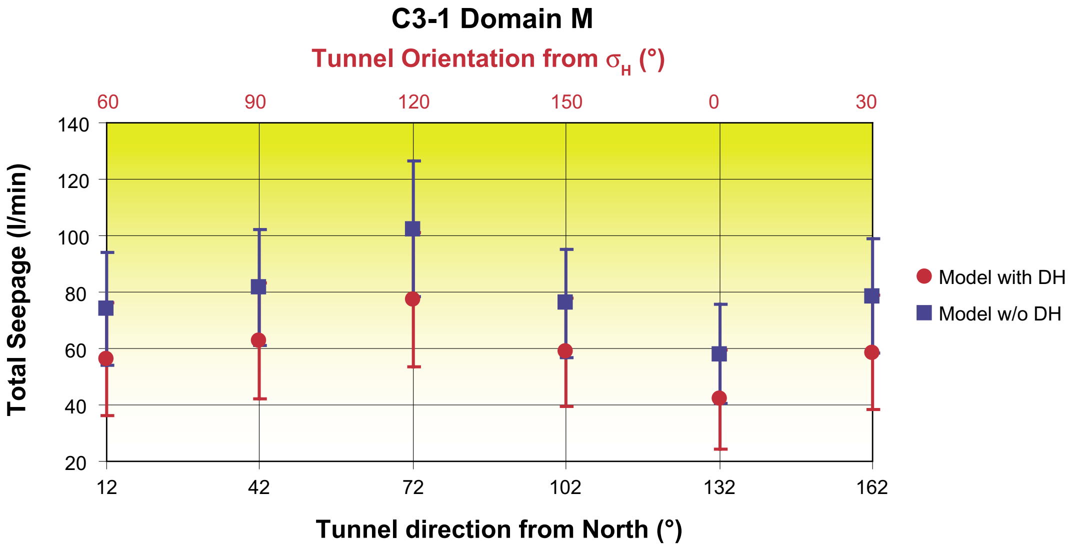


Figure 4-10. *Comparison of results from modelling with deposition holes and modelling without deposition holes, for Rock domain M and the 95% probability interval.*

For Rock domain A, the results from DFN indicate a minor difference in seepage for different tunnel orientations.

For Rock domain M, the results from DFN indicate the largest seepage for orientation N72^o and the smallest seepage for orientation N132^o.

Point 2: Stress-induced spalling in the deposition tunnels

The final results with safety factors and probability for stress-induced spalling are presented in Figure 4-11 and Figure 4-12.

The results presented above, show that the probability of stress-induced spalling increases with increased depth. The analysis shows that at a depth of 500 m, the probability of stress-induced spalling is zero, irrespective of the angle (0, 45 or 90^o) to the largest main stress. At a depth of 600 m and a tunnel orientation perpendicular to the largest main stress, the probability of a stress-induced spalling is 40%. However, spalling gives marginal volumes of loose rock.

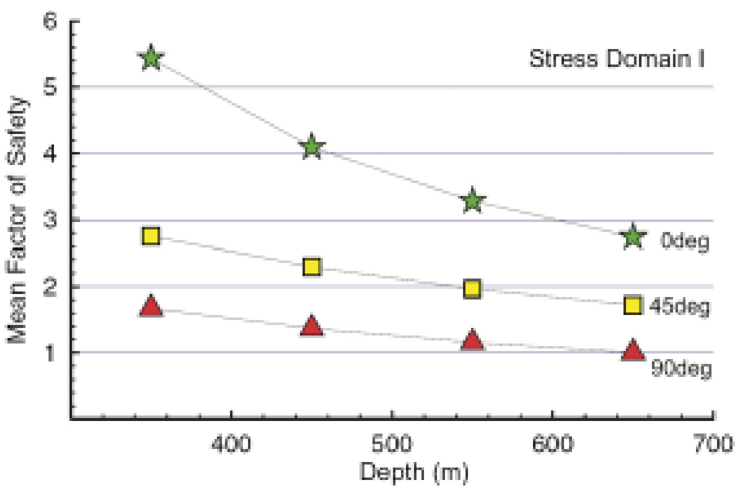


Figure 4-11. *Analysed safety factor (mean value) for stress-induced spalling in deposition tunnels with angles of 0^o, 45^o and 90^o to the largest main stress /Martin 2005/.*

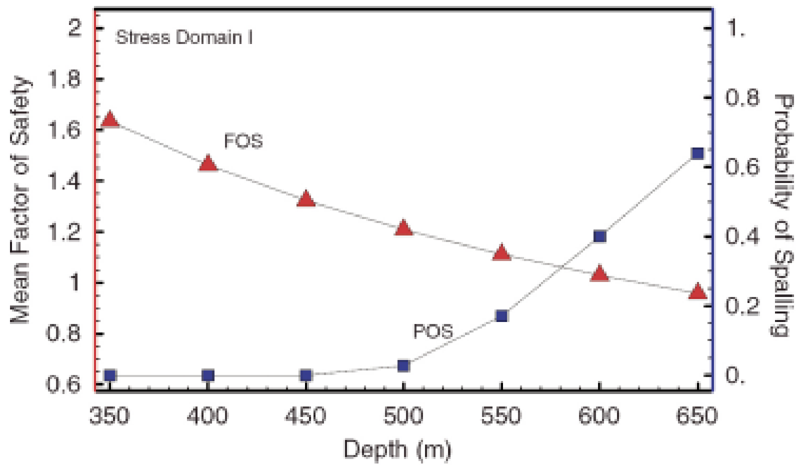


Figure 4-12. *Analysed safety factor (FOS) and probability (POS) for stress-induced spalling in the deposition tunnel with an angle of 90° to the largest main stress /Martin 2005/.*

Point 3: Tunnel orientation with respect to volume of potentially unstable wedges in deposition tunnels and deposition holes

The results of the calculations of unstable wedges in deposition tunnels and deposition holes show almost the same optimal orientation. The most optimal orientation for the deposition tunnels is approximately 60° to the largest horizontal main stress, see Figure 4-13 and Figure 4-14 for depths 500 and 600 m respectively. The most favourable orientation for the deposition tunnels is therefore approximately N12°.

4.3.4 Discussion

Figure 4-15 is a compilation that shows seepage into the deposition tunnel and deposition holes in the direction of the tunnel orientation for Rock domains A and M. In addition, the total wedge volume for the deposition holes towards the tunnel orientation at depths of 500 m and 600 m are shown.

The analyses in respect to water seepage into the deposition tunnel and deposition holes, indicate a weak direction dependency for Rock domain M and an even weaker direction dependency for Rock domain A. The most optimal orientation of the deposition holes in Rock domain M is N132° whereas the analysis for Rock domain A does not give any clear optimal orientation.

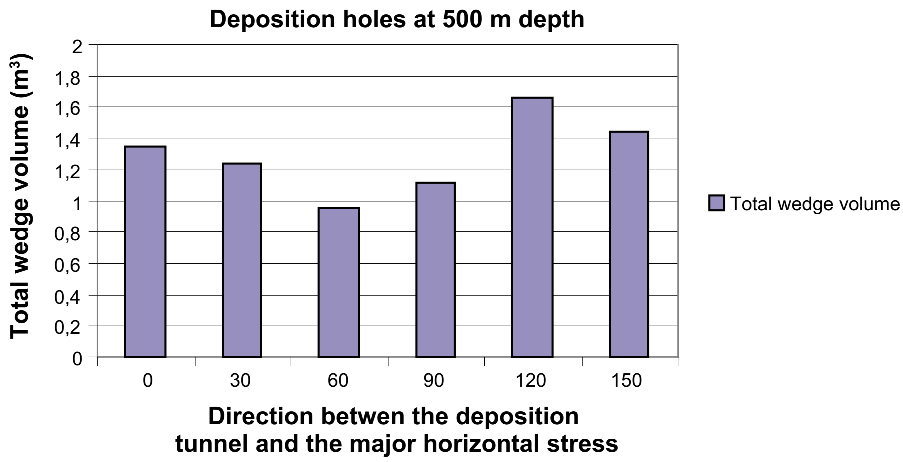


Figure 4-13. *The graph shows the total wedge volume for unstable wedges. The total wedge volume is plotted towards the direction of the tunnel at a depth of 500 m.*

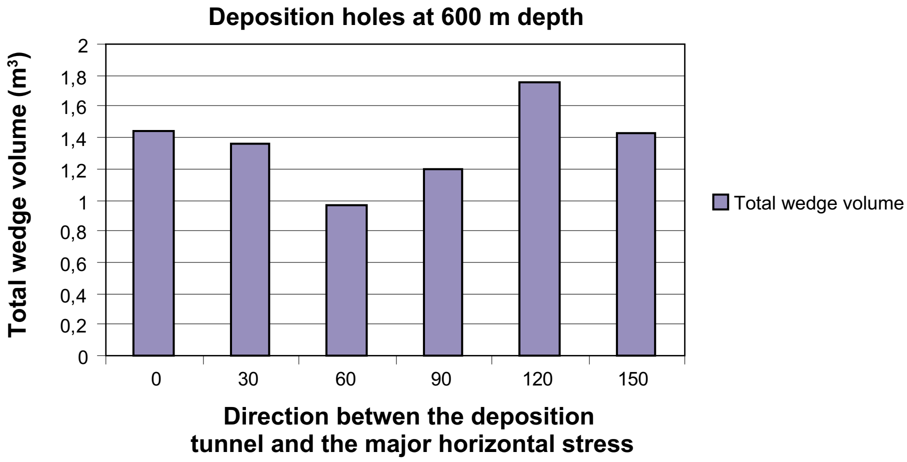


Figure 4-14. *The graph shows the total wedge volume for unstable wedges. The total wedge volume is plotted towards the direction of the tunnel at a depth of 600 m.*

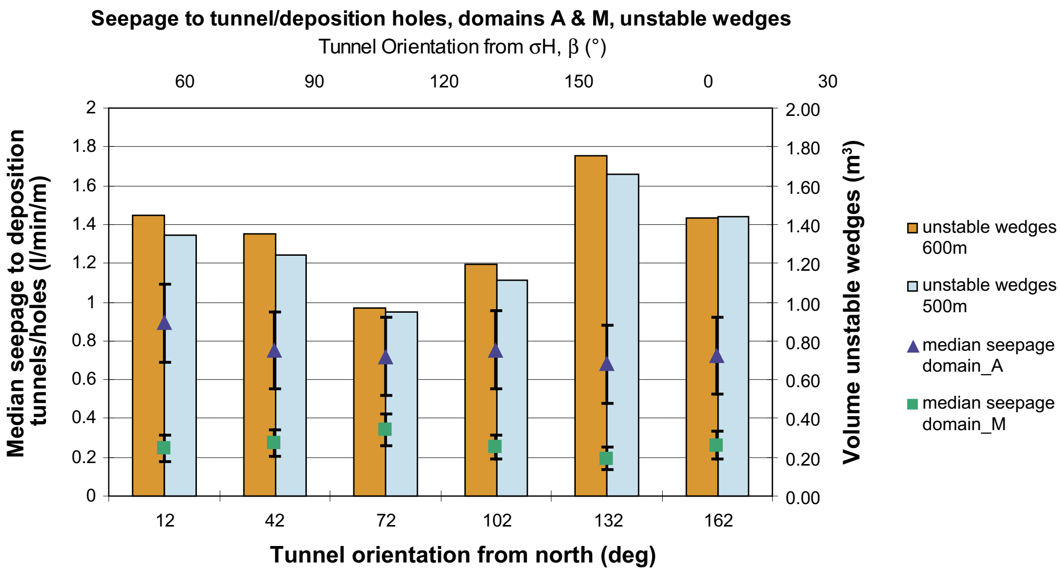


Figure 4-15. *Graph showing the mean value of seepage into the deposition tunnel and deposition holes towards the tunnel orientation for Rock domains A and M (500 m depth). In addition, the total wedge volume in the deposition holes towards the tunnel orientation at depths of 500 m and 600 m are shown.*

The separate study /Martin 2005/ shows that at a depth of 500 m, the likelihood of stress-induced spalling is equivalent to zero, regardless of the tunnel angle to the main stress. If, on the other hand, the depth is increased, there is a greater probability of stress-induced spalling with increasing depth. Furthermore, the most unfavourable tunnel direction, with increasing depth, is perpendicular to the main stress.

Wedge calculations show that the optimal orientation for the deposition tunnels is N12° regardless of depth. The experience from Äspö HRL show that only a few marginal volumes of loose rock occur in one of 17 drilled deposition holes.

As a recommendation, it can be stated that a certain degree of freedom exists in the orientation of the deposition tunnels, but a state of preparedness should be accounted for measures that may be necessary in order to prevent wedge failure and seepage. Based on the analyses in this section, if there is a need to choose one orientation only, this is N132°, i.e. parallel to the main horizontal stress.

4.4 Loss of deposition holes

4.4.1 Input data and assumptions

The purpose of this section is to determine the loss of deposition holes with respect to:

1. intersection by stochastically determined fractures (elongated fractures),
2. quantity of water leaking into the deposition holes, with a specified criterion,
3. wedge outfall in deposition holes, with a specified criterion,
4. risk of stress-induced spalling in deposition holes.

The analysis of Points 2 and 3 has been performed within this project and the result is a direct precondition for corresponding points in Section 4.3. The analysis of Point 1 has been made by SKB and for the analysis of Point 4, SKB has appointed a separate study /Martin 2005/.

The coming description of input data, execution and presentation of results is made separately under respective sub-analyses, i.e. Points 1 to 4.

Point 1: Loss of deposition holes with respect to intersection of elongated fractures

Elongated fractures, in the SKB study, is understood to refer to fractures with a radius of 50 m < r < 600 m, which is a deviation from design premises of r > 100 m /SKB 2004a/.

Point 2: Loss of deposition holes with respect to seepage

Loss of deposition holes with respect to water seepage in, at criterion q > 10.0 l/min, is based on the DFN modelling in Section 4.3. There is probably a link between Point 1 (elongated fractures) and this point, but it has not been possible to analyse it in this work.

The loss of deposition holes, P2, is expressed as:

$P_2 = N_F/N_{TOT}$ Equation 4-4

where N_F is the number of deposition holes when seepage exceeds a certain criterion and N_{TOT} is the total number of deposition holes. For both the depths 500 m and 600 m, it is assumed that N_{TOT} is 38 pcs, i.e. no consideration has been given in the analysis to increased distance/spacing between deposition holes as a result if increased initial temperature with depth.

The analysis has been carried out by using the NAPSAC Version 9.0 program, and is based on the same assumptions and input data as in Section 4.3.1 "*Point 1",* with respect to model size, geometries, conditions and available data from HydroDFN with respect to fracture properties.

Point 3: Loss of deposition holes with respect to volume of potentially unstable wedges in deposition holes

The definition for the loss is given as a wedge outfall greater than 0.15 m³. The loss of deposition holes is expressed as:

$P_3 = N_B/(N_{TOT}-N_F)$ Equation 4-5

where N_B is the number of deposition holes when the volume of wedges exceeds the criterion (> 0.15 m³), N_F is the number of deposition holes when seepage exceeds the criterion (q > 10 l/min) and N_{TOT} is the total number of deposition holes.

The analysis of unstable wedges in deposition holes is also based on the corresponding, assumptions in "*Point 3*" and input data as in Section 4.3.1.

Point 4: Loss of deposition holes with respect to stress-induced spalling in deposition holes

The analysis of stress-induced spalling in deposition holes is also based on the corresponding assumptions, in "*Point 2*", and input data as in Section 4.3.1.

4.4.2 Execution

Point 1: Loss of deposition holes with respect to intersection of elongated fractures

The results in the SKB analysis are presented in this report and are calculated according to /Hedin 2005/.

Point 2: Loss of deposition holes with respect to seepage into the deposition holes

The loss of deposition holes as a consequence of water seepage has been calculated for a main criterion of q > 10.0 l/min. In order to highlight the sensitivity of the seepage criterion, the loss has also been analysed for q > 1.0 and q > 0.1 l/min.

The analysis follows the description in Section 4.3.2 and "*Point 1*", i.e. the same varying tunnel orientation in relation to the main stress, number of Monte Carlo simulations and a subdivision of an inner and an outer fracture region in the model area.

In addition to the analyses, the results and experience from seepage to drilled deposition holes in Äspö HRL have been studied.

Point 3: Loss of deposition holes with respect to the volume of potentially unstable wedges in deposition holes

The analysis has been carried out by using the program *Unwedge,* see description in Section 4.3.2 and "*Point 3*".

When analysing wedges in the deposition holes, the heights are corrected when necessary so that they do not exceed 8 m. An example of a calculation with *Unwedge* is shown in Figure 4-16.

In addition to the loss criterion 0.15 m³, a sensitivity analysis has also been conducted with the volume criterion V > 0.1 m³ and V > 0.2 m³. The purpose of this is to show how great an impact the size of the wedges has.

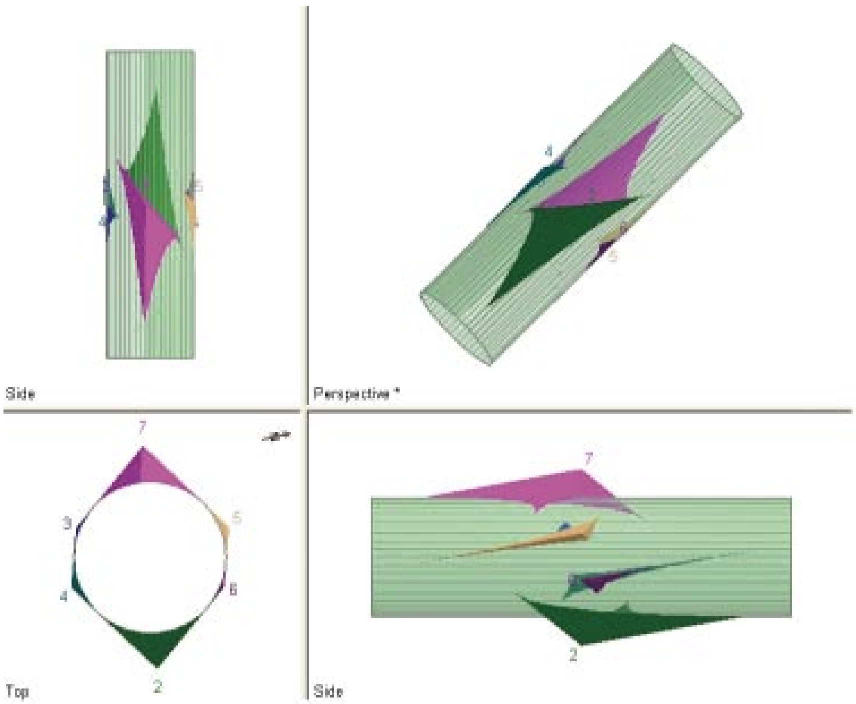


Figure 4-16. *Example of a calculation with UNWEDGE.*

In the DFN modelling, a 300 m-long deposition tunnel with 38 deposition holes has been analysed, see example in Figure 4-17.

The stocastically-generated fractures that cross each deposition hole were compiled for each realisation. This was followed by the actual wedge analysis using *Unwedge*. It was assumed that the fractures in the deposition holes can form tetraedric wedges over a maximum length of 8 m. The number of compiled fractures in each deposition hole varied from 0 to 15 pcs. When more than 3 fractures are found in a deposition hole, every conceivable wedge combination with three fracture planes has been analysed. In the case of 5 fracture planes, this will give 10 combinations, with 6 fracture planes 20 combinations, with 8 fracture planes 56 combinations, etc.

By means of checks, we have made sure that deposition holes which have been lost as a result of excessive inflowing water, as per *Point 2*, have not been further analysed for loss as a result of unstable wedges.

Comparisons have also been made by considering the number of displaced wedges during the drilling of 17 deposition holes in Äspö HRL.

Point 4: Loss of deposition holes with respect to stress-induced spalling in deposition holes

Only the results from the separate study /Martin 2005/ were presented in this report.

4.4.3 Results

Point 1: Loss of deposition holes with respect to intersection of elongated fractures

SKB's study has stated that loss as a consequence of elongated fractures that cross deposition holes shall be set at 16%.

Point 2: Loss of deposition holes with respect to seepage

The results of the calculations are presented for each selected tunnel orientation in the form of percentiles (50, 90, 95 and 99 percentiles), median value, i.e. 50 percentiles, and the mean value and its 95% probability interval. Figure 4-18 and Figure 4-19 show the results for Rock domain A and M, respectively, for the criterion q > 10 l/min.

For Rock domain A, the analysis shows a loss of 1.7–2.3 pcs deposition holes per deposition tunnel, which means a loss proportion of 4.5–6.1%. The variation between the tunnel orientations is so small that it is not statistically secured. The total seepage into the deposition holes has been calculated as a mean value at between 0.25 and 0.31 l/min/m, i.e. for an 8 m long hole, 2.0–2.5 l/min.

Figure 4-17. *An example of a number of joints that intersect the 38 deposition holes from an NAPSAC generation.*

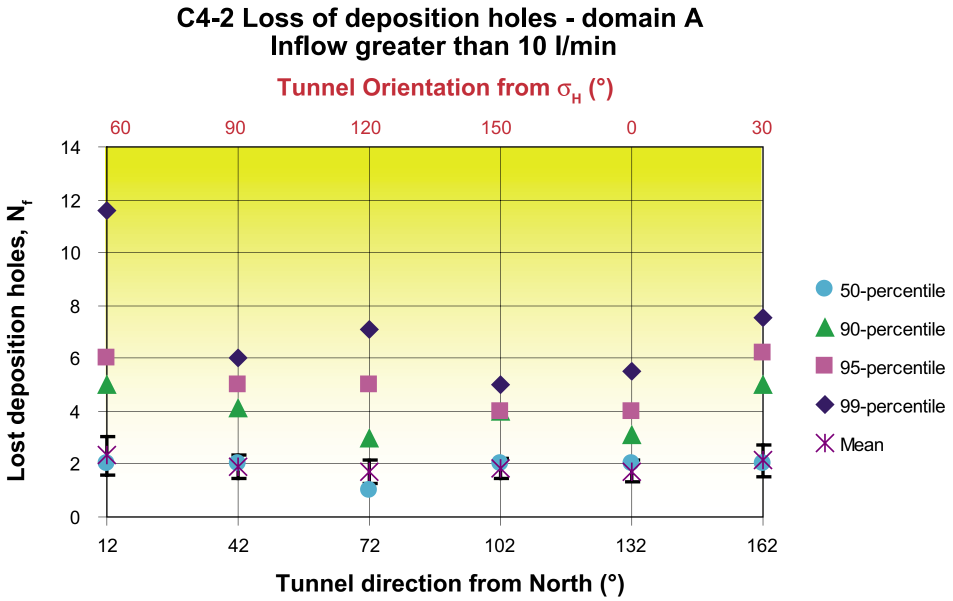


Figure 4-18. *Number of deposition holes with q > 10 l/min (Rock domain A).*

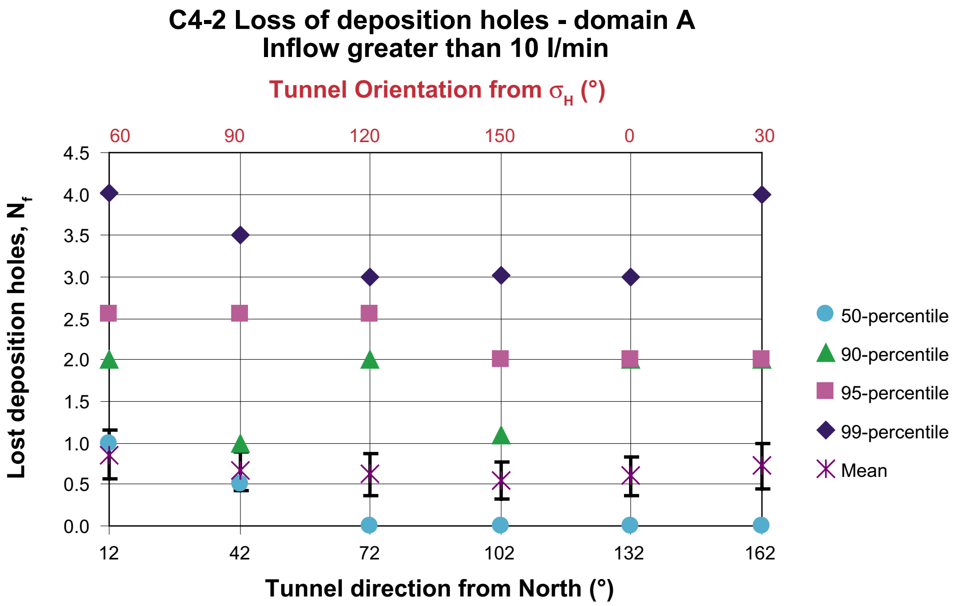
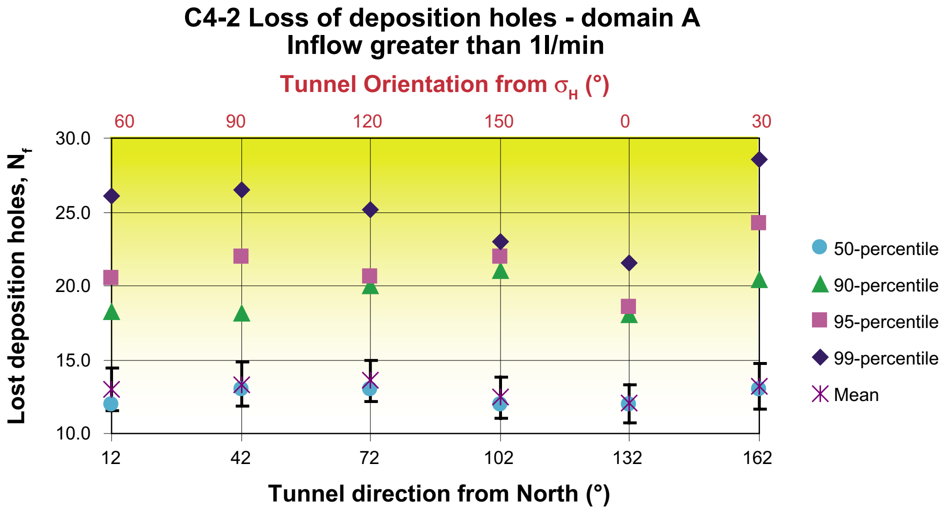


Figure 4-19. *Number of deposition holes with q > 10 l/min (Rock domain M).*

The analysis for Rock domain M indicates a marginal loss of deposition holes between 0.5–0.9 pcs deposition holes per deposition tunnel, which means a loss proportion of 1.4–2.3%. Total seepage to deposition holes in Rock domain M have been calculated as a mean value between 0.08–0.12 l/min/m, i.e. for an 8 m-long hole 0.6 and 1.0 l/min.

The sensitivity of the loss criterion is illustrated by comparing the results for q > 10 l/min in Figure 4-18 and Figure 4-19 with the results for q > 1 l/min and q > 0.1 l/min in Figure 4-20 and Figure 4-21 for the respective rock domains.

If the requirements for the loss criterion are set at q > 1 l/min, the loss proportion increases to 32–36% for Rock domain A. With the loss criterion q > 0.1 l/min, the loss proportion increases to 60–64%. This can be compared to the loss proportion 4.5–6.1% for the criterion q > 10 l/min.



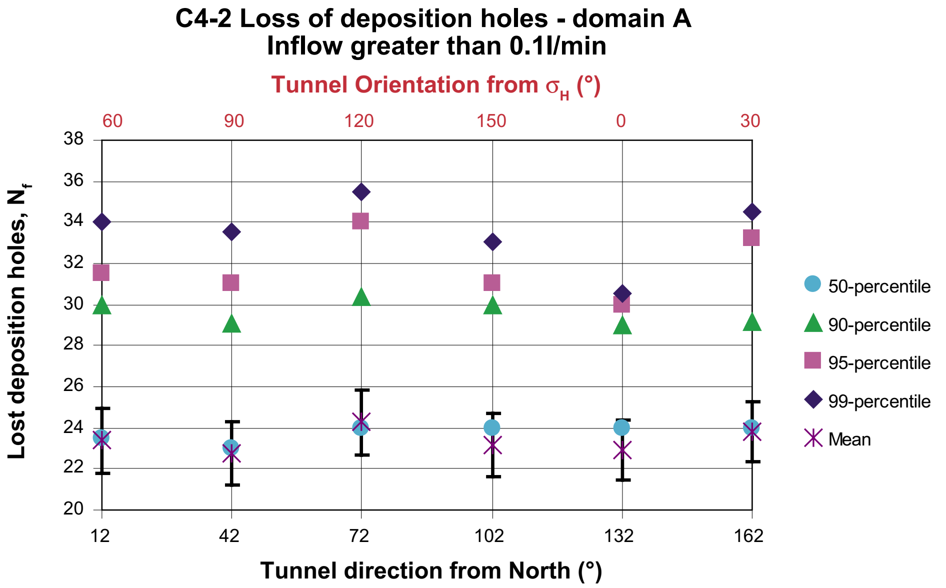


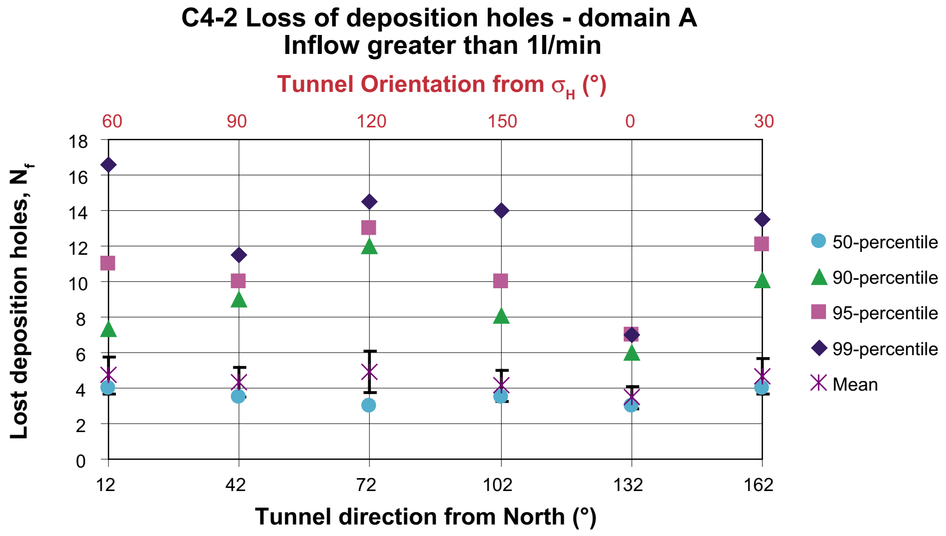
Figure 4-20. *Number of deposition holes that are lost with seepage criterion q > 1 l/min and q > 0.1 l/min (Rock domain A).*

For Rock domain M, the corresponding loss proportion will be 9.1–12.9% at q > 1 l/min and 21–25% at q > 0.1 l/min while the loss proportion for q > 10 l/min it will be 1.4–2.3%.

At Äspö HRL, 13 drilled deposition holes, the rock type for the holes is similar to Rock domain A, have been studied by SKB with respect to seepage. In two of the deposition holes, seepage has been greater than 1 l/min. This gives a loss percentage of approximately 15%, for q > 1 l/min.

Point 3: Loss of deposition holes with respect to the volume of potential unstable wedges in deposition holes

According to an agreement with SKB, the results presented for the wedge analysis have been based on the preliminary DFN-model. The seepage calculations have been updated when the final DFN model for Laxemar was ready, which not the wedge calculation was.



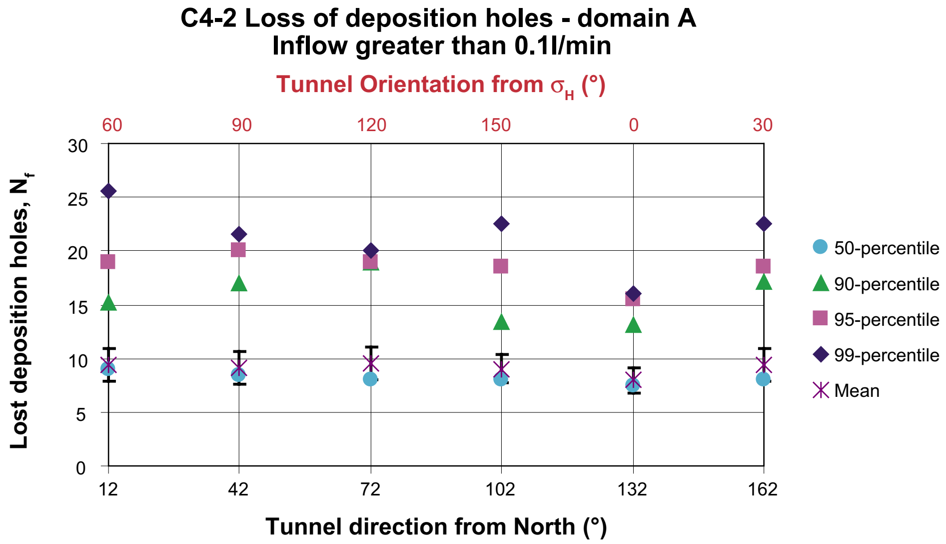


Figure 4-21. *Number of deposition holes that are lost with seepage criterion q > 1 l/min and q > 0.1 l/min (Rock domain M).*

Figure 4-22 shows the distribution of the size of all calculated wedges for 20 simulations in one tunnel direction (N132°). For the simulations, all fractures in the DFN-model were used including sealed fractures. However, wedges smaller than 0.1 m^3 were not presented.

Table 4-8 shows the loss proportions as a percentage for the mean value and different percentiles, with respect to the main criterion $V > 0.15$ m^3. The results are based on 20 simulations with NAPSAC and have been carried out for a depth of 500 m and two tunnel orientations.

Table 4-8. Loss as a percentage for two tunnel orientations, parallel to (132°) and perpendicular to (42°) the largest horizontal stress, for wedges larger than 0.15 m³.

Orientation	Percent of deposition holes containing spalling greater than 0.15 m³					
	50-percentile	90-percentile	95-percentile	99-percentile	Mean value	Std. deviation
42	3.7	10.1	11.1	11.6	5.1	3.9
132	5.9	8.9	9.8	12.0	5	3.5

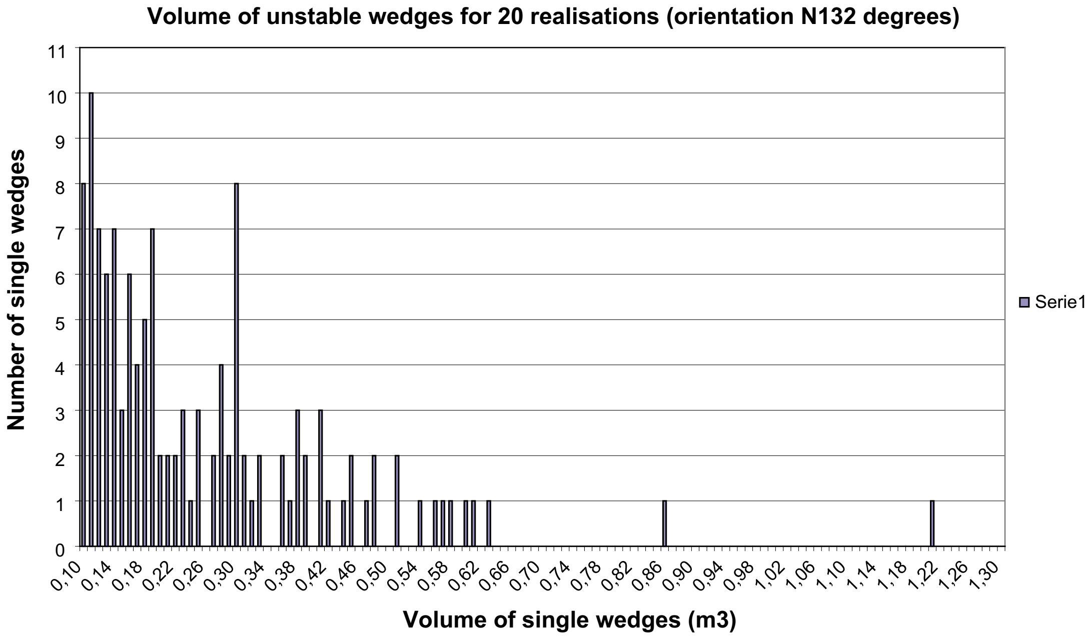


Figure 4-22. *Graph showing the distribution by volume of unstable wedges for 20 simulations and with a tunnel oriented parallel (N132°) to the largest horizontal stress.*

The results of the sensitivity analyses, i.e. with the criteria V > 0.1 m³ and > 0.2 m³, are presented in Table 4-9 and Table 4-10.

At Äspö HRL, there are 17 drilled deposition holes at a depth between 420 and 450 m. In all holes except one, geological mapping has been carried out. The mapping shows that in one of the holes there are wedge volumes caused by the natural fallout of unstable wedges as a consequence of spalling geometry. These volumes of loose rock are small and the wedges failures that have occured are considerably smaller than 0.15 m³, which is the demand for loss of deposition holes.

Table 4-9. Loss as a percentage for two tunnel orientations, parallel to (132°) and perpendicular to (42°) the largest horizontal stress, for wedges larger than 0.1 m³.

Orientation	Percent of deposition holes containing spalling greater than 0.1 m³					
	50-percentile	90-percentile	95-percentile	99-percentile	Mean value	Std. deviation
42	7.2	12.3	13.8	14.5	7.4	3.9
132	6.6	12.5	13	14.4	7.2	4.1

Table 4-10. Loss as a percentage for two tunnel orientations, parallel to (132°) and perpendicular to (42°) the largest horizontal stress, for wedges larger than 0.2 m³.

Orientation	Percent of deposition holes containing spalling greater than 0.2 m³					
	50-percentile	90-percentile	95-percentile	99-percentile	Mean value	Std. deviation
42	3.3	9.2	10.1	10.9	4	3.8
132	3.3	6.9	8.6	9.2	3.7	3.3

Point 4: Loss of deposition holes with respect to stress-induced spalling

The final results with safety factors and probability for stress-induced spalling are presented from Figure 4-23 to Figure 4-25.

The reported results show that the probability for stress-induced spalling increases with depth. The analysis shows that at a depth of 500 m, the probability for stress-induced spalling is approximately 5%, regardless of angle (0, 45 or 90°) to the largest main stress. At a depth of 600 m, the probability for stress-induced spalling increases to about 40%.

When interpreting the study by /Martin 2005/, the following uncertainties must be taken into account:

- Uniaxial compressive strength for the different rock types (between and alongside holes).
- Variations in in-situ stress (25% according to /SKB 2006a/).
- Proximity to weakness zones.
- Variations in tangential stresses around deposition holes (between and alongside holes).

These uncertainties can both reduce and increase the probability of stress-induced spalling.

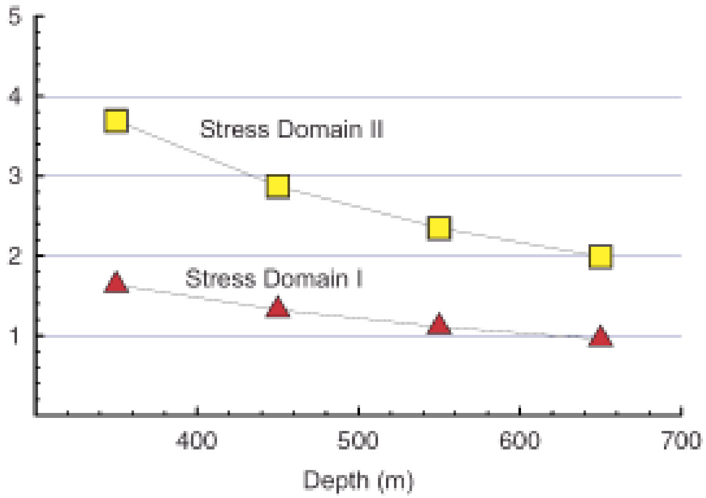


Figure 4-23. *Analysed safety factors (mean value) for stress-induced spalling in deposition holes for Stress Domains I and II /Martin 2005/.*

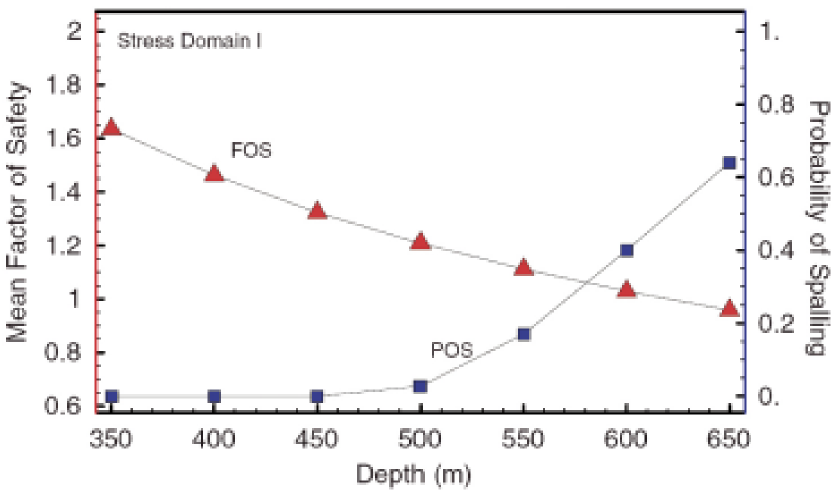


Figure 4-24. *Analysed safety factor (FOS) and probability (POS) for stress-induced spalling in deposition holes for Stress Domain I /Martin 2005/.*

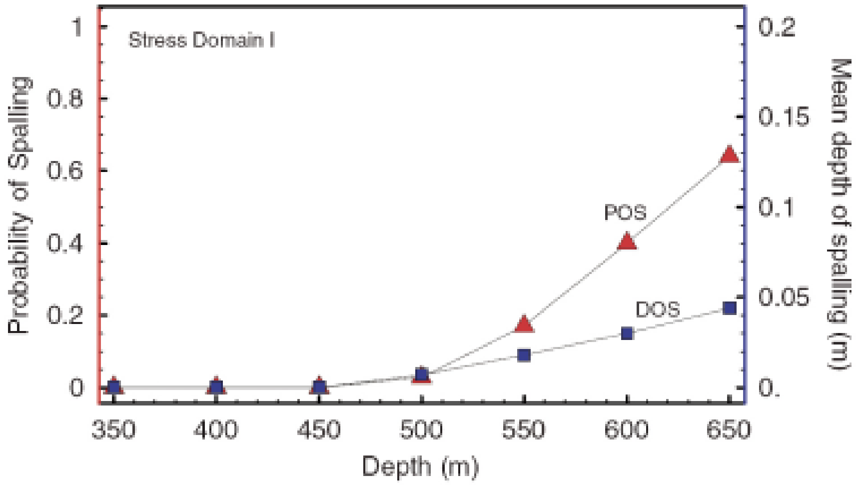


Figure 4-25. *Analysed probability (POS) for stress-induced spalling in deposition holes for Stress Domain I and estimated fracture depth /Martin 2005/.*

It is estimated in the study that at a depth of 500 m, the mean thickness of the spalling is approximately 0.005 m whereas at a depth of 600 m it is estimated to be approximately 0.03 m. In order to obtain the same loss criterion, V > 0.15 m³, as for wedge analysis, the areas of spalling must be larger than 30 m² and 5 m² respectively. This would appear to be unrealistic.

Interpretation of the results of the study shows that we can not expect any wedges with a volume > 0.15 m³ as a result of stress-induced spalling. This means that the loss percentage can be set at zero. It is however highly likely that there will be some kind of spalling as a result of the uncertainty and variation in input data, but for continuation of this project, we will exclusively use the results of the external study by /Martin 2005/.

4.4.4 Discussion

The previous seepage analyses were carried out without disregarding deposition holes which intersect fractures with radii greater than 50 m. An analysis of the overlap between loss of deposition holes due to large intersecting fractures (radius greater than 50 m) and loss due to large seepage (> 10 l/min) has been performed for Domain A and tunnel direction N132°. The results indicate almost 100% overlap. Hence the loss of deposition holes as a result of large seepage (> 10 l/min) may be much lower than the analysed results since the deposition holes do not intersect a fracture with radius greater than 50 m.

The results of the wedge failure analyses indicate that the loss could be about 5% if we choose the volume criterion with wedges larger than 0.15 m³. The calculations also show that the difference in the loss percentage does not depend in particular on the tunnel orientation. It does not really make any difference whether the deposition tunnels are oriented parallel to or perpendicular to the largest horizontal mains stress. Nor is there any difference between the depths of 500 m and 600 m. A review of the results from drilled deposition holes in Äspö indicates a zero per cent loss, which affects the assessment of the loss proportion for future design.

The external study by /Martin 2005/, indicates that at a depth of 500 m, the probability for stress-induced spalling is 5%, but the volume of the overbreak is minimal. At a depth of 600 m, the probability of stress-induced spalling is significantly higher, 40%, but also here it is considered to be a question of small volumes of overbreak.

The overall assessment of the total loss percentage for all analyses is presented in Table 4-11.

Table 4-11. Compilation of recommended loss percentage for continued layout work.

Depth	Due to elongated fractures	Due to seepage	Due to wedges	Due to stress-induced spalling	Total
500 m	16%	1.5–6%	0%	0%	17.5–22%
600 m	16%	–	0%	0–40%	16–56%*

* No analysis for water seepage.

4.5 Repository depth

4.5.1 Input data and assumptions

The purpose of this section is to specify recommended repository depth based on analyses and results from previous sections, i.e.:

- Respect distance to deterministically determined deformation zones and the potential of the site to accommodate the repository which is constructed at depths of 400, 500, 600 and 700 m.
- Design of deposition tunnels, deposition holes and main tunnels.
- Distance between deposition tunnels and between deposition holes, which are constructed at depths of 400, 500, 600 and 700 m.
- Orientation of deposition tunnels and which are constructed mainly for depths of 500 and 600 m.
- Loss of deposition holes and which are constructed mainly for depths of 500 and 600 m.

4.5.2 Execution

Results from the analyses presented in previous sections are collected in order to specify a recommended depth:

- In Section 3.2, an understanding is given for the entire deposition area where the deformation zones that cut off the whole area in many disposal areas are also considered.
- In Section 4.1, basic input is given for the design of the deposition tunnels, deposition holes and main tunnels.
- In Section 4.2, the distance between deposition tunnels and between deposition holes is given with consideration of the thermal properties.
- In Section 4.3, optimal tunnel orientation has been analysed with consideration to: quantity of water seepage, the risk of spalling and the volume of unstable wedges.
- In Section 4.4, the loss proportion of deposition holes has been analysed with respect to: elongated fractures, seepage, wedge failure and the risk of spalling.

4.5.3 Results

The available deposition areas within the entire Laxemar area are enough to accommodate the repository and varies between 5.30 and 5.58 km^2 depending on the repository depth (400–700 m) even when consideration is given to the determined deformation zones and their distances (respect distance, MFE and distance for uncertainty). Most of the deformation zones within the area of the repository facility, are assumed to be vertical or sub-vertical in the site description /SKB 2006a/. This means that the differences between the various depths are small and that the available deposition area is therefore not dependent on depth with respect to the deformation zones.

The design of deposition tunnels, deposition holes and main tunnels is independent of depth. The cross-sectional geometries are shown in Figure 4-1.

The distance between the first deposition hole and main tunnel shall be 20 m and the distance between the tunnel face and deposition holes shall be 8 m.

The distance (c/c) between deposition tunnels is set at 40 m irrespective of depth. The distances between deposition holes vary with respect to the rock domain and its thermal properties as well as the initial temperature at respository depth. For a depth of 400 m, the shortest c/c distance is obtained between the deposition holes because the initial temperature increases with depth. The thermal properties are constant with depth as per site description and therefore they do not influence the choice of recommended depth.

Three analyses with respect to optimum deposition tunnel orientation have been made:

- regarding the quantity of water seepage into the deposition tunnels,
- regarding the risk of spalling in deposition tunnels,
- depending on the volume of unstable wedges in deposition tunnels and deposition holes.

The two methods of analysis, one analytical and one numerical, for the quantity of water seepage, have been performed for the depth of 500 m only. The analyses indicate that the optimal tunnel orientation is N102-132°. The inflow depends on the groundwater pressure, which increases with depth, and the hydraulic conductivity of the in situ rock mass that decreases with depth /SKB 2006a/. Increasing or decreasing depth is not expected, however, to change the optimal tunnel orientation that is calculated for the depth of 500 m.

The risk of spalling is negligible down to 500 m depth, irrespective of tunnel orientation, and after that depth, the risk of spalling increases. At a depth of 600 m and the least favourable tunnel orientation, the probability of spalling is 40% /Martin 2005/, although the volume of overbreak is small.

The optimal orientation of the deposition tunnels with respect to unstable wedges is approximately N12° irrespective of whether the depth is 500 or 600 m.

Analyses of the loss proportion for deposition holes have been conducted by considering the:

- elongated fractures,
- water seepage,
- wedge failure,
- the spalling phenomenon.

The results of analyses of elongated fractures have only been assigned one value, regardless of depth.

The two methods of analysis, one analytical and one numerical, for the quantity of water leaking in have been performed for a depth of 500 m only. The analyses indicate a loss of between 1.5 and 6% for the criterion $q > 10$ l/min, depending on the rock domain. The inflow to the deposition holes depends on the groundwater pressure, which increases with depth, and the hydraulic conductivity of the rock mass, that decreases with depth /SKB 2006a/. For a recommendation of depth, it is assumed that the inflow is of the same order of magnitude between 400 and 600 m irrespective of depth.

The analysis concerning wedge failure in deposition holes indicates that the difference between the depths of 500 and 600 m is negligible.

In the separate study for spalling phenomenon, the results indicate that the probability of spalling increases with depth. The results presented indicate at a depth of 500 m a probability of spalling of less than 5% and at 600 m approximately 40%, but the volume of overbreak is small at both depths.

4.5.4 Discussion

The various analyses give the following recommendations for continued design with respect to recommended repository depth:

- All repository depths are suitable for continued layout work with regard to the locations of deformation zones and their protective distances (respect distance and margin for excavation).
- The distance between main tunnels and the first deposition holes, and between deposition holes and the tunnel face, is the same regardless of repository depth.
- A repository depth of 400 m is recommended in preference to other depths. This is because a depth of 400 m gives the smallest c/c distance between deposition holes as a result of the fact that the initial temperature increases with depth.
- Favourable with a low hydrostatic pressure as possible, i.e. the location of the repository as near to the surface as possible. However, with this follows a lower hydraulic conductivity as this parameter is depth dependent /SKB 2006a/. Thus this conclusion is uncertain.
- The recommendation for continued layout work is to locate the repository at a depth of 400 or 500 m in order to avoid spalling rock in the deposition tunnels. Other results with respect to orientation of the deposition tunnels (water seepage and wedge failure) are not affected by increasing depth.
- The analyses for loss proportions of deposition holes, conducted primarily for 500 and 600 m, gives as a recommendation for continued layout work the location of the repository as near to the surface as possible. This is primarily in order to avoid spalling rock in the deposition holes. Other results with respect to loss proportion (elongated fractures, water seepage in and wedge failure) are not affected, or are marginally affected, by increasing depth.

By considering all of the above, it is recommended that the repository should be located as close to the surface as possible, i.e. at a depth of 400 m.

However, there are factors of importance for the long-term safety that are not considered in the Design premises /SKB 2004a/. Several of these factors will, according to SKB, result in a deeper placement of the repository.

Since the benefits of placing the repository at 400 m compared to the initial reference level of 500 m are marginal, the reference level of 500 m has been maintained for the purposes of the current Laxemar D1 layout.

4.6 Design of other rock excavations

4.6.1 Demands

For design step D1, no requirements are made on specific documentation of the design of other rock cavern, since these are available in Layout E /SKB 2002/. Design of other rock excavations, as tunnels and rock caverns in central area, shaft, ramp and transport tunnels (see Figure 1-2) 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 other rock excavations 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 location of other rock excavation (central area etc) is illustrated in Chapter 5 together with the tunnels in the deposition area. More detailed design of the other rock excavations will be carried out in later design steps.

4.6.2 Layout of other rock excavations

Figure 4-26 illustrate the central area with ramp and shafts.

The central area will be constructed for the operation of the repository and deposition of canisters. The design and location o tunnels and rock caverns in the central area are given by SKB. Figure 4-27 show a schematic plan of the central area.

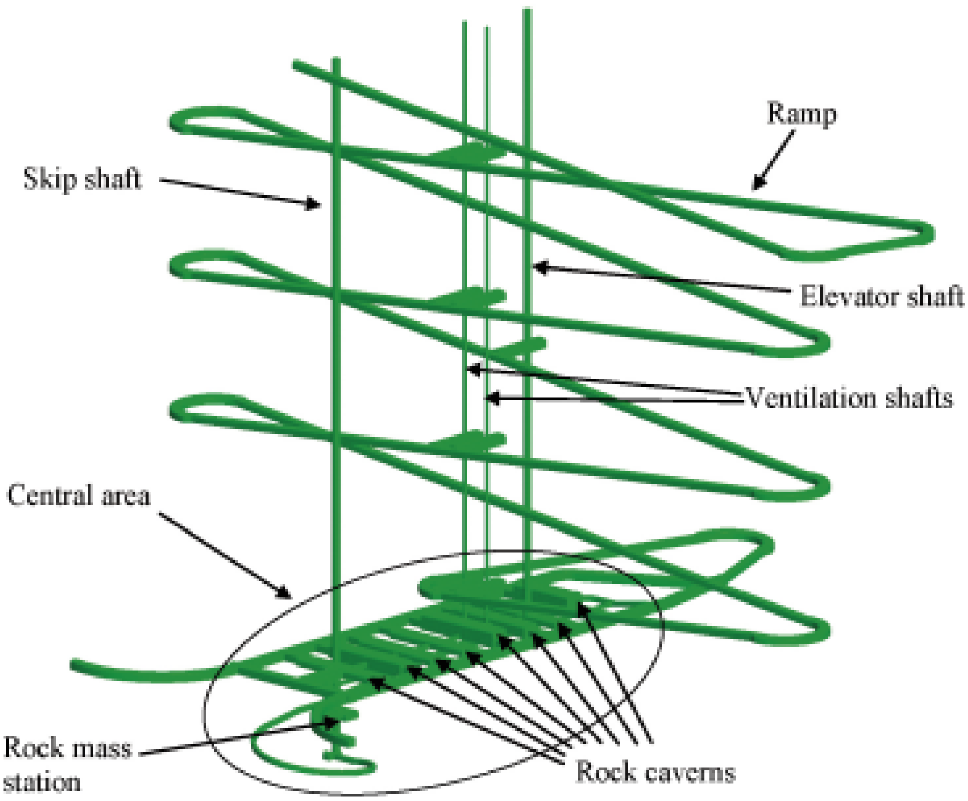


Figure 4-26. *Central area with ramp and shafts (figure from SKB).*

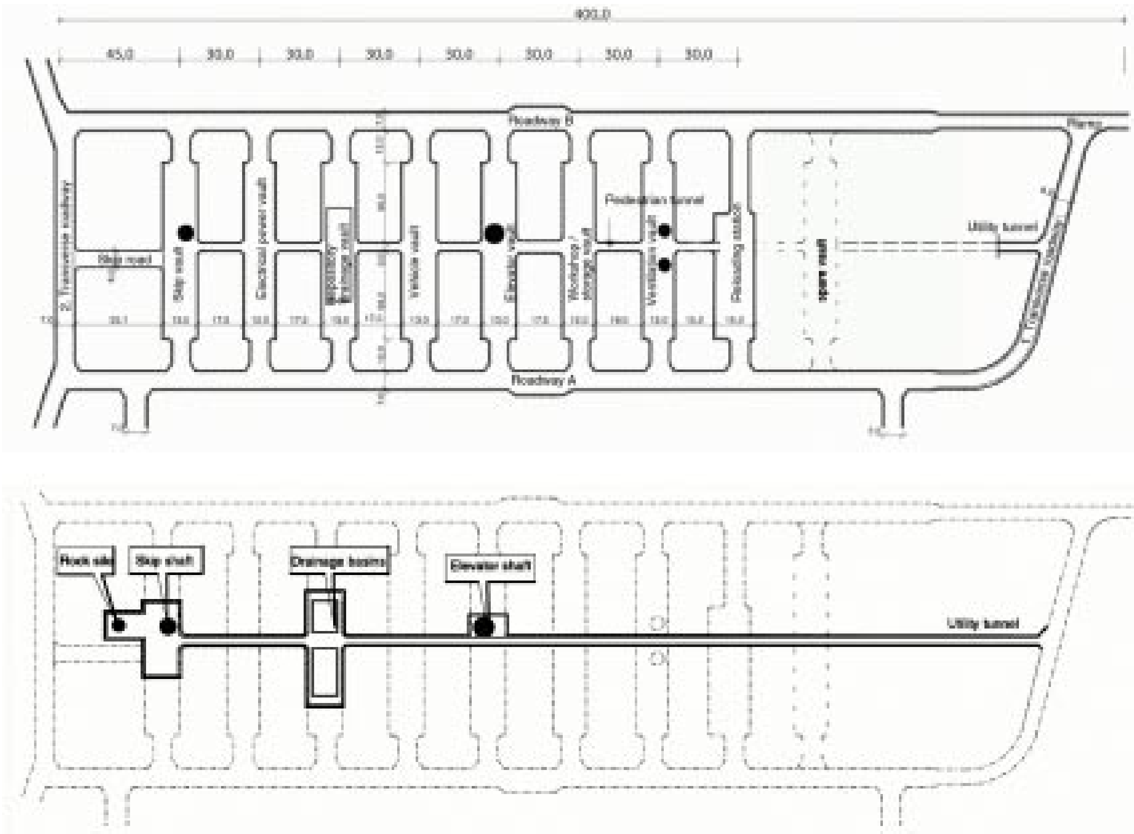


Figure 4-27. Schematic plan of the central area /SKB 2002/.

The functions of the central area are to:

- transport of personnel,
- ventilation,
- handling of excavated rock,
- power supply,
- water handling,
- workshops and storage facilities,
- handling of canisters with spent fuel,
- handling of bentonite for the buffer.

5 Layout studies

5.1 General

Based on results presented in the previous chapters, possible design alternatives for the repository were studied. A total number of 6,000 deposition holes, for the canisters, will be accommodated within the repository.

5.2 Execution

The various alternatives for how the repository should be designed have been developed on the basis of UDP /SKB 2004a/. The results presented in Chapters 3 and 4 have served as important basic input for determining the optimum location. In some cases, consultation with SKB has resulted in deviations from UDP. A list of the specific preconditions for the layout work at Laxemar is given below:

- SKB has decided that layout alternatives will only be developed for the depths of 500 m and 600 m. Two layout alternatives will be designed for each depth.
- The location of the operational area, the central installation or centre, above ground will in turn determine where the central area at a specific depth can be situated below ground. Restrictions for the alternative locations proposed by SKB for this centre apply.
- Only one layout will be selected as the "basic layout", for further development. A sensitivity analysis has been performed for the selected basic layout.
- The orientation of the deposition tunnels has not been adapted to the optimum orientations in the layout that are presented in Section 4.3.
- In the sensitivity analysis of the basic layout, four changes have been studied. A more detailed sensitivity analysis of the layout is presented in Chapter 10, where changes in a larger number of factors are analysed.

Furthermore it was decided by SKB that the area east of "Kustvägen" was not to be included in the design studies. Later it was clear that it can be used. The excluded area is shown in Figure 5-1 to Figure 5-7.

5.3 Alternative possibilities for site application

The final repository should, according to Layout E /SKB 2002/, be designed around the central area (see Figure 1-1 and Figure 1-2. The repository consists of three different types of tunnels apart from the central area:

1. Deposition tunnels that contain the actual deposition holes in which the canisters will be stored.
2. Main tunnels that connect the deposition tunnels together.
3. Transport tunnels that run between different deposition areas and between the deposition areas and the central area.

The location of the operational area above ground controls, to great extend, the localisation of the repository at depth. The alternative locations that are proposed in Laxemar for the operational areas above ground by SKB are shown in Figure 5-1. The locations are referred to as alternatives "West" and "Central" and will not be mixed up with designations of the underground deposition areas which are divided up into central, south, north, west and east. The designed layout depth alternatives are 500 m and 600 m.

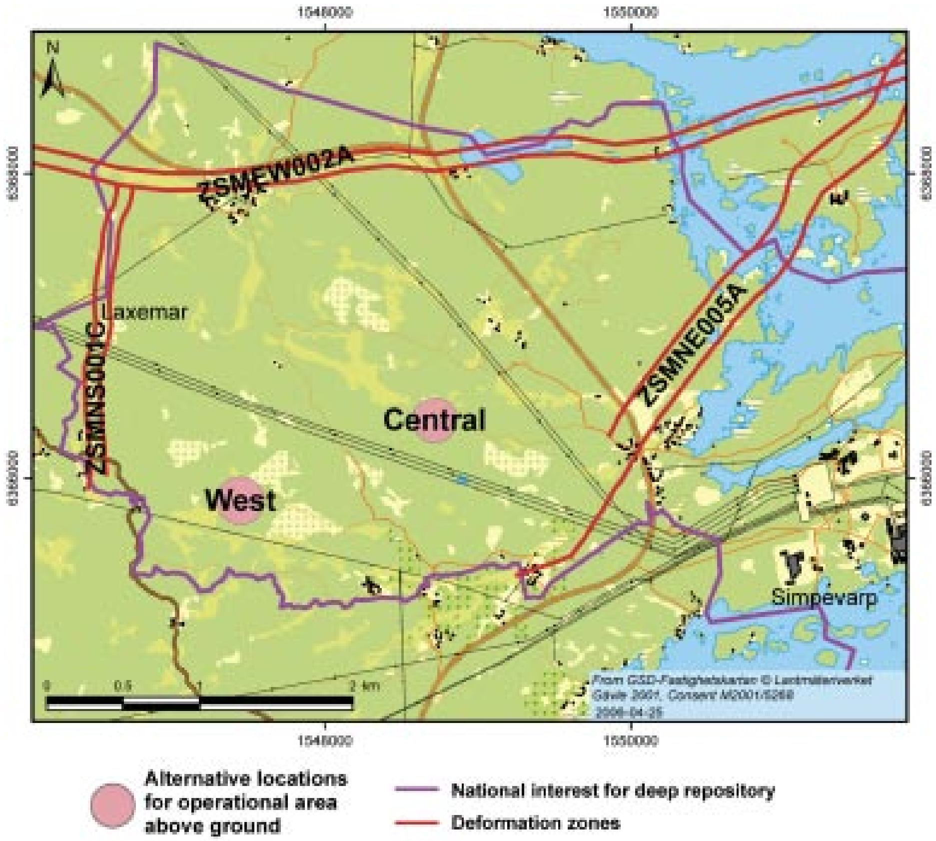


Figure 5-1. Alternative locations for an operational area above ground designated Central and West. The figure also shows how the Laxemar area is delimited by deformation zones and national interest for a deep repository.

Number of deposition holes

The extent of different rock domains within the Laxemar area has already been established as well as the deposition areas that can be placed between the zones of deformed rock. Figure 5-2 and Figure 5-3 show the location of the available deposition areas in relation to the rock domains at depths 500 m and 600 m. As many deposition holes as possible will be situated within these areas.

The thermal properties of the rock domains vary, which means that the smallest distance between deposition holes differs for different rock domains. The distance between deposition holes for the rock domains in Laxemar is described in Section 4.2. At a depth of 500 m, the c/c distance between deposition holes varies from 7.0 to 8.1 m depending on the specific rock domain concerned. At a depth of 600 m, the span is between 7.4 and 8.6 m. For rock domain M (RSMM), there are two alternative c/c distances depending on two alternative interpretations of the thermal conductivity in the bedrock, see SDM v 1.2 and /Sundberg et al. 2006/. The mean values for hole distances that are used for the different layout alternatives are presented in Table 5-1.

The most advantageous tunnel orientations with respect to water seepage, wedge stability and rock stress, have been analysed in Section 4.3. The difference between various orientations has proven to be very small. In the following discussions with SKB it was therefore decided that the tunnels should only be oriented with respect to geometrical conditions. The attempts to optimise the number of deposition holes within each deposition area have therefore been performed without considering the orientation of the tunnels.

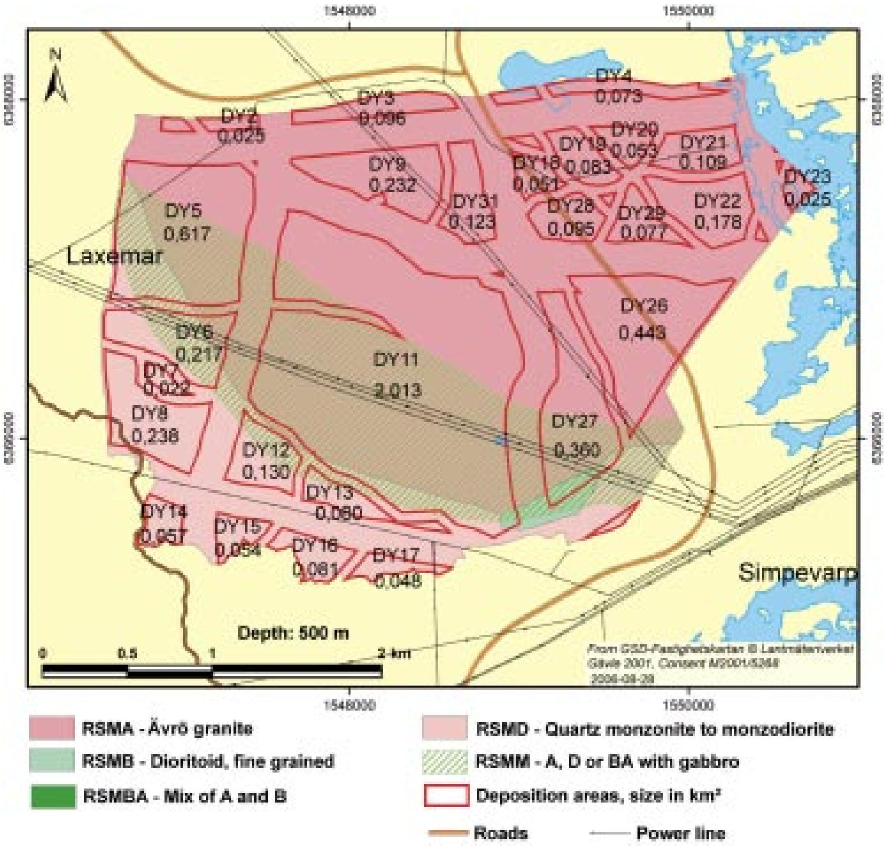


Figure 5-2. *Extent of rock domains and available deposition areas at a depth of 500 m.*

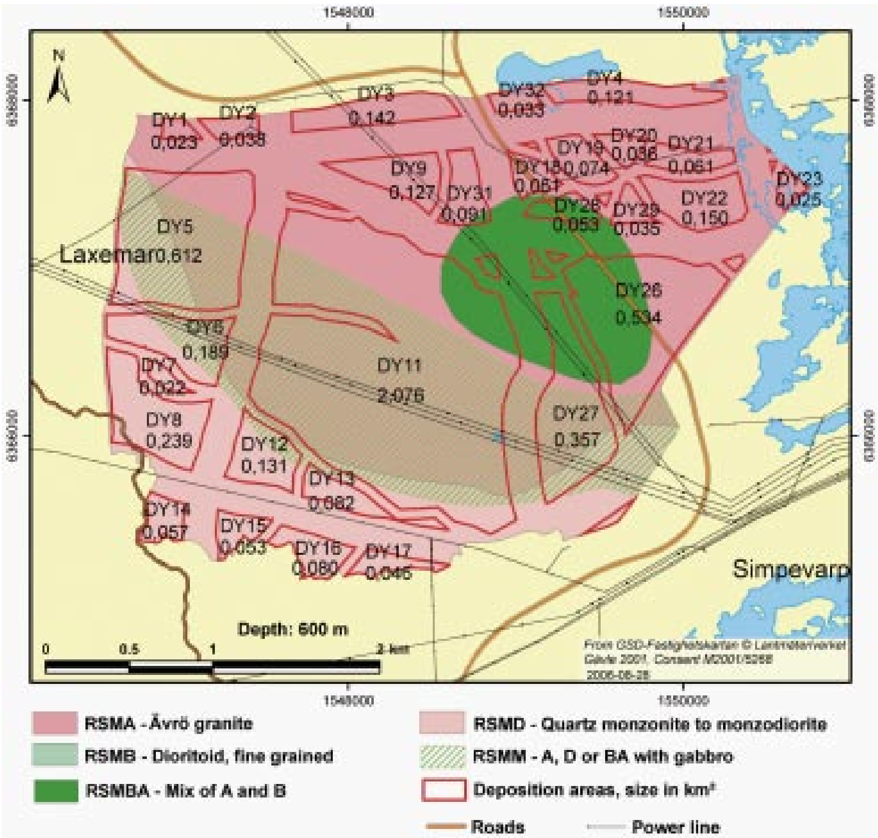


Figure 5-3. *Extent of rock domains and available deposition areas at a depth of 600 m.*

Table 5-1. Distance between deposition holes that have been used for the different layout alternatives.

Subarea within Laxemar	Deposition areas within the demarcation lines	Mean distance depth 500 m	Mean distance depth 600 m
South	DA7, 8, 12, 13, 14 15, 16, 17	7.6 m	8.1 m
Central and West	DA5, 6, 11	7.4 m	7.8 m
North and East	DA2, 3, 9, 18, 26, 27, 28, 31	7.2 m	7.6 m

Considering the number of deposition holes that have to be discounted within a certain deposition area as a result of elongated fractures/fracture zones, water seepage or unstable rock, the depth of 500 m is more advantageous than 600 m. The overall loss of deposition holes assessment for the depth of 500 m is between 17.5–22%, see Section 4.4. For 600 m the loss is between 16–56%, but for this level no analysis for water seepage is available. The loss due to water seepage was 1.5–6% at the 500 m level.

For further studies in this chapter 20% and 40% overall loss of deposition holes will be used for the 500 m and 600 m alternatives respectively. These values were decided after discussion with SKB.

In summary, this means that the distance between deposition holes and the calculated loss, dictates how many deposition holes the different layout alternatives incorporate. The demand is to have 6,000 holes after reduction. For the depths in question, the overall planning needs to cover the following numbers of holes:

- The layout alternative for a repository depth of ***500 m*** is planned for ***7,500 deposition holes*** (including loss), from which 250–500 holes for initial operation.
- The layout alternative for a repository depth of ***600 m*** is planned for ***10,000 deposition holes*** (including loss), from which 330–660 holes for initial operation.

Construction of the facility

An operational area will be constructed above ground. The location is dependent on factors both above and below ground. Two alternative locations have been proposed by SKB, see Figure 5-1. They are designated as the West Alternative and the Central Alternative. A layout proposal has been developed for each depth, i.e. 500 m and 600 m, for each of the areas. In addition to the location above ground, the central area under the ground is defined by the location of the deformation zones that in a large-scale intersect the area.

The underground central area will be excavated by blasting the ramps and full-face boring of vertical shafts. This will be followed by excavation of the transport-, main- and deposition tunnels. The deposition holes in the area will be bored successively. The layout of the facility gives several alternative extraction sequences for the deposition units.

Volume of excavated rock

The excavated rock volumes for each alternative are presented in Table 5-2.

The volume of excavated rock varies depending on the type of tunnel excavation, the length of the tunnel, its cross-sectional area, etc. It is in fact the length of the different types of tunnel that is the distinguishing feature of the various layout alternatives. The volume of excavated rock for the central area has not been specified for the different alternatives since it is the same for all layout alternatives, apart from the difference in depth. The tunnels located at a depth of 600 m must generally be longer in order to accommodate the planned number of deposition holes, which means that the volume of blasted rock will be greater. The blasting volume does not differ much between the Centre alternative and the West alternative for one and the same depth. At a depth of 500 m, the volume of blasted rock will be approximately 2 million cubic metres and at 600 m approximately 3 million respectively.

Additional deposition areas

When the final repository has been accommodated in a certain area, there are deposition areas remaining around the proposed layout. This means that there is space to locate several more deposition holes, something that applies for the 3 of the 4 alternative designs. Only in Alternative West 600 m, is there a lack of potential for further deposition holes. The locations of the additional areas are shown in Figure 5-4 to Figure 5-7.

Considerably more additional areas are available at a depth of 500 m than at 600 m. The potential for the largest number of extra deposition holes, 3,390 pcs, exists in the case of the West alternative at a depth of 500 m (see Table 5-2). The overall deposition hole loss of 20% has been deducted.

Choice of basic layout

A list of excavated rock volumes, after blasting, and the number of additional deposition holes for the various layout alternatives is shown in Table 5-2. The difference between the two depths is considerable: the volume of rock will be approximately a million cubic metres or almost 50 per cent more at a depth of 600 m compared to 500 m. Furthermore, all the available surrounding additional areas are used up in both the 600 m alternatives. The greatest difference is the fact that the loss of deposition holes is significantly greater in the 600 m compared to the 500 m alternative respectively. At 500 m depth, the loss is 20% and at 600 m it is 40%. This has been analyzed in Section 4.4.

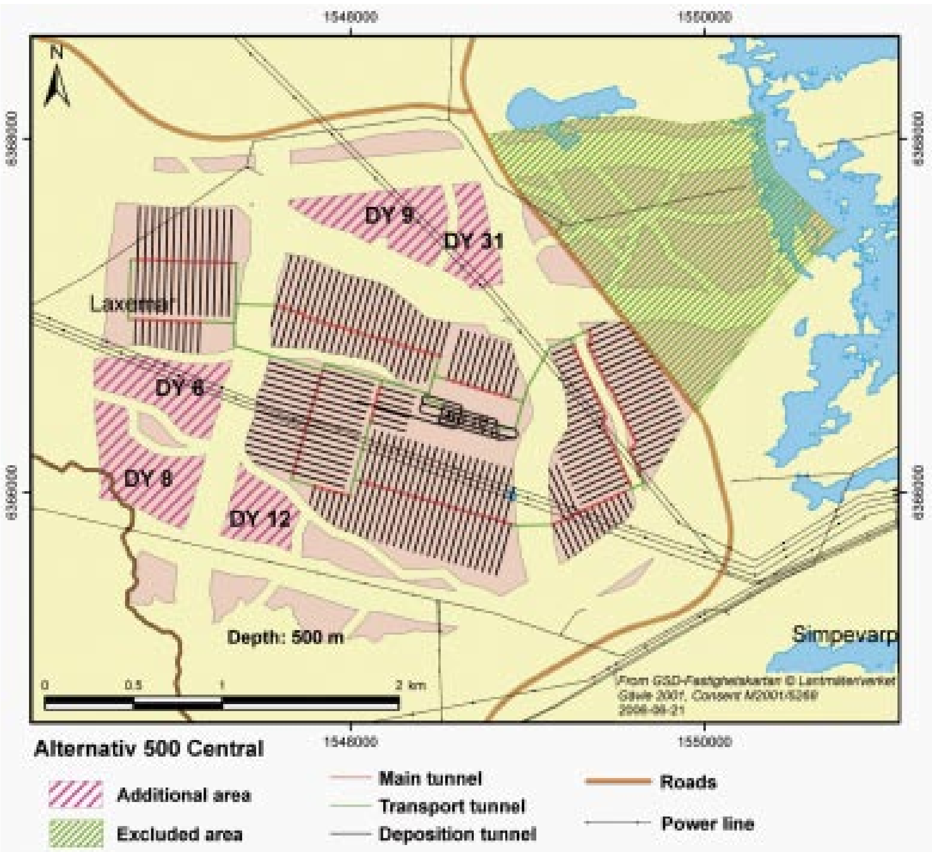


***Figure 5-4.** Additional areas at location alternative Central, 500 m depth. The excluded area can be used but were not included in this study.*

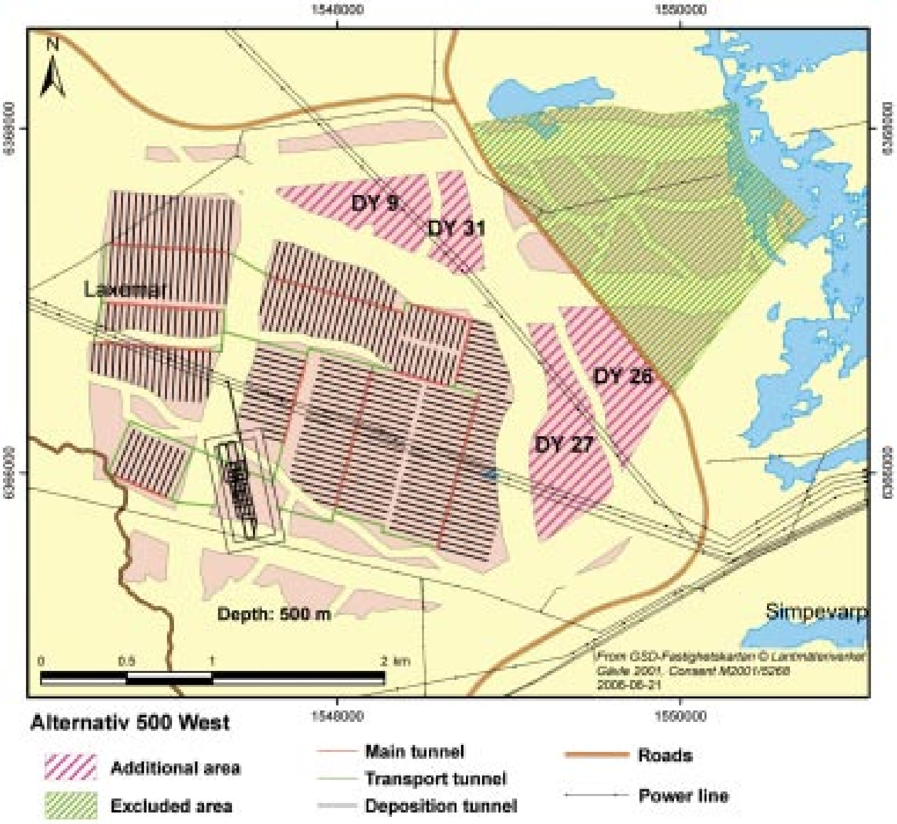


Figure 5-5. Additonal areas at location alternative West, 500 m depth. The excluded area can be used but were not included in this study.

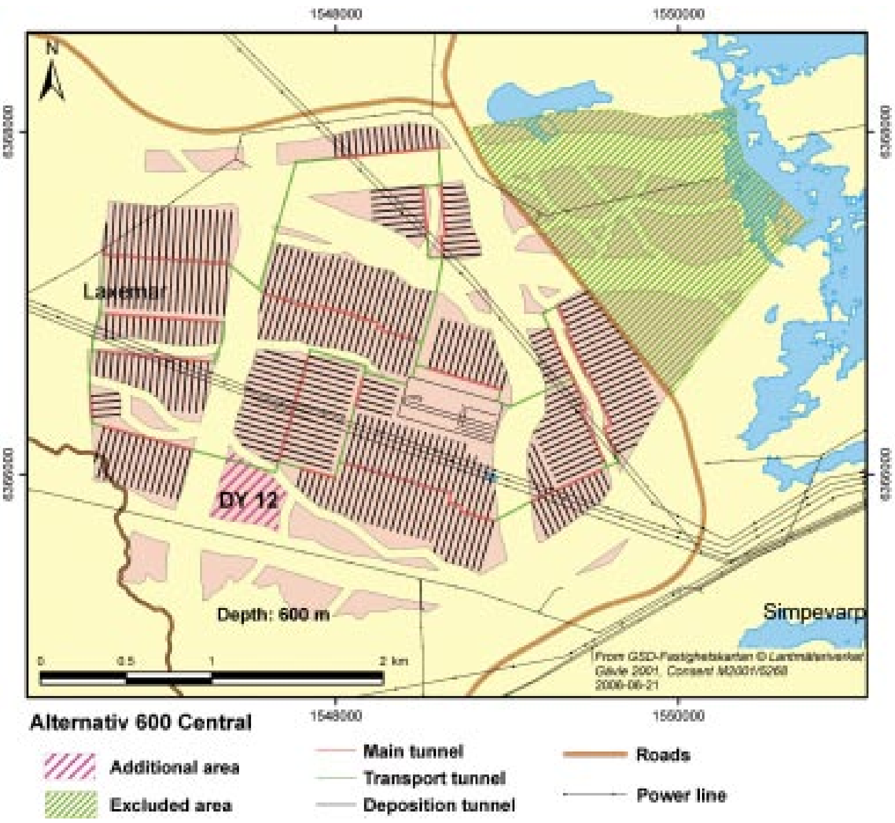


Figure 5-6. Additonal areas at location alternative Central, 600 m depth. The excluded area can be used but were not included in this study.

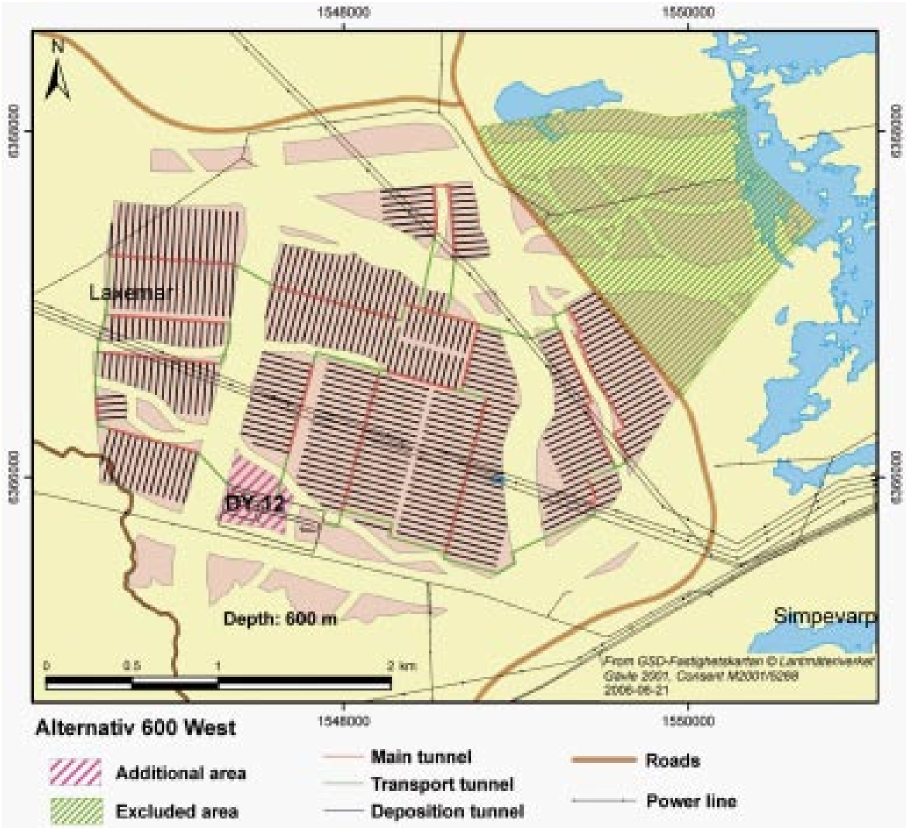


Figure 5-7. Location West, 600 m depth, where no additonal areas are available. The excluded area can be used but were not included in this study.

However, there are also differences between the alternatives at the more favourable depth of 500 m. The volume of rock is largely identical for both alternatives, but when it comes to the area of additional deposition areas there are differences. According to the calculation method that has been used, the alternative West 500 m provides space for nearly 800 more additional deposition holes than alternative Central at the same depth. It was, however, decided by SKB that the alternative "Central" and a repository depth of 500 m should be the base layout for the continuation of this study. The reason for this is that this location is considered to be more advantageous when it comes to the conditions for the above-ground facility. Consequently, for the purpose of continued layout work in this context, alternative 500 Central has been used as the "basic layout". The total lengths and volumes of the excavated rock mass for the different tunnel types for the basic layout are presented in Table 5-3.

Table 5-2. List of preconditions for the different layout alternatives.

Alternative	Rock volume after excavation (m^3)	No of additional deposition holes
500 Central	2,067,973	2,594
500 West	2,072,197	3,390
600 Central	2,967,937	252
600 West	3,096,376	0

Table 5-3. Length and excavated volume for each tunnel type for alternative 500C which was chosen as "basic layout".

	Length (m)	Rock volume after excavation (m³)
Deposition tunnels	60,620	1,515,500
Transport tunnels	4,600	212,980
Main tunnels	6,500	429,650
Central area	–	154,000
Ramps	–	181,500
Shafts	–	41,250
Total	–	2,534,880

Concerning the repository in its full extend, it will not be in use at the same time. The excavation of the repository will be done in a set of construction steps and each step will be held open for a limited time. These construction steps were decided by SKB and are shown in Figure 5-8.

3-D illustrations of the basic layout

In order to give an impression of the location of the basic layout in relation to the deformation zones 3-D illustrations of the repository and deformation zones are presented i Figure 5-9 and Figure 5-10.

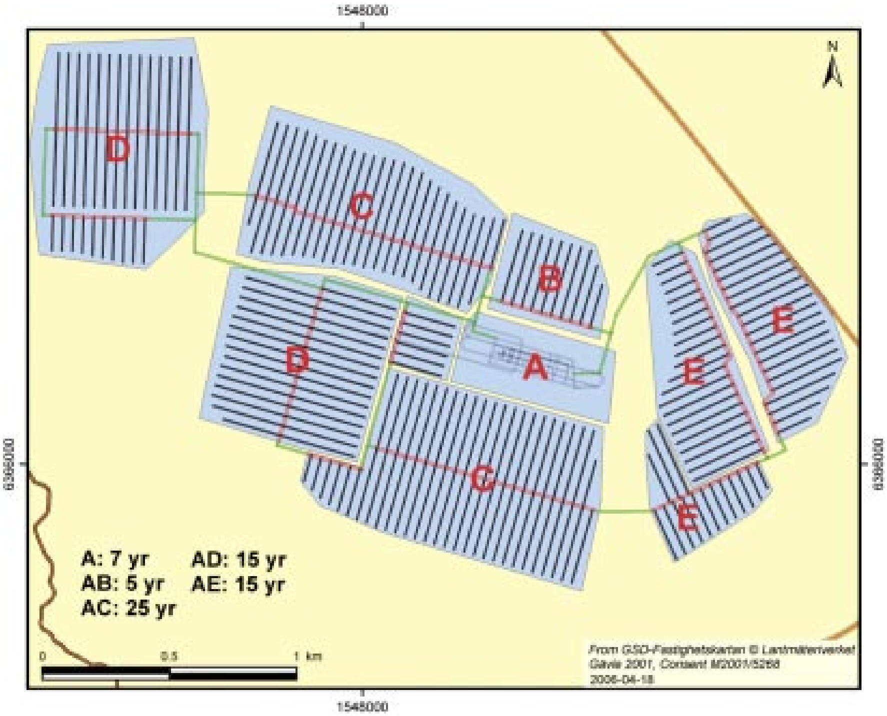


Figure 5-8. Location for the different constructions steps and the time range each step is constructed and held open.

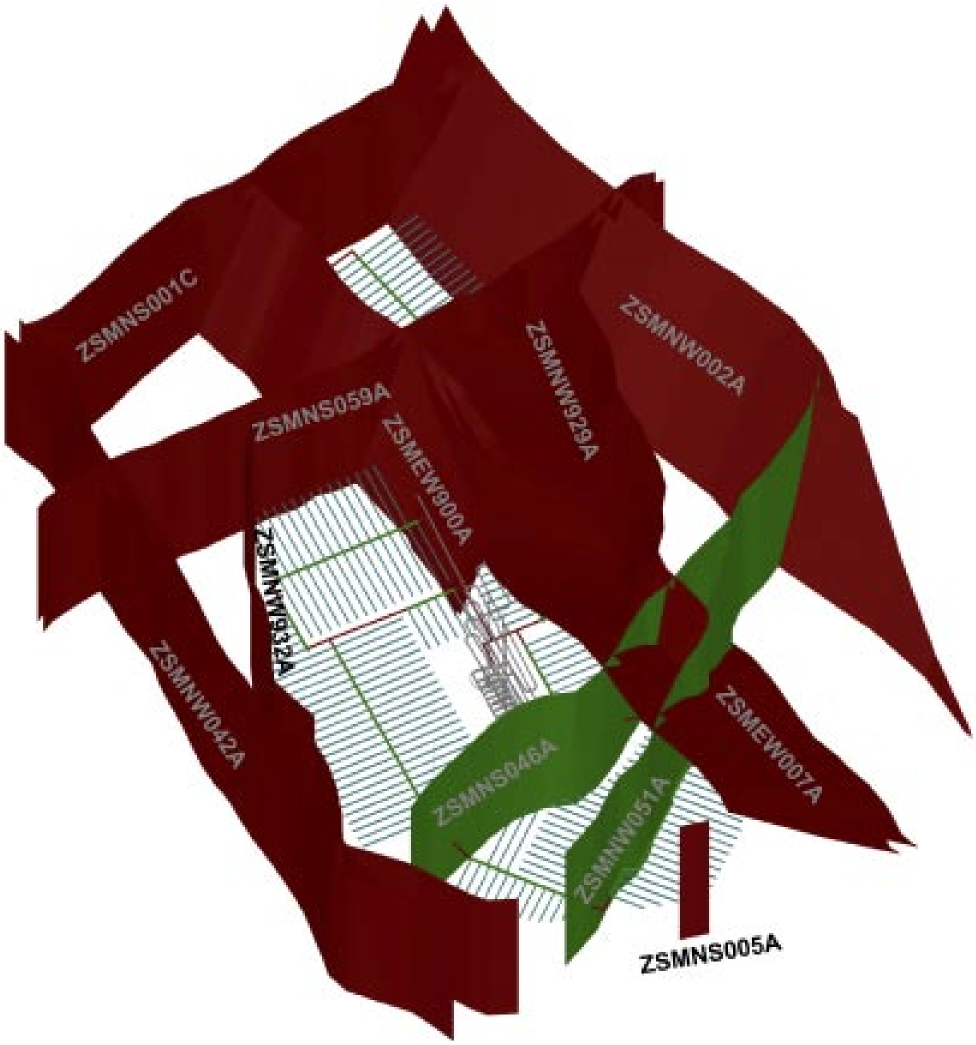


Figure 5-9. *3-D Illustration of the whole repository and deformation zones. Only the zones included in the sections are shown.*

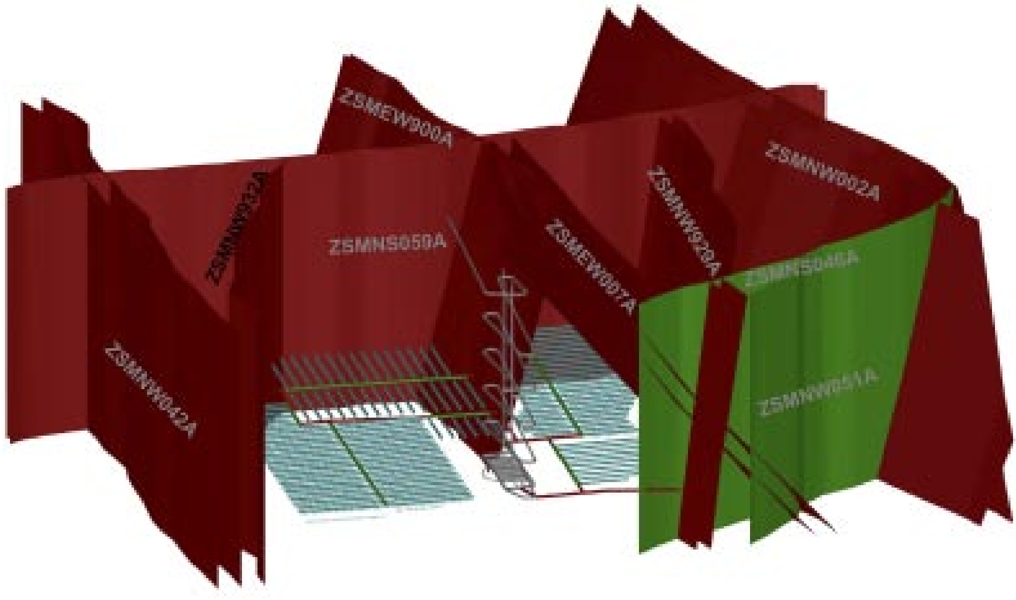


Figure 5-10. *3-D Illustration showing parts of the repository and deformation zones. Only the zones included in the sections are shown.*

Exhaust air shafts and caverns of the central area

Separate exhaust air shafts are needed in the repository facility. In order to fulfil maximum function, these shafts have been situated as far as possible from the central area (see Figure 5-11). One shaft is located in the north-western corner of the facility and one in the south-eastern corner. The various restrictions and considerations that are made must of course be observed at a later detailed design stage.

General outline of the caverns of the central are shown in Figure 1-1 and Figure 1-2. It is described in detail in Layout E /SKB 2002/.

5.3.1 Sensitivity analysis

Sensitivity analysis deals in a general way with the type of negative consequences a changed layout conditions could give. No changes that could lead to positive consequences have been studied at this stage.

The changes in design parameters that have been analysed concern the following aspects:

- Presence of deterministically-interpreted deformation zones.
- Orientation of deterministically-interpreted deformation zones.
- Mean value of the thermal conductivity.
- Criteria for the loss of deposition holes.

Changes in the rock volume, after excavation, have not been calculated for the different changes in design parameters. The consequences that only affect the available area and the number of deposition holes in the basic layout have been studied.

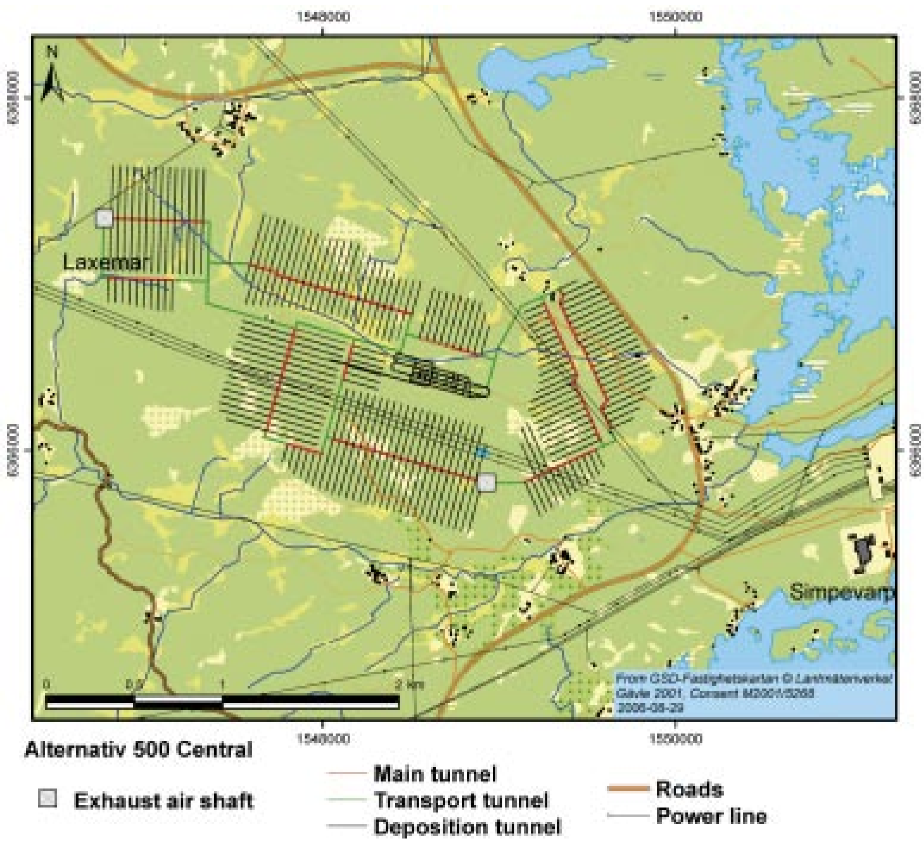


Figure 5-11. Location of exhaust air shaft for alternative 500 C which was chosen as basic layout.

Presence of deterministically-interpreted deformation zones

There are a number of identified factors that could influence the design of the layout in a negative way. Here, the impact of deformation zones with a low level of confidence has been studied. The influence of the low confidence zones can be studied by inserting them in the layout, see Figure 5-12. They have been given a MFE and a distance for uncertainty of 30 m.

The deformation zones with a low level of confidence intersect the area in a way that the basic layout is divided up into 13 different deposition areas instead of 4. The area available in the basic layout decreases by approximately 322,400 m^2 and 10% of the available deposition holes are lost.

Orientation of deterministically-interpreted deformation zones

The deformation zone that is assumed to have the greatest impact on the layout and whose location on the 500 m level is uncertain is zone ***ZSMEW007A***. It was decided to examine the consequences on the layout alternative, if this zone had a vertical dip instead of the dip that is currently specified in the site characterization. This has been done by inserting the zone in the basic layout with the trace it has on the ground level, see Figure 5-13.

If zone ZSMEW007A dips vertically, there is a risk that the central area will have to be relocated since the deformation zone could in that case cut straight through the area. The available areas will be approximately the same as in the basic layout, but more passages with transport tunnels would probably have to be excavated through the zone. In other words, the change in dip would have no major effect on the number of deposition holes, but could lead to significant engineering problems due to the hydrogeological properties of zone ZSMEW007A (see Chapter 6).

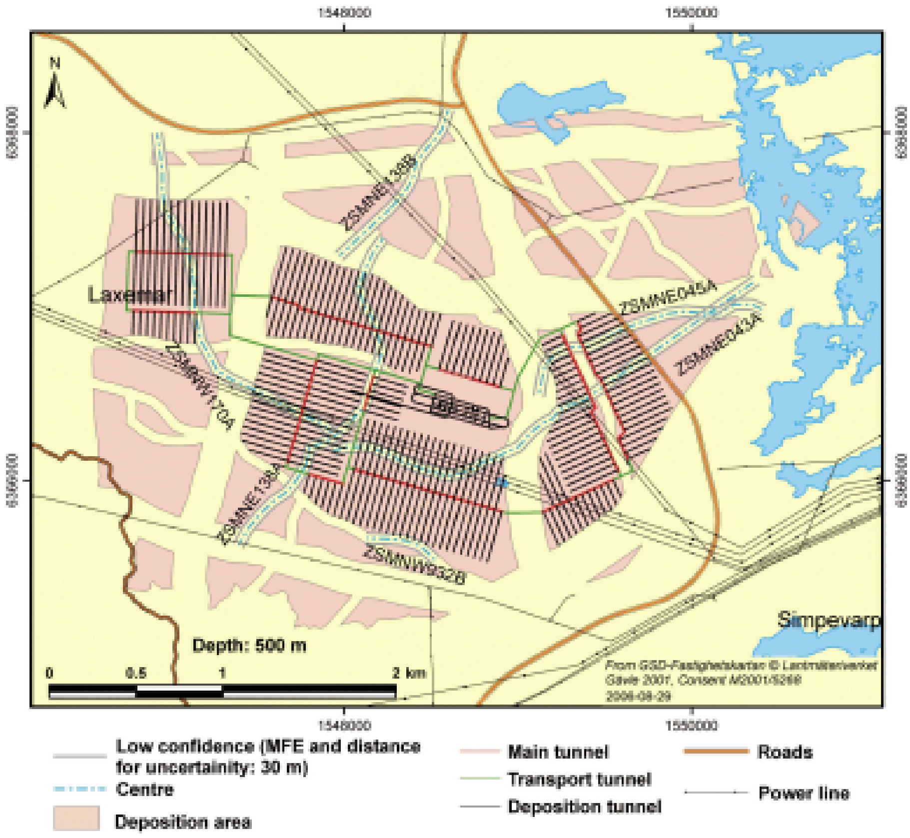


Figure 5-12. *Deformation zones with a low level of confidence inserted for the basic layout Central 500 m. The MFE and margin for uncertainty is set to 30 m.*

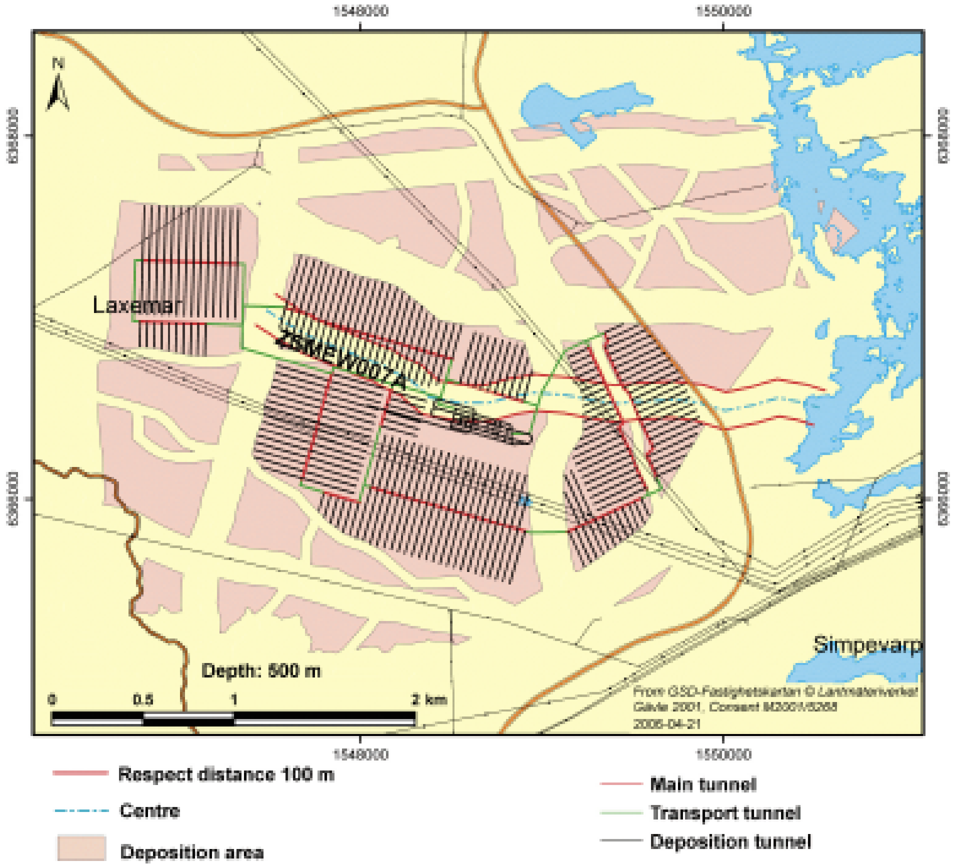


Figure 5-13. *Deformation zone ZSMEW007A as it is interpreted at ground level, inserted here for the layout alternative 500 Central. The respect distance is 100 m.*

Thermal conductivity

As mentioned earlier, the distance between the deposition holes depends to a large extent on the thermal conductivity and the initial temperature in the rock mass. More on this matter is described in Section 4.2, where a sensitivity analysis has been made for these parameters, see Table 5-4.

An analysis is made of how a standard deviation in thermal conductivity from the mean value of 5% affects the distance between deposition holes in the various rock domains. The same analysis was made for the 2.5 percentiles of thermal conductivity. Recorded data on the initial temperature show a spreading in values that varies between 1.0 and 1.5 degrees. In this case, only negative effects have been analysed.

The greater hole distance leads to a decrease in the number of deposition holes by 5–18% in the basic layout depending on the rock domain and subarea concerned.

Table 5-4. Variation in hole distance as a consequence of uncertainties in thermal conductivity and the initial temperature of the rock mass.

Subarea within Laxemar:	Reference distance	Standard deviation 5%	2.5 percentiles	Initial temp variation
South	7.6 m	8.2 m	7.7 m	8.1 m
Central and West, Alt. 1	7.4 m	7.8 m	–	7.8 m
Central and West, Alt. 2	8.1 m	8.7 m	8.5 m	8.6 m
North and East	7.2 m	7.8 m	8.5 m	7.6 m

Loss of deposition holes

The loss of deposition holes has been studied not only in the uncertainty analyses mentioned above but also with respect to a number of different factors, such as elongated fractures/fracture zones, water seepage, potential wedge breakout and the spalling rock phenomenon (see Section 4.4). The results for the 500 m level alternative are presented in Table 5-5. The total loss of deposition holes for the layout work was estimated to 20%.

The loss of deposition holes varies from 17.5 to 22 per cent depending on which parameters are studied. A reasonable distribution of the loss is assumed to be from 15 to 30%, which includes a certain percentage of wedge outfall and/or spalling rock. In the worst case, the loss would thus increase by 10% compared with the 20% that has been applied in the basic layout.

A summary of the entire sensitivity analysis is presented in Table 5-6.

5.4 Discussion and recommendations

Four alternative layouts have been studied: two at a depth of 500 m and two at 600 m. The two alternatives at 500 m indicate the most favourable conditions. They both give smaller loss of deposition holes as well as a considerably smaller volume of excavated rock. Furthermore, the higher loss of deposition holes at 600 m, which could be as much as 40%, means that in reality all the additional holes would have to be used in these alternatives.

Table 5-5. Summary of the analysed loss of deposition holes at the depth of 500 m for continued layout work.

	Loss due to elongated fractures [1]	Loss due to water seepage	Loss due to potential wedge breakout	Loss due to spalling rock	Total loss
Analysed results	16%	1.5–6%	0%	0%	17.5–22%

[1] I.e stochastically determined fractures/fracture zones with radius 50 m < R < 600 m.

Table 5-6. Summary of sensitivity analysis.

Negative change in conditions	Decrease in number of deposition holes	Other possible consequences
Consideration given to the deformation zones with low confidence	approx. 10%	The number of deposition areas increases. The available area decreases. Some of the available additional area has to be used.
Deformation zone ZSMEW007A dips vertically from the ground surface	0%	The zone intersects the central area “Central”, which is divided into two parts
Thermal conductivity deviates by 5% from the mean value or the initial temperature of the repository is 1.5° higher	max. 8%	Some of the available additional areas have to be used.
The hole distance is based on the thermal conductivity’s 2.5 percentile	max. 18%	Some of the available additional areas have to be used.
The hole distance in domain RSMM is based on the thermal conductivity’s mean value from Alt. 2	Approx. 10%	Some of the available additional areas have to be used.
The maximum value from the analysed loss variations is chosen	Approx. 10%	Some of the available additional areas have to be used.

One of the two alternatives at 500 m is located within the central area situated in the south western part (West) of the Laxemar area, whereas the other is situated in the central part (Central). The area in the central part is considered to be more advantageous, primarily with respect to the conditions for the surface facilities, and has therefore been chosen as the basic layout.

A number of negative changes in assumptions for the basic layout, have been studied in an overall sensitivity analysis. The results of the analysis are expressed as a reduction of deposition holes, which could be as much as 18 per cent for the case of reducing the thermal conductivity of the rock.

There would also be a major impact on the appearance of the basic layout if the deformation zone ZSMEW007A were to dip vertically from the ground surface. A major impact on the appearance of the basic layout is also given by considering the deformation zones with a low level of confidence.

6 Identification of passages through deformation zones

6.1 Input data and assumptions

The identification of passages through deformation zones is based on the final layout "basic layout" that was chosen in Chapter 5, i.e. a storage depth of 500 m and with the central area located in the "Central" alternative.

The method of describing the passages through the deformation zones has been divided into the following items:

1. Identification of passages through deterministically chosen deformation zones.
2. Assessment of the length of each passage.
3. Classification of each passage with respect to rock quality.
4. Assessment of anticipated problems for each passage with respect to excavation, rock support and grouting.

As a start, information for Items 1 to 3 has been taken from Chapter 3 and 5 as well as from the SDM v 1.2 /SKB 2006a/. For Item 4, different criteria for levels of difficulty have been produced.

Information on the deformation zones such as rock quality and hydraulic conductivity is either limited or not existing. Despite this lack of information, the various passages have been classified into levels of difficulty. After this evaluation, suggestions are given on how each passage should be dealt with in connection with excavation, rock support and grouting.

The deformation zones, have been classified into levels of difficulty I to IV depending on the rock quality and hydraulic conductivity of the extent and methods of excavation, rock support and grouting depends in a great extent on the degree of difficulty involved.

Deformation zones with a level of I to III are generally described on the basis of empirical experience. Criteria for these zones have been taken from experience literature from the Äspö HRL /SKB 1997a–c/.

Deformation zones with a difficulty level of IV are mainly based on SKB's study of hydraulically conductive zones of Type NE-1 /Chang et al. 2005/.

A description of the levels of difficulty and assessed consequences without measures is presented in Table 6-1.

The selected layout gives a total of 6 passages through deformation zones, see Figure 6-1.

Table 6-1. Description of levels of difficulty and assessed consequences. These are based on SKB's study R-05-25 /Chang el al. 2005/, previous experience from the Äspö HRL /SKB 1997a–c/ and empirical experience.

Level of difficulty	Type description of zone, based on zones in Äspö HRL	Assessed consequences, without measures
I "fair"	Discrete crushed zone, approx. 0.5–2 m wide, with broken surrounding rock (10–20 m). Hydraulic mean conductivity: approx. 2×10^{-6} m/s. RQD = 50–75%, RMR = 50–70. Type description based on a so-called "minor zone" /SKB 1997a–c/.	Possible stability problems with wedges failures. Risk of high water inflow in individual boreholes, which means large grout quantities.
II "medium-difficult"	10–15 m wide zone with a 2–3 m thick core of heavily fractured rock. Hydraulic mean conductivity: approx. 2×10^{-6} m/s. RQD = 50–75%. RMR = 30–60. Type description mainly based on EW-3 /SKB 1997a–c/.	Possible stability problems with tunnel deformations and wedge failures. Partially high water pressure and large water inflows, which means large grout quantities and the need for a modified grouting programme.
III "difficult"	20 to 30 m wide zone containing a 2–5 m wide core of heavily fractured rock. Hydraulic mean conductivity: approx. 2×10^{-5} m/s. RQD = 25–50%, RMR = 25–40. Type description mainly based on NE-3 /SKB 1997a–c/.	Stability problems with unstable tunnel face, tunnel deformations and wedge failures, which leads to reduced/cautious advance per round and extensive rock support. High water pressure and high water inflow in poor quality rock means extensive grouting with special equipment and a specially-designed grouting programme.
IV "very difficult"	30 to 40 m wide zone with a 5 to 10 m, heavily fractured and altered core that may contain clay transformations. Hydraulic conductivity: 0.5–1×10^{-4} m/s in the fractured core and 0.1–0.2×10^{-4} m/s outside the core. RMR = 21–40 in the fractured core (< 21 for clay dykes) and 41–60 in the transitional areas. Type description mainly based on NE-1 /Chang et al. 2005/.	Stability problems with unstable tunnel face, large tunnel deformations, wedge failures and possibly swelling clay. Tunnel excavation and rock support problems can be expected. High water pressure and high water inflow in very poor quality rock means extensive grouting with special equipment and a specially-designed grouting programme. Freezing with a concrete lining may be necessary.

The measured lengths of the passages are based on the width of the zone and the orientation of the passages in relation to the zones. The lengths of the passages through respective deformation zones are shown in Table 6-2.

Table 6-2. Passages through deformation zones.

Passage	Deformation zone	Degree of confidence	Passage length (interval)	Any comments
1	NS059A	High	Approx. 50 m (20–60 m)	
2	NS059A and EW900A	High and high	Approx. 85 m (65–105 m)	Passage at the zones intersection point
3	NW051A	Medium	unknown	Passage made immediately south of zone NW929A
4	NS046A	Medium	unknown	
5	NS046A	Medium	unknown	
6	NW051A	Medium	unknown	
Reserve*	EW007A	High	Approx. 50 m (20–60 m)	The zone has extremely varying properties and extent

* The passage of EW007A will have to be executed if it becomes necessary to use the reserve areas.

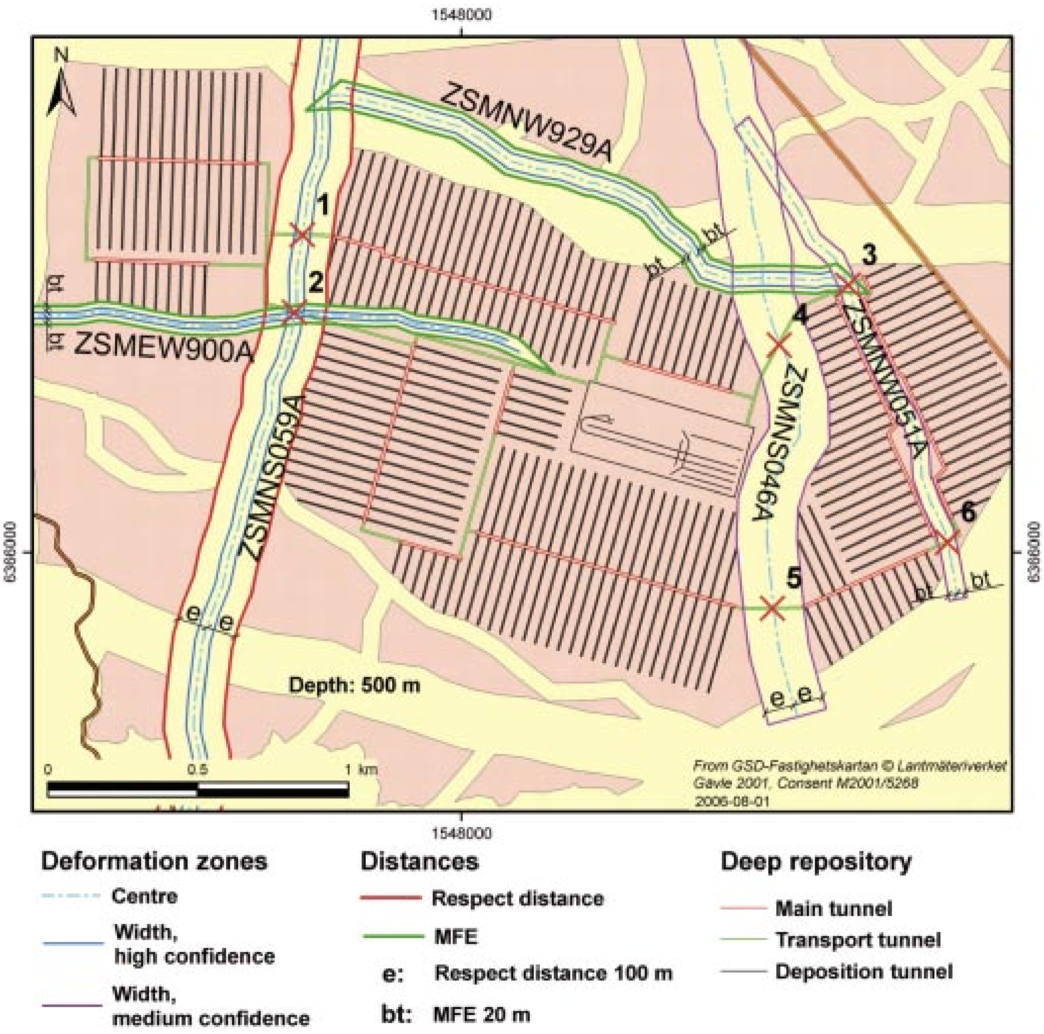


Figure 6-1. *Identified passages through deformation zones for selected layout. The deformation zones have been drawn up with their centre lines and width in accordance with the site description /SKB 2006a/. Respect or structural engineering (construction) distances are shown in Section 3.2.*

Deformation zone NW929A coincides with zone EW007A, see Figure 6-1. The depth level, however, is uncertain, since EW007A has very varied properties and width according to the SDM v 1.2 /SKB 2006a/.

6.2 Execution

6.2.1 Rock quality through the passages

Information on the rock quality in respective passages is based on the corresponding deformation zones compiled in Table 6-3.

Table 6-3. Compilation of the rock quality of passages/deformation zones.

Passage	Deformation zone	Available information on rock quality and conductivity /SKB 2006a/
1	NS059A	Described as a complex zone with indications of a 5–15 m wide core of heavily fractured rock. The zone has a seismic velocity of 3,500 to 3,900 m/s /Lindqvist 2004/. The zone may be influenced by hydraulic conductivity from zone EW007A.
2	NS059A och EW900A	The passage crosses the intersection points of NS059A with a core of heavily fractured rock of 5 to 15 m and at the same time EW900A with a corresponding core of approx. 10 m. Zone EW900A is hydraulically conductive (T = 5.13E–05 m^2/s).
		The seismic velocity through EW900A has with the aid of /Lindqvist 2004/ been interpreted to 2,800–3,600 m/s, i.e. a lower value and probably worse rock quality than in Passage 1.
3	NW051A	Lineament on the ground surface. Otherwise no information. Belongs to Rock Domain A.
4	NS046A	Lineament on the ground surface. Otherwise no information. Belongs to Rock Domain A.
5	NS046A	Lineament on the ground surface. Otherwise no information. Belongs to Rock Domain M.
6	NW051A	Lineament on the ground surface. Otherwise no information. Belongs to Rock Domain M.
Reserve	EW007A	Described as a regional, complex zone with an approximately 10 m wide core of heavily fractured rock. The zone is hydraulically conductive (T = 4.57E–05 m^2/s). The lowest recorded seismic velocity in the zone is approximately 2,800 m/s. The spare passage will probably be located in the western part of the zone.

6.2.2 Assessment of level of difficulty

In general, the information available on the deformation zones is limited, something that make it rather difficult to optain a good classification of the passages according to their level of difficulty.

In the case of Passages 1, 2 and the reserve passage, recorded seismic velocities (V_p) are presented in Table 6-3. With the aid of the empirical equation between V_p and Q /Barton 2002/ the Q value can be calculated:

$$Q_c = 10^{\left(\frac{V_p - 3500}{1000}\right)} \qquad \text{Equation 6-1}$$

where Q_c is the normalised Q value (for definition of Q see Section 9.2):

$$Q_c = Q\frac{\sigma_C}{100} \qquad \text{Equation 6-2}$$

The determination of σ_c (uniaxial compressive strength) for the different rock domains is in SDM v 1.2 /SKB 2006a/.

The elasticity module for the rock mass (Em) can be calculated by using Equation 6-3 /Stille and Nord 1990/:

$$\text{Em} = V_p^2/430 \qquad \text{Equation 6-3}$$

The density and Poisson`s ratio are assumed to be 2.6 t/m^3 and 0.2 respectively.

The results of the equation and information on zone widths and hydraulic conductivity have then been compared with the contents in Table 6-1. The assessed level of difficulty, according to Table 6-4, is therefore weighing between the result and the available information on respective zones.

For Passages 3 to 6, the lowest level of difficulty has been assumed since there is no information available on the zones, only length and orientations /Wahlgren et al. 2005/.

Table 6-4. Assessment of level of difficulty.

Passage	Assessed level of difficulty	Motivation behind assessment of level of difficulty
1	II "medium difficult"	The passage passes through zone NS059A which is anticipated to have a 5–15 m wide core of heavily fractured rock. A relatively high level of hydraulic conductivity is expected because the zone is elongated and cuts through several other zones with a high level of hydraulic conductivity. Q = 0.5–1.3 and E_m = 28–35 Gpa, based on V_p
2	II "difficult"	Passage through the intersection point of zones NS059A and EW900A (this should be avoided by tunnel probing). The zones have a heavily fractured core of 5–15 m that influences the passage over a distance of 10–20 m. However, no clay alterations have been found. The passage is affected through EW900A, which is primarily characterised as a hydraulically conductive zone. However, available information indicates that the rock mechanical properties are not difficult to handle. For EW900A, the following properties are obtained: Q = 0.15–0.8 and E_m = 18–30 Gpa, based on V_p.
3	I "fair"	The passage is considered possible without any major stability problems, even if a certain level of high hydraulic conductivity can be expected. The zone is considered to be a "minor zone" with RMR = 64 ± 10 (Q = 1–5) according to SDM v 1.2 and Rock domain A. However, the passage should be moved by approximately ten metres south, so that it falls outside the structural engineering distance of zone NW929A.
4	I "fair"	The passage is considered possible without any major stability problems, even if a certain level of high hydraulic conductivity can be expected. NS046A is expected to have a sufficiently high level of rock quality, despite a long elongation. The zone is considered to be a "minor zone" with RMR = 64 ± 10 (Q = 1–5) according to SDM v 1.2 and Rock domain A.
5	I "fair"	The passage is considered possible without any major stability problems, even if a certain level of high hydraulic conductivity can be expected. NS046A is expected to have a sufficiently high level of rock quality, despite a long elongation. The zone is considered to be a "minor zone" with RMR = 72 ± 10 (Q = 9–13) according to SDM v 1.2 and Rock domain M.
6	I "fair"	The passage is considered possible without any major stability problems, even if a certain level of high hydraulic conductivity can be expected. NW051A is not expected to be a major zone within the area and is considered to have a sufficiently high level of rock quality. The zone is considered to be a "minor zone" with RMR = 72 ± 10 (Q = 9–13) according to SDM v 1.2 and Rock domain M.
Reserve-passage	III "difficult"	The passage is affected through EW007A, which is primarily characterised as a hydraulically conductive zone with varying extent and properties. However, available information indicates that the rock mechanical properties that are not extremely difficult to handle. The core exhibits limited cracking at a boring length of 10 m, but with no clay transformations. Lowest Q = 0.15 and E_m = 18 Gpa, based on V_p.

6.3 Results

The description of how the passages are considered, i.e. the proposed methods of excavation, rock support and grouting, are divided into four sections, based on assessed level of difficulty and rock domain:

1. Passage 2 and Reserve passage, Level of Difficulty III.
2. Passages 1, Level of Difficulty II.
3. Passages 3 and 4, Level of Difficulty I and Rock Domain A.
4. Passages 5 and 6, Level of Difficulty I and Rock Domain M.

Excavation and rock support

The method of excavation is based on the difficulties recorded in Table 6-4. For the passages, the permanent rock support has been designed on the basis of Figure 6-2. The transport tunnel span is 7 m.

For Passages 1, 2 and the reserve passage, the value of the rock quality has been based on the specified Q values in Table 6-4 and the following equation /Stille and Nord 1990/:

$$RMR = 9 \ln Q + 44 \qquad \text{Equation 6-4}$$

For Passages 3 to 6, the Q value has been taken from SDM v 1.2 /SKB 2006a/ and the specified rock domain, see the values in Table 6-4.

Grouting

For Passages 1, 2 and the reserve passage, a specific grouting programme is recommended in which long investigation holes and gained experience from SKB's study /Chang et al. 2005/ on how heavily hydraulically conductive zones at a great depth are treated. For Passages 3 to 6, grouting is effected on the basis of the normal grouting programme, possibly with a certain modification of the grout and pressure, since large quantities of grouting can be expected in individual grout holes.

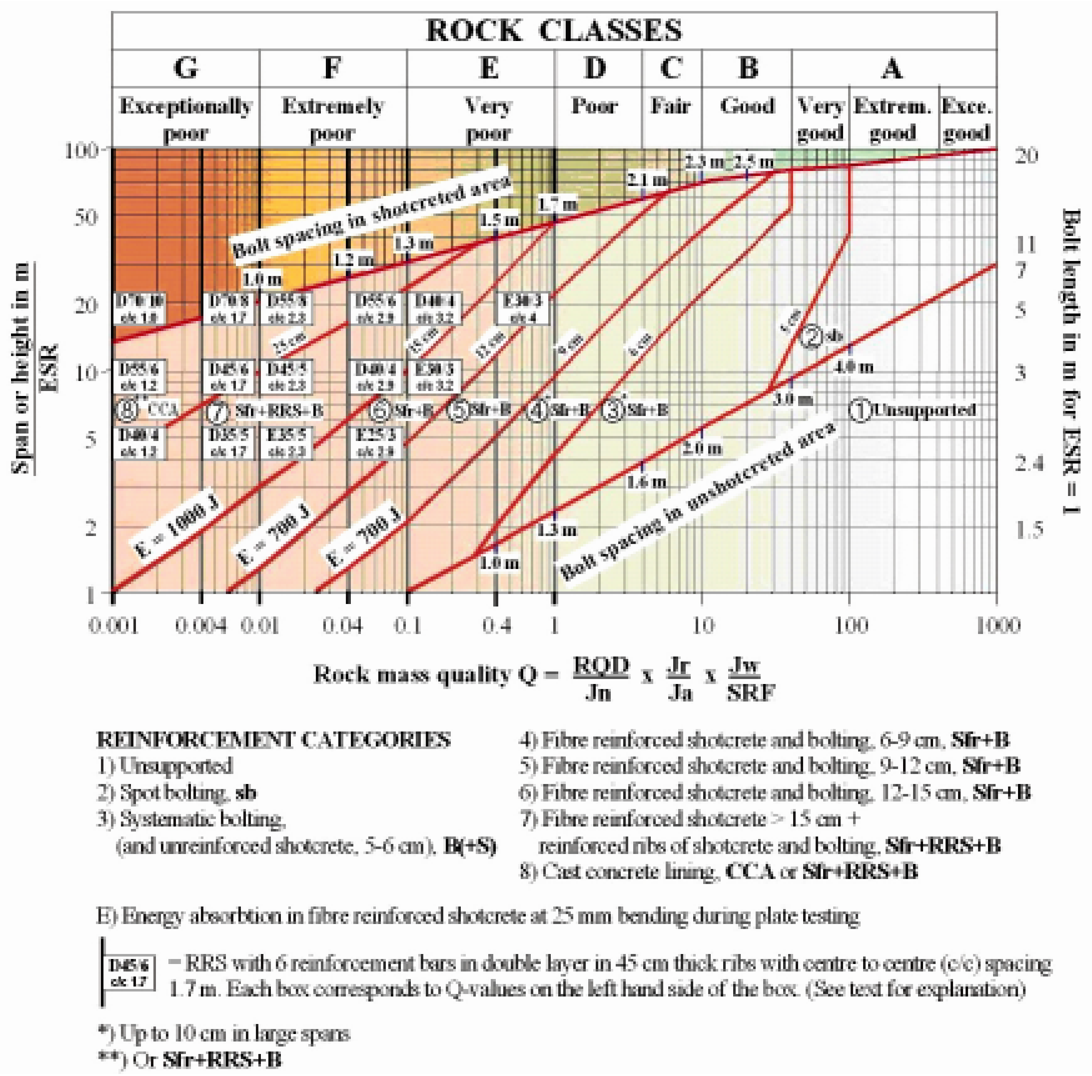


Figure 6-2. Design of rock support, after /Grimstad et al. 2002/.

In order to get an idea of the level of difficulty involved in the grouting, the expected water seepage into the tunnel, without grouting measures, can be calculated as follows /Alberts and Gustavsson 1983/:

$$q = \frac{2 \cdot \pi \cdot K \cdot h}{\ln\left[\frac{2 \cdot h}{r}\right] + \xi} \qquad \text{Equation 6-5}$$

where q is the inflow into the tunnel (m^3/s,m), K is the hydraulic conductivity for the zone (m/s), h is the groundwater pressure (m), r is the equivalent tunnel radius (m) and ξ is the skin factor (–).

The groundwater pressure has been assumed at 500 m, the equivalent tunnel radius is approximately 3.9 m and the skin factor that varies between 0 and 10 /SKB 1997b/, has been set at 5.

6.3.1 Passage 2 and reserve passage

The level of difficulty has been put at III, see Table 6-4.

Excavation:

Extensive pre-probing, i.e. long investigation holes, is conducted on the zones from the tunnel and wall niches, 30–50 m from the core of the zone. Excavation is carried out with reduced advance per round (shorter and possibly split rounds). Probing is carried out by registering bore parameters during drilling (MWD, measurements while drilling) in order to get a better understanding of the susceptibility to fractures and hardness in the deformation zone. After this, hydraulic tests are performed in the probing holes in order to assess the hydraulic conductivity and water pressure of the zone.

The length of the excavation rounds is reduced to 2–3 m depending on the actual site preconditions.

During excavation, pre-rock support through the core and stabilization of the tunnel face with shotcrete, may be required. Pre-rock support may be applied by means of spiling bolts from abutment to abutment.

The distance from tunnel face to permanent rock support is assessed to be approximately 2–4 m, after which the excavation can continue. Temporary support at the tunnel face will probably be necessary from the working environment point of view. This rock support can be considered as permanent rock support as long the requirements for materials and performances are adequate.

A complete programme for the measurement of tunnel deformations will be available before excavation through the zone is started.

Rock support:

The RMR value in the deformation zone has been estimated to be 25–40. In the case of the core of the zone, the RMR value has been assessed at 25–30, which gives a Q index of 0.1–0.2. In the transition zone, the RMR value has been estimated to 35–40, which corresponds to a Q index of 0.35–0.6.

According to the above assumed Q values in the deformation zone, the rock support required in the transport tunnel would be as follows:

Core of zone

Fibre-reinforced shotcrete, 120–140 mm and bolting spacing with c/c 1.3–1.4 m, bolt length = 3.0 m.

Transition zone

Fibre-reinforced shotcrete, 90–100 mm and bolting spacing with c/c 1.5–1.6 m, bolt length = 3.0 m.

In the case of poorer rock quality, Q < 0.1 (RMR < 25) and depending on the results of the deformation measurement, it could be necessary to introduce supplementary rock support in the form of cast in concrete support or shotcrete arches of reinforced concrete.

Grouting:

The mean transmissivity is 5.13E–05 m^2/s (geological thickness 20 m) and 4.57E–05 m^2/s (geological thickness 50 m) for Passage 2 and reserve passage respectively, according to SDM v 1.2.

The water seepage without grouting measures is estimated to be in the order of approximately 0.8 and 0.3×10^{-3} m^3/s, m, i.e. approximately 3,900 and 820 l/min, zon, for Passage 2 and reserve passage respectively.

Extensive grouting is expected with one or more specially designed grouting programmes, including drilling, probing and the checking of each grouting curtain. The grouting programme will be based on the results of the pre-probing.

The first long-hole probing shall be carried out in connection with the excavation to be followed by probing and measurements in each grouting curtain. Special equipment for drilling and probing may be necessary as a result of the high water pressures and the high level of hydraulic conductivity. In SKB's study /Chang et al. 2005/ the use of specialist contractors with special equipment such as blow-out-preventers is recommended.

It is proposed that grouting should start by means of so-called long-hole grouting (coarse grouting) at a grouting pressure with a high safety factor and grout with stable mortar properties. Grouting at a high yield point shall continue until the conductivity of the zone has been lowered by a power of ten.

After this, grouting curtains will be made with a short length and a varying extent, after several rounds of grouting and with a large overlap. The grouting pressure and grout properties will be decided after each round of grouting on a basis of a predetermined programme.

There will be contingency plans for different kinds of problems such as additives (cement accelerators), chemical grouting agents and certain types of special equipment such as blow-out-preventers or other types of sleeves (long, hydraulic, double sleeves) as well as drilling equipment for high water flows and pressures.

6.3.2 Passage 1

The level of difficulty has been set at II, see Table 6-4.

Excavation:

Extended pre-probing will be made in the zone through long investigation holes and reduced advance rate per round. Probing shall be carried out approximately 30 m from the core of the zone. The length of the excavation rounds will be reduced to 2–4 m.

The distance from the tunnel face to the permanent rock support will be assessed as excavation progresses but it can be assumed to be approximately 4–5 m. For working safety reasons temporary support at the tunnel face will probably be needed. This rock support can be counted as permanent rock support provided the stipulated demands for materials and performance are adequate.

Contingency plans and a programme for the measurement of tunnel deformations will be designed up before the start of zone excavation.

Rock support:

The RMR value in the deformation zone is 30–60. For the core of the zone, the RMR value has been estimated to be at 30–40, which gives a Q index of 0.2–0.6. In the transition zone, the RMR value is considered to be 40–50, which corresponds to a Q index of 0.6–1.9.

According to the Q values calculated above, in the deformation zone the required rock support in the transport tunnel will be as follows:

Core of zone

Fibre-reinforced shotcrete, 90–120 mm and pattern bolting with c/c 1.4–1.6 m, bolt length = 3.0 m.

Transition zone

Fibre-reinforced shotcrete, 50–90 mm and pattern bolting with c/c 1.6–1.8 m, bolt length = 3.0 m.

Grouting:

No data is available concerning the mean transmissivity in the SDM v 1.2.

The water seepage without grouting measures is estimated to be in the order of approximately 0.6×10^{-3} m^3/s, m, i.e. approx. 1,790 litres/min, zon, for a hydraulic conductivity corresponding to Table 6-1 and passage length to Table 6-2, for the level of difficulty in question.

Grouting will mainly be performed in accordance with the basic programme (drilling, probing, curtain geometry), even if several rounds of grouting can be expected as well as a considerable overlap between the grout curtains. A certain modification of the grouting pressure and grout is also likely to occur, where grouting pressure and grout properties are determined after each probing and round of grouting and on the basis of a fully designed grouting programme. Furthermore, there will be contingency plans for different kinds of problems such as additives (cement accelerators), chemical grouting agents and certain types of special equipment. Special equipment can be necessary to use such as different types of sleeves (long, hydraulic, double sleeves) as well as drilling equipment for high water flows and pressures.

6.3.3 Passages 3 and 4

The level of difficulty has been estimated at I, see Table 6-4. Passage 3 should be moved a few metres in relation to the basic layout so that the construction distance increases in relation to the adjacent zone ZSMNW929A.

Excavation:

No special measures are necessary. Depending on the results of the normal probing, in connection with the grouting drilling the excavation round length could be possibly reduced.

During excavation, selective bolting may be necessary in order to prevent the outfall of rock wedges.

Rock support:

The extent of bolt and shotcrete rock support will be based on the results of tunnel mapping and normal rock support programmes. In Table 6-4*,* an expected Q index of 1–5 is specified ("minor deformation zones" in SDM v 1.2), which gives a non-reinforced shotcrete thickness of 40–100 mm and bolting spacing with c/c 1.8–2.2 m and a bolt length of = 3.0 m, as per Figure 6-2.

Grouting:

No data is available concerning the mean transmissivity in the SDM v 1.2.

The water seepage without grouting measures is estimated to be in the order of approximately 0.6×10^{-3} m^3/s, m, i.e. approx. 36 litres/min, m at a hydraulic conductivity equivalent to that in Table 6-1 for the level of difficulty in question.

Grouting will mainly be performed in accordance with the basic programme (drilling, probing, curtain geometry, pressure, grout properties, checking). An anticipated major consumption of grout in individual holes will be controlled/limited by combining suitable grouting pressures and grout properties.

Furthermore, there will be contingency plans for high water pressure and a high level of conductivity in individual grout holes, such as different types of sleeves (long, hydraulic, double sleeves).

6.3.4 Passages 5 and 6

No special measures are needed. Depending on the results of the normal probing, the length of excavation rounds may have to be reduced.

During excavation, selective (spot) bolting may be necessary in order to prevent the outfall of rock wedges.

Rock support:

The extent of bolt and shotcrete rock support will be based on the results of tunnel mapping and normal rock support programmes. Table 6-4 specifies an expected Q index of 9–13 ("minor deformation zones" in SDM v 1.2), which gives a bolting spacing of c/c 2.2–2.5 m with a bolt length of 3.0 m or only a selective bolting as per Figure 6-2.

Grouting:

No data is available concerning the mean transmissivity in the SDM v 1.2.

The water seepage without grouting measures is estimated to be in the order of approximately 0.6×10^{-3} m^3/s, m, i.e. approx. 36 litres/min, m at a hydraulic conductivity equivalent to that in Table 6-1 for the level of difficulty in question.

Grouting will be carried out in accordance with a normal grouting programme (drilling, probing, curtain geometry, pressure, grout properties, checking). An anticipated major consumption of grout in individual holes will be controlled/limited by combining suitable grouting pressures and grout properties.

Furthermore, there will be contingency plans for high water pressure and a high level of conductivity in individual grout holes, such as different types of sleeves (long, hydraulic, double sleeves).

7 Seepage and hydrogeological situation around the repository

The construction of the deep repository will result in a disturbance of the groundwater pressure levels of the ambient rock mass as an effect of seepage into the repository. These disturbances and the inflow will remain throughout the entire period of time any part of the repository is held open. The size and intensity of the disturbances will change as different parts of the repository are active.

The seepage and change in pressure levels are likely to cause a lowering of groundwater levels around the repository which could have a negative effect on nearby wells and surface waters in the area. There is also a risk that deeper situated, more saline, groundwater will reach the repository as the pressure levels change. If the salinity of the groundwater entering the repository increases significantly, the buffer around the canister enclosing the nuclear waste could be affected.

To limit the groundwater inflow in the repository, grouting will be carried out. The amount of grouting depends on the seepage magnitude.

To make it possible to predict the seepage magnitude to the repository and the effects on the groundwater levels and the saline groundwater, both analytical and numerical calculations have been carried out. Predictions have been made with respect to different steps of the construction of the repository as well as for different levels of grouting.

The parameters that should be predicted are; seepage to the repository, the size of the influenced area, the size of drawdown in groundwater levels and the risk for upconing of saline groundwater. The analytical calculations are based on general principles/assumptions, and equations given in the UDP /SKB 2004a/.

The numerical and analytical calculations were done as two separate projects. The numerical calculations have been thoroughly presented in a separate report and therefore only conclusive results from those calculations are presented here. For detailed information on input data and execution of the numerical calculations the reader is referred to /Svensson 2006/.

During the process of solving the analytical calculations several problems and uncertainties were encountered for some of the calculation methods. Due to these problems some of the results could not be regarded as reliable and are therefore not presented in this report. The concerned calculations are those regarding radius of influence (Equation 7-5), drawdown (Equation 7-6) and upconing of saline groundwater (Equation 7-7).

7.1 Input data and assumptions

To illustrate the change in seepage magnitude and the effect on the surrounding groundwater level during the construction of the repository, five construction steps has been chosen where different parts of the repository are open. Each calculation has been carried out for each of the five construction steps. The parts of the repository that are kept open during each step and the duration of the steps can bee seen in Figure 7-1.

Step A is the initial step where only the central area is opened. This area is then kept open throughout the entire time the repository is open. Therefore the following steps are named AB-AE. For one of the analytical calculation methods however, it was not possible to include the central area in the calculation, these steps are only referred as step B–E for this method.

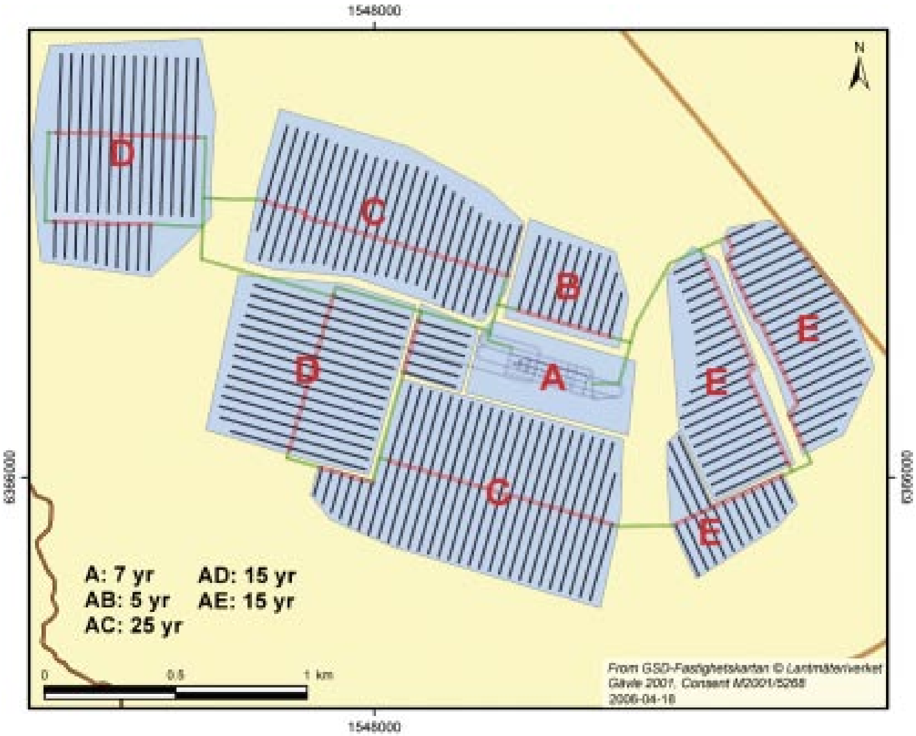


Figure 7-1. *Location and duration of the different construction steps that were used for the calculations.*

Apart from the different construction steps, the level of grouting is also taken into account. Three different levels of grouting have been used, one of which is the natural case i.e. where no grouting has been done. The other two levels are represented as a hydraulic conductivity (K) of the grouted rock mass surrounding the repository.

The three levels of grouting that were used are:

- No grouting
- $K = 10^{-7}$ m/s
- $K = 10^{-9}$ m/s

7.2 Execution

As mentioned before, the numerical calculations are presented in a separate report /Svensson 2006/. Therefore this report will only deal with the execution of the analytical calculations.

For all analytical calculation methods the used equations and basic assumptions are presented. For some of the methods, however, the results are not included as these were not considered to be reliable. This concerns the methods regarding radius of influence (Equation 7-5), drawdown (Equation 7-6) and upconing of saline groundwater (Equation 7-7). For these methods an explanation for the unreliable results is included.

Monte Carlo simulations

As the input data are uncertain for several of the parameters, it is of great interest to be able to vary them for the calculations. This can be done by use of so called Monte Carlo simulations. The idea is that you allow a computer to perform a large number of calculations of your equation (in our case 10,000) where the parameters varies independently for each calculation. The variation for each of the parameters is limited by a chosen distribution function.

7.2.1 Seepage

Two separate methods were used for the predictions of seepage. The assumptions of the two methods differ significantly, but both methods assume steady state conditions, i.e. that the groundwater levels and the seepage are balancing each other.

Seepage, Method 1 (Equation 7-1)

For this method, the repository is represented by a tunnel at a given depth below the groundwater level. The seepage is calculated per meter of tunnel. The grouting around the repository is represented by a zone around the tunnel having a lower hydraulic conductivity.

The central area was not included in this calculation as it was difficult to apply the method to that part of the repository with its ramps, halls and shafts. Thus the calculated seepage only includes deposition tunnels, transport tunnels and main tunnels. Separate calculations were also performed where transport tunnels are intersected by deformation zones.

To get the seepage to the repository (Q_s) the q_s were multiplied by the entire length of the tunnels:

$$\mathbf{Q_s = q_s L} \qquad \text{Equation 7-2}$$

Input data

Table 7-1 shows the values used for the calculations of seepage using Equation 7-1. The representative hydraulic conductivity of rock mass was calculated from data in the preliminary site description of Laxemar /SKB 2006a/. To obtain a representative value, semi-correlated conductivity data corresponding to 100 m blocks were weighted between the A (41%) and M (59%) rock domains.

The representative radii that were used in the calculation of the different tunnel types together with their corresponding lengths for each construction step can be found in Table 7-2.

$$q_s = \frac{2\pi K_b d}{\ln\left[\frac{2d}{r_w}\right] + \left[\frac{K_b}{K_t} - 1\right]\ln\left[1 + \frac{m}{r_w}\right] + \sigma}$$

$$\sigma = K_b / K_t \; \xi$$

Equation 7-1

d = depth of the centre of the tunnels below groundwater table (m)

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

K_t = representative hydraulic conductivity of grouting (m/s)

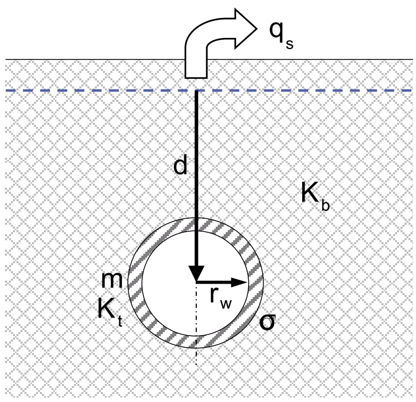
m = thickness of grouting (m)

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

r_w = representative tunnel radius (m)

ξ = natural skin factor (dimensionless)

σ = skin factor inside grouting (dimensionless)



A number of tunnel passages through deformation zones were identified and they are presented in Chapter 6, see Figure 6-1. Since these passages are likely to have much higher hydraulic conductivities than the rock mass, they contribute considerably to the seepage to the repository. Because of this, separate calculations of seepage were carried out for each passage. The values for hydraulic conductivity of the deformation zones are presented in Table 7-3 and are the same as in Chapter 6.

Variation of data (Equation 7-1)

Depending on the nature of the input data, several different types of distributions can be used for data variation. For the grouting thickness (m) and the natural skin factor (ξ) a uniform distribution were applied using the values from Table 7-1. A normal distribution was assumed for the logarithm of the hydraulic conductivity of the rock mass (K_b, Table 7-1). The width of the tunnel passages through deformation zones (L) was given a triangular distribution based on the most likely value and the interval given in Table 7-3.

Examples of the distributions that were used for the calculations are shown in Figure 7-2.

Table 7-1. Values used for the parameters in Equation 7-1.

Parameter	Value	Explanation
d	500 m	Distance from repository to undisturbed groundwater level.
K_b*	log K = –8.67* Std: 0.86	From /SKB 2006a/. For K_b of the passages of deformation zones, see Table 7-3.
K_t	Varies according to the levels of grouting as described earlier.	From /SKB 2004a/
m	3–5 m	Based on earlier experience
r_w	2.8–4.6 m	Representative tunnel radius for each tunnel type.
ξ	3–7	/Dalmalm 2001/

* To get a representative value, semi-correlated K_z data corresponding to 100 m blocks were weighted between the A (41%) and M (59%) rock domains.

Table 7-2. The values used as representative tunnel radii for different tunnel types and the tunnel lengths for each construction step. The central area is not included.

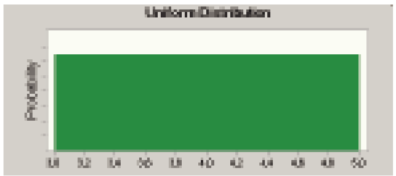
Tunnel type	Radius, r_w [m]	Tunnel length, L [m] AB	AC	AD	AE
Deposition tunnel	2.8	2,720	24,410	20,110	13,395
Transport tunnel	3.8	655	3,315	3,300	2,380
Main tunnel	4.6	380	4,085	2,970	3,755

Table 7-3. Values used for calculations of seepage at the passages of tunnels through deformation zones using Equation 7-1. The values come from Chapter 6 and /SKB 2006a/.

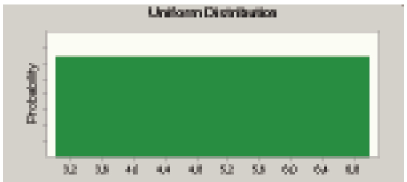
Passage of zone	Length of passage, L [m] Most likely value	Min–Max	Conductivity, log K_b
NS059A (1)	50	20–60	–5.70 (std: 0.41)
NS059A och EW900A (2)	85	65–105	–5.59 (std: 0.41)*
NW051A (3)	10	0–20	–5.70 (std: 0.41)
NS046A (4)	10	0–20	–5.70 (std: 0.41)
NS046A (5)	10	0–20	–5.70 (std: 0.41)
NW051A (6)	10	0–20	–5.70 (std: 0.41)

* The log K_b value is taken from /SKB 2006a/ and is valid for EW900A. It represents a more conservative value than –5.70.

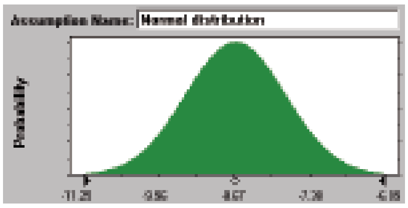
m: 3-5 m



ξ : 3-7



Kb: log K -8.67, std 0.86



Length, L:
Example passage 1: 20 – 60 m, most likely value 50 m

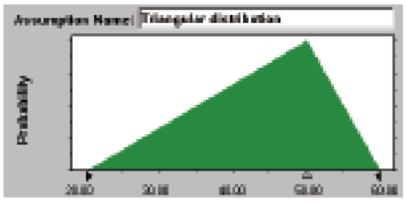


Figure 7-2. *Variation of parameters for Equation 7-1.*

Seepage, Method 2 (Equation 7-3)

This method has a rather different approach to the problem compared to method 1 (Equation 7-1).The repository is here represented by a large diameter well instead of a long tunnel. The grouting around the repository is calculated in the same way as for method 1, i.e. as a zone of lower hydraulic conductivity around the repository. As for method 1, it is assumed that the system has reached steady state.

D = distance to bottom of repository from groundwater table (m)

$\mathbf{h_0}$ = undisturbed groundwater level (m)

$\mathbf{h_w}$ = groundwater level inside repository (m)

$\mathbf{K_b}$ = representative hydraulic conductivity of rock mass (m/s)

$\mathbf{K_t}$ = representative hydraulic conductivity of grouting (m/s)

m = thickness of grouting (m)

$\mathbf{Q_s}$ = seepage under steady-state conditions (m^3/s)

$\mathbf{r_w}$ = representative radius of deep repository (m)

$\mathbf{R_0}$ = radius of influence (m)

$\mathbf{T_b}$ = representative transmissivity of rock mass (m^2/s)

$\mathbf{\Delta_s}$ = drawdown (m)

ξ = natural skinfactor (dimensionless)

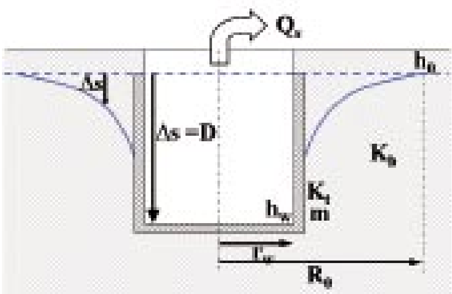
σ = skinfactor inside grouting (dimensionless)

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

$$T_b = K_b\, h_{avg} = K_b\,(h_0 + h_w)/2$$

$$\sigma = K_b / K_t\, \xi$$

Equation 7-3



The value of R_0 was adjusted until a balance was reached between the Q_s calculated using Equation 7-3 and Q_s calculated using Equation 7-4.

$$\mathbf{Q_s = \pi R_0^2 W} \qquad \text{Equation 7-4}$$

where *W* is groundwater recharge. Considering the depth of repository a groundwater recharge of 5 mm/yr was assumed.

Input data

All input data for Equation 7-3 are shown in Table 7-4. The value of $\mathbf{r_w}$ varies for each construction step and is based on the radii presented in Figure 7-3 to Figure 7-7.

Variation of data (Equation 7-3)

Where the parameters used for Equation 7-3 coincide with the parameters for Equation 7-1 the variations are done in the same way.

The representative radius of the repository was given a uniform distribution with an interval of ±50 m from the radii shown in Figure 7-3 to Figure 7-7.

After adjustments of Q_s, compared to the assumed groundwater recharge (Equation 7-4), R_0 was given a value of 600 m for the calculations using the grouting level K = 10^{-9} m/s and 800–900 m for the calculations using the grouting levels, no grouting, and K = 10^{-7} m/s.

7.2.2 Radius of influence

This is the first calculation where the results were assumed not to be reliable and therefore no results are presented. The reasons why the results were thought unreliable are described under the headline "*Uncertainty*".

The basic assumptions are the same as for Equation 7-3 where the repository is represented by a large diameter well. It is assumed that the system has reached steady state.

$\mathbf{h_0}$ = undisturbed groundwater level (m)
$\mathbf{h_w}$ = groundwater level inside repository (m)
$\mathbf{K_b}$ = representative hydraulic conductivity of rock mass (m/s)
$\mathbf{Q_s}$ = seepage under steady-state conditions (m³/s)
$\mathbf{r_w}$ = representative radius of deep repository (m)
$\mathbf{R_0}$ = radius of influence (m)
$\mathbf{T_b}$ = representative transmissivity of rock mass (m²/s)
$\Delta\mathbf{s}$ = drawdown (m)

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

$$T_b = K_b h_{avg} = K_b (h_0 + h_w)/2$$

Equation 7-5

Table 7-4. The used numbers for the parameters in Equation 7-3.

Parameter	Used value	Explanation
$\mathbf{h_0}$	500 m	Distance from bottom of repository to undisturbed groundwater level.
$\mathbf{h_w}$	0 m	Groundwater level inside repository. It is assumed that the drawdown within the repository is 500 m
$\mathbf{\Delta s}$	500 m	It is assumed that the drawdown within the repository is 500 m
$\mathbf{K_b}$	log K = –8.67 (std = 0.86)	From /SKB 2006a/. For K_b of the passages of deformation zones, see Table 7-3.
$\mathbf{K_t}$	Varies according to the levels of grouting as described earlier.	From /SKB 2004a/
m	3–5 m	Based on earlier experience
ξ	3–7	/Dalmalm 2001/
$\mathbf{r_w}$	300–750 m	Varies depending on construction step according to Figure 7-3–Figure 7-7.
$\mathbf{R_0}$	Initially set to 2,500 m	Based on an assumed groundwater recharge of 5 mm/yr.
W	0.005 m/yr	

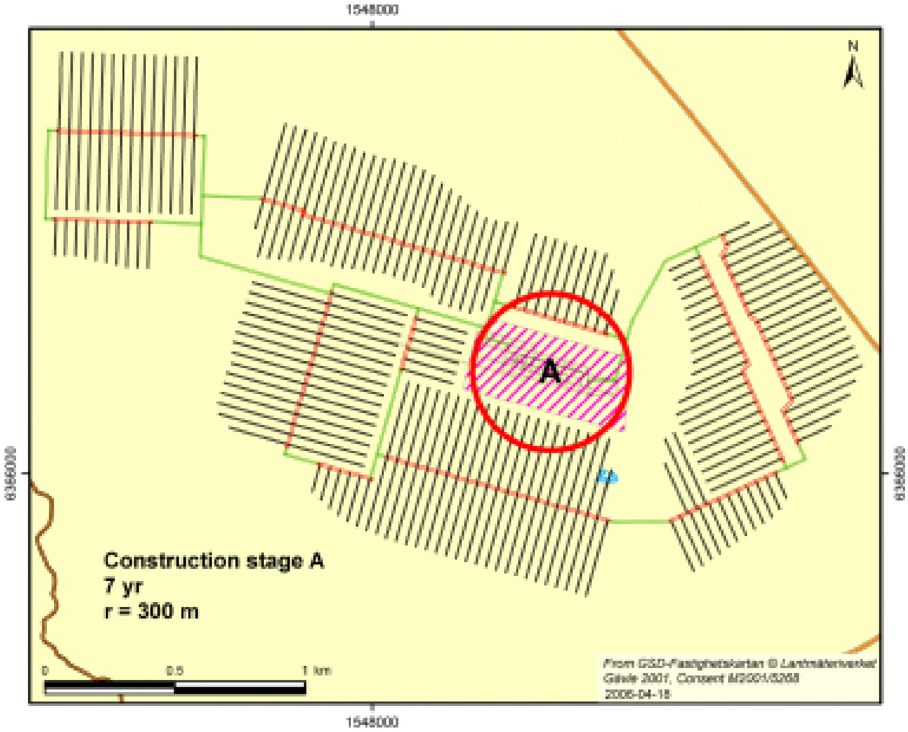


Figure 7-3. Representative radius for construction step A.

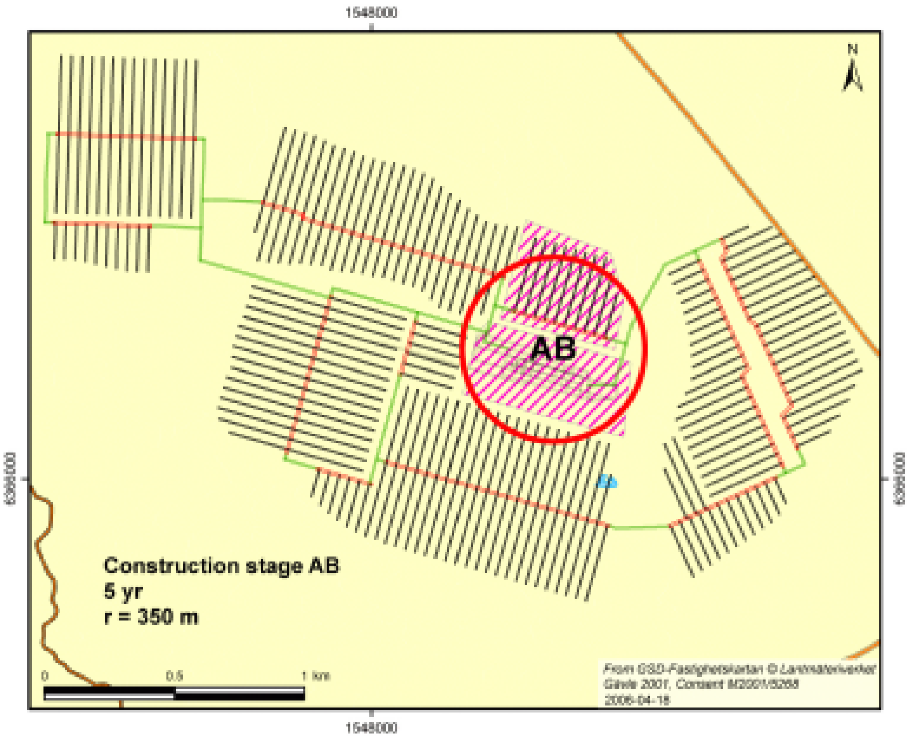


Figure 7-4. Representative radius for construction step AB.

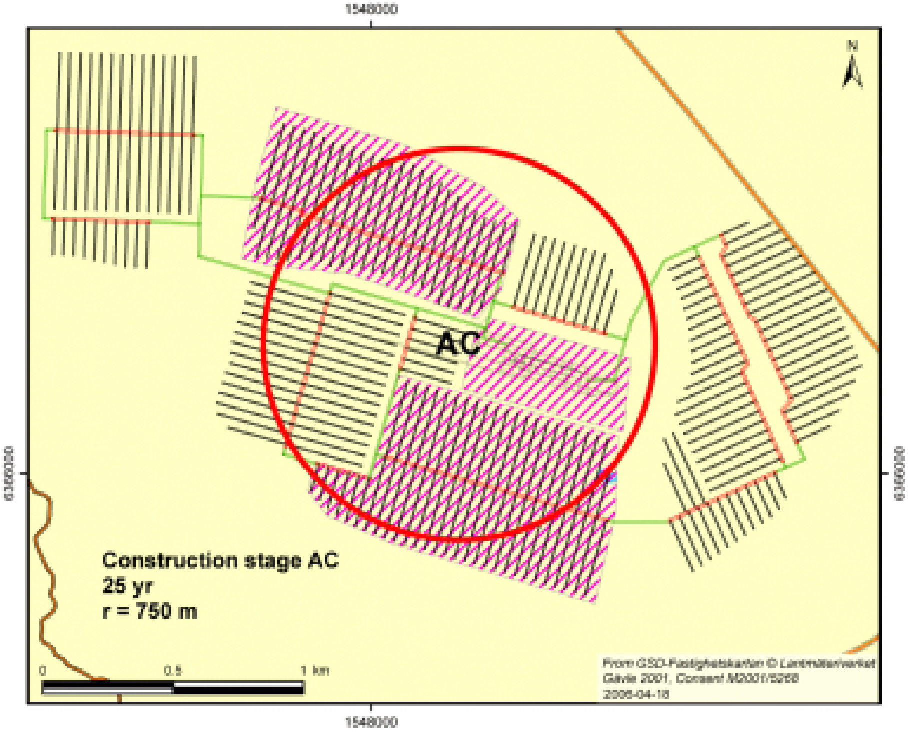


Figure 7-5. Representative radius for construction step AC.

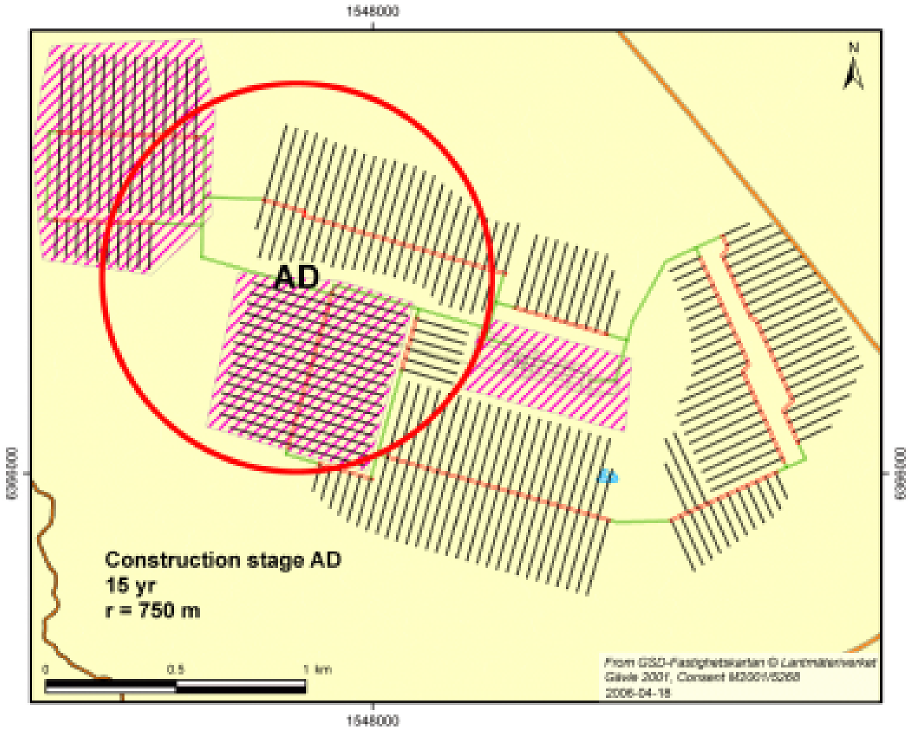


Figure 7-6. Representative radius for construction step AD.

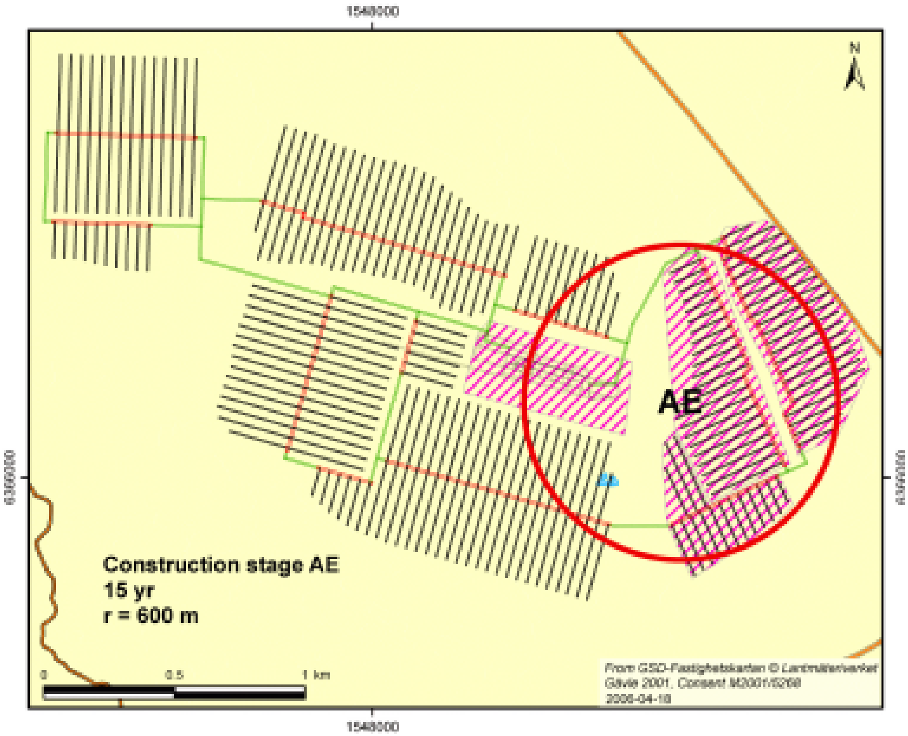


Figure 7-7. *Representative radius for construction step AE.*

Uncertainty

In general the calculation method is most suitable for single calculations with well defined parameters. The exponential function with dependent parameters make the method less suitable for Monte Carlo simulations, something that will be further explained.

The equation consists of an exponential function that mainly depends on transmissivity and seepage. The seepage is not known, so there are two ways to estimate its value:

1) Use the result from Equation 7-1 and Equation 7-3 to define possible intervals for the seepage values.
2) Assume that Q_s is a function of T_b (like $Q(K) = 1\times10^8\times K$) according to results from Equation 7-1 and Equation 7-3

Alternative 1 means that we get a biased result as the seepage values in the denominator is based on one transmissivity and another transmissivity might be used in the nominator using Monte Carlo simulation. As Monte Carlo calculations are performed within certain intervals, difficulties occur when e.g. Q_s is low and T_b is high. The exponent grows fast and we get unrealistic values on R_0.

Alternative 2 means that we will get a very high value in the exponent's denominator, which implies that the exponent will fall against the limit 0. That implies that R_0 will equals r_w, which is not realistic.

7.2.3 Drawdown

Similar to the calculation method for radius of influence, the method for calculating drawdown did not give reliable results. Thus, no results are presented for this method either. A possible explanation for the problems can be found under "*Uncertainty*".

This calculation method is typically used to estimate the resulting drawdown at ground level as a result of pumping groundwater from a well. The drawdown can be calculated at any distance from the well and for any given time after the pumping was started. Thus, this calculation is transient, i.e. different times can be used together with different seepages calculated with Equation 7-1 and Equation 7-3.

The method is based on the use of a so called well function. The well function itself is calculated in advance and can be found in tables /Fetter 2001/. Based on calculations of chosen parameters the tabulated value is then in return used to calculate the drawdown. This procedure makes it impossible to run Monte Carlo simulations and so instead a "most likely" case was used for these calculations.

h_0 = undisturbed groundwater level (m)

h_1 = groundwater level at time and distance for calculation (m)

K_v = vertical hydraulic conductivity for rock mass and grouting (m/s)

K_h = horizontal hydraulic conductivity for rock mass and grouting (m/s)

Q = seepage (m^3/s)

r = distance from repository (m)

t = time (s)

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

Δs = drawdown (m)

S = storativity (dimensionless)

Sy = specific yield (dimensionless)

W = wellfunction

$$\Delta s(t) = h_1 - h_0 = \frac{Q}{4\pi T_b} W\left[u_A, u_b, \Gamma\right]$$

$$\Gamma = \frac{r^2 K_y}{h_0^2 K_h}$$

$$u_A = \frac{r^2 S}{4 T_b\, t} \text{ (early stage)}$$

$$u_b = \frac{r^2 S_y}{4 T_b\, t} \text{ (late stage)}$$

Equation 7-6

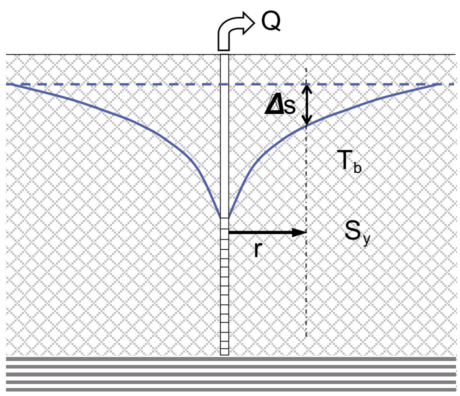


Table 7-5. Assumed values (where they differ from Equation 7-1 and Equation 7-2).

Parameter	Used value	Explanation
K_v	$= K_b$	Same as for Equation 7-1 and Equation 7-2
K_h	$= K_b$	Same as for Equation 7-1 and Equation 7-2
Q from Equation 7-1	1.5–12 l/s	The sum of seepage to tunnels and deformation zones for each construction step (Table 7-6)
Q from Equation 7-3	0.14–0.33 l/s	Results from Table 7-7
t	3–25 yr	Depending on construction step
S_y	0.001	The porosity of the rock is assumed to be 0.1%

Uncertainty

One of the basic assumptions for this calculation method is that the "well" (in our case the repository) has a negligible diameter, i.e. the size of the "well" does not have any affect on the drawdown. In this case, that the "well" is very large compared to the radius of influence, this assumption will obviously not be true.

Furthermore the radii used to calculate the drawdown is the distance from the negligible well to the position where we want to know the drawdown. This raises the question of what distance we are actually calculating, as our "well" has a huge radius compared to the negligible well.

These discrepancies, from the calculation method, make it very hard to evaluate the results as we do not know from were the distances are calculated.

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

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

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

ρ_s = density of saline groundwater (kg/m^3)

ρ_f = density of non-saline groundwater (kg/m^3)

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

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

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

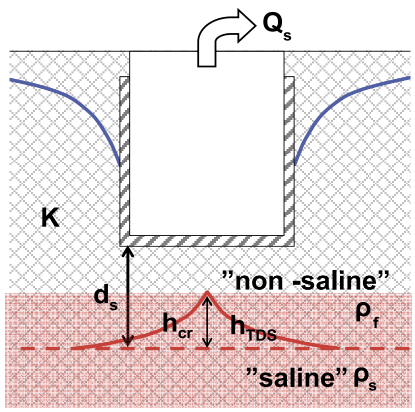
$$Q_{max} = 2\pi d_s^{\,2}K\left[\left(\rho_s - \rho_f\right)/\rho_f\right]$$

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

$$h_{cr} = d_s C$$

(C ~ 0.25-0.60)

Equation 7-7



When the higher values for seepage were used, the returned values of drawdown were unrealistic (in some cases more than thousand metres below the bottom of the repository) which is probably due to the discrepancies described above.

Altogether, these uncertainties leads to that the results from this calculation do not seem realistic.

7.2.4 Upconing of saline groundwater

To assess the risk for upconing of saline groundwater to the repository, a method taking that there is an interface between saline and non-saline groundwater was used as a starting point. It is further assumed that a critical height where stable upconing of saline groundwater to the repository is avoided, if a factor C is multiplied by the distance d_s from the bottom of repository to the saline water interface. The calculated upconing height h_{TDS} must not exceed this limit.

Uncertainty

The calculation of the upconing effect is based on the difference in density of the non-saline and the saline groundwater and the transition depth between them. Thus it is assumed that this transition is rather distinct and can be treated as a strict boundary.

In the Laxemar area, however, no distinct transition is present according to SDM v 1.2 /SKB 2006a/. Instead the salinity increases gradually in a more or less linear manner from 500 m and downward. These conditions make it almost impossible to establish values for both differences in density and depth to the saline groundwater interface. The assumed values become arbitrary and thus it is possible to obtain almost any result depending on how the values are chosen.

7.3 Results

7.3.1 Analytical calculations

Seepage, Method 1 (Equation 7-1)

The results from the calculations are shown in Table 7-6. Comparisons between the calculations using the *no grouting* level and the calculations using K = 10^{-7} m/s show some differences, but they are too small to be shown in the table. The central area is not included in the results, as this calculation method was not easily applied to this part of the repository.

The results from the calculations for the case, no grouting, show a huge range where most of the seepage come from the passages of deformation zones. For the two other cases where grouting is assumed to be of the equivalent to K = 10^{-7} m/s or K = 10^{-9} m/s the impact of the passages are of much less importance.

The largest seepage is estimated for the construction step C, with a median seepage of 29 l/s for the K = 10^{-7} m/s case, and 12 l/s for the K = 10^{-9} m/s case.

The seepage for the tunnel passages through deformation zones where dominated by passage no 2. That is explained by the zone's higher conductivity and its larger passage length.

This calculation method does not take the drawdown into account which, in general, should give an over estimation of the seepage. This becomes evident in the cases with no grouting, especially for the passages through the deformation zones where the greatest drawdown could be expected. Likewise is the interference between tunnels disregarded.

Table 7-6. Results from the calculations of seepage to the deposition-, transport- and main tunnels, along with tunnel passages through the deformation zones. For each result the median value together with the 5- and 95-percentiles are presented.

Construction step	Percentile [%]	Seepage (l/s) No grouting	Grouting level $K_t = 1\times10^{-7}$ m/s	Grouting level $K_t = 1\times10^{-9}$ m/s
Step B	5	0.09	0.09	0.18
Deposition-, transport- and main tunnels (passages through deformation zones excluded)	**50 (median)**	**2.3**	**2.3**	**1.4**
	95	60	60	2.4
Step C	5	0.77	0.77	1.5
Deposition-, transport- and main tunnels (passages through deformation zones excluded)	**50 (median)**	**20**	**20**	**12**
	95	510	510	21
Step D	5	0.64	0.64	1.3
Deposition-, transport- and main tunnels (passages through deformation zones excluded)	**50 (median)**	**16**	**16**	**9.8**
	95	420	420	17
Step E	5	0.47	0.47	0.95
Deposition-, transport- and main tunnels (passages through deformation zones excluded)	**50 (median)**	**12**	**12**	**7.3**
	95	310	310	13
Passages of transport tunnels through deformation zones	5	58	6.3	0.07
	50 (median)	**140**	**8.7**	**0.09**
	95	420	13	0.14

Seepage, Method 2 (Equation 7-3)

This method gives a considerably lower seepage in comparison to method 1 (Equation 7-1). The calculations show that the largest seepage can be expected for construction step AC where the median value is 0.3 l/s when the grouting level is $K = 10^{-7}$ l/s and 0.2 l/s when the grouting level is $K = 10^{-9}$ m/s. Note that the central area A is included in these calculations.

7.3.2 Numerical simulations

All results regarding the numerical simulations come from /Svensson 2006/.

It was not possible to simulate the case with no grouting, as the drawdown in groundwater level became too large for the model to handle. Therefore, only results for the grouting levels $K = 10^{-7}$ m/s and $K = 10^{-9}$ m/s are presented.

Table 7-8 and Table 7-9 show the estimated seepage to the repository for different construction steps and grouting levels. The resulting drawdown around the repository for a case where all parts are held open is presented in Figure 7-8. Figure 7-9 and Figure 7-10 show the salinity distribution around the repository for construction step AE.

Regarding the results, the main conclusions can be summarized as follows:

- The inflow to tunnels and deposition holes will be of the order of 60 l/s for the lower grouting efficiency (maximum conductivity 10^{-7} m/s) and about 30 l/s for the higher grouting efficiency. Considering the uncertainties of the simulations (e.g. uncertainties in the deterministic fracture network) one should not draw any conclusions from the differences between the case "all parts open" and the different inflows in the sequence of open/closed parts.
- The drawdown area will be significant. For both grouting cases an area of about 10 km^2 will get a groundwater table that is depressed by 0.3 m or more.
- The upconing of salt water seems to be small for the cases considered.

Table 7-7. Results from the calculations using Equation 7-3.

Construction step	Percentile [%]	Seepage (l/s) No grouting	Grouting level $K_t = 1×10^{-7}$ m/s	Grouting level $K_t = 1×10^{-9}$ m/s
A	5	0.01	0.01	0.06
	50 (median)	**0.28**	**0.28**	**0.14**
	95	6.9	7.0	0.22
AB	5	0.01	0.01	0.07
	50 (median)	**0.29**	**0.29**	**0.14**
	95	7.2	7.2	0.23
AC	5	0.01	0.01	0.12
	50 (median)	**0.33**	**0.33**	**0.17**
	95	8.2	8.2	0.30
AD	5	0.01	0.01	0.12
	50 (median)	**0.33**	**0.33**	**0.17**
	95	8.2	8.2	0.3
AE	5	0.01	0.01	0.11
	50 (median)	**0.32**	**0.32**	**0.16**
	95	8.0	8.0	0.25

Table 7-8. Inflow (in l/s) to different tunnel sections as a function of time. The opening times in years are given in brackets. Maximum conductivity for tunnel wall cells is set to 10^{-9} m/s.

Tunnel section	Open section A (7)	AB (5)	AC (25)	AD (15)	AE (15)	All (5)
A	4.4	4.2	3.7	4.2	4.5	3.2
B		2.2				1.8
C			15.2			12.9
D				9.5		8.2
E					7.2	6.6
Total inflow	4.4	6.4	18.9	13.7	11.7	32.7

Table 7-9. Inflow (in l/s) to different tunnel sections as a function of time. The opening times in years are given in brackets. Maximum conductivity for tunnel wall cells is set to 10^{-7} m/s.

Tunnel section	Open section A (7)	AB (5)	AC (25)	AD (15)	AE (15)	All (5)
A	14.0	13.1	9.6	16.3	18.3	7.8
B		4.1				2.9
C			27.6			21.8
D				15.3		12.2
E					12.7	10.7
Total inflow	14.0	17.2	37.2	31.6	31.0	55.4

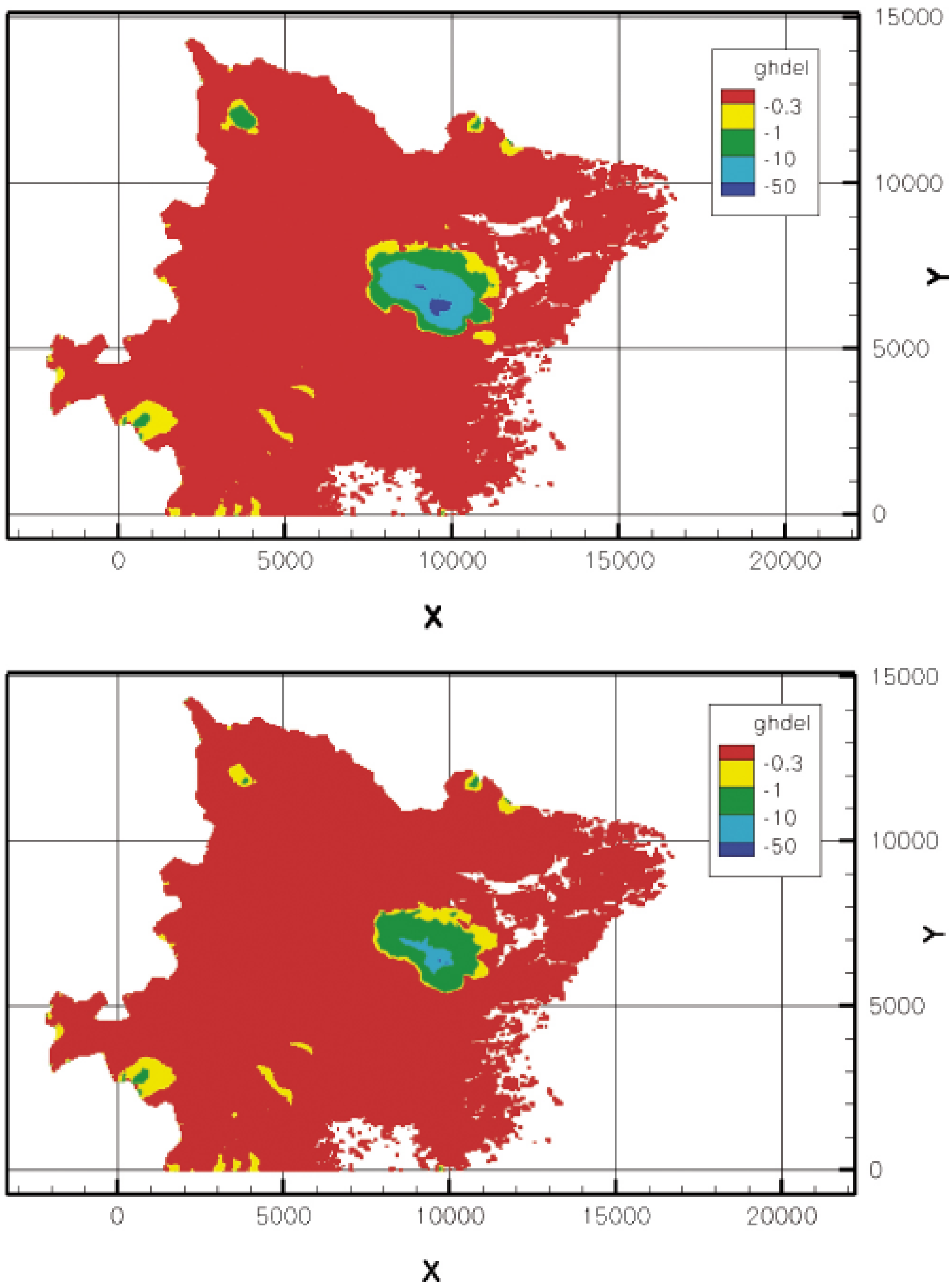


Figure 7-8. *Drawdown at ground level for the case "all parts open". Maximum grouting conductivity is set to 10^{-7} m/s (top) and 10^{-9} m/s (bottom) respectively. Drawdown is calculated with reference to virgin conditions.*

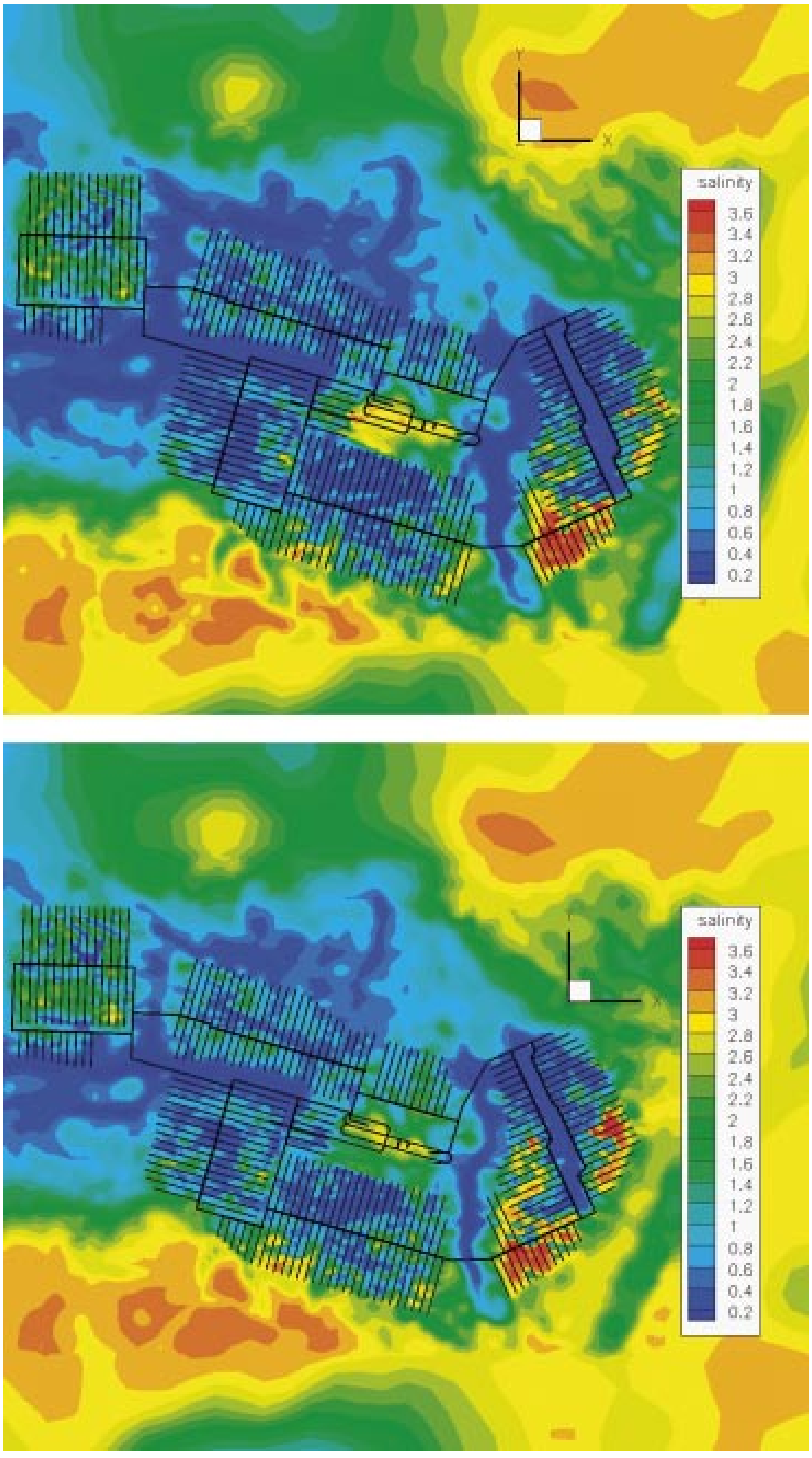


Figure 7-9. *Salinity distribution at repository depth for construction step AE. Maximum grouting conductivity is set to 10^{-7} m/s (top) and 10^{-9} m/s (bottom) respectively.*

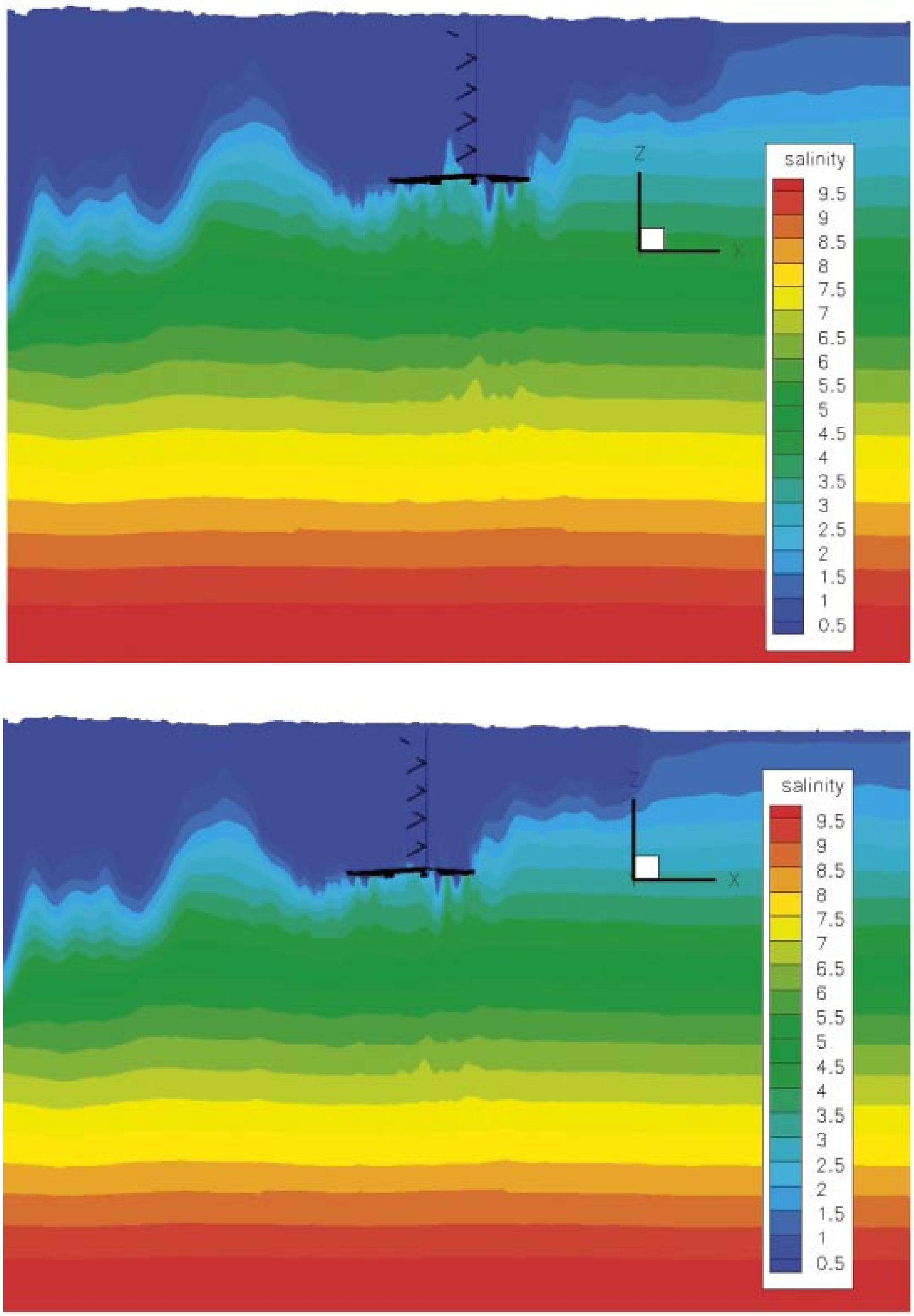


Figure 7-10. Salinity distribution in a west to east section for construction step AE. Maximum grouting conductivity is set to 10^{-7} m/s (top) and 10^{-9} m/s (bottom) respectively.

7.4 Discussions and conclusions

In general, the equations given in the UDP /SKB 2004a/ for analytical solutions are not fully applicable in this context. Specifically, the encountered problems with the analytical calculations for radius of influence, drawdown and upconing of saline groundwater made it clear that no reliable results were given. Thus no comparison between the analytical and numerical methods was possible for these parameters.

Regarding discussion and uncertainties for the numerical calculations, the reader is referred to /Svensson 2006/. In this report, only the results from the numerical calculations are presented and discussed.

The results from the calculations of seepage to the repository show a strong variation depending on calculation method, grouting level and construction step. Method 2 (Equation 7-3) gives much lower values compared to the other analytical method (Equation 7-1) and the numerical calculations for all cases. The method is originally designed for water supply wells with considerable smaller well radii. In such cases, radial flow mainly drains the aquifer, whereas downward flow is neglected. In this case, the "well radius" is hundreds of meters and especially the central area with its shafts and ramps will cause a downward flow that is not included in the method.

The results from Equation 7-1 and the numerical method are consistent for the case where the grouting level is set to be equal to a hydraulic conductivity of 10^{-9} m/s. However, when the lower grouting level is used (10^{-7} m/s), the analytical results becomes considerably higher compared to the seepage values calculated by the numerical method. For the case where no grouting was used, no comparison is possible since this could not be simulated by the numerical model. However, please note that the central area is not included in the analytical method 1.

The discrepancy with the lower grouting level could be explained by the fact that drawdown is not considered in the analytical method. This means that the water level is unchanged as the seepage to the repository starts. Neither is the interference between other tunnels taken into consideration. Remember that the analytical method (Equation 7-1) assumes a single tunnel at a certain depth from the groundwater level. In reality, the water level will decrease as the seepage to the repository starts, and the drawdown caused by nearby tunnels most certainly will have an impact. The effect of the drawdown, as well as the explanation why the case with the higher level of grouting (10^{-9} m/s), gives a better comparison between the analytical and the numerical calculations, evident from Figure 7-11.

As can be seen from Figure 7-11, the pressure levels (which could be said to represent the drawdown) at repository depth, for the two levels of grouting looks rather different. We do not have to consider the exact values of pressure to justify this, we just have to consider the pressure distribution. For the lower level of grouting, the pressure levels are quite evenly distributed over the repository, thus the area between single tunnels is clearly affected. This, in return, means that the reduction in pressure (or drawdown) caused by one tunnel will have a considerable effect on the surrounding tunnels. For this case, the assumption of the single tunnel not affected by drawdown is not very good if you believe in the numerical calculations. The results from the analytical and the numerical calculations therefore differ considerably.

However, if we look at the case with the higher level of grouting, the assumption now is more in accordance with the numerical calculations (and probably the true case). We can see that the influence of each single tunnel is much more limited, and thus the effect on a single tunnel by the nearby tunnels is not as big as for the former case. In this case, the assumptions for the analytical calculation are more in accordance with the numerical calculations and thus the results are more similar.

Both the numerical and the analytical solutions show that the percentage of seepage through the intersections between transport tunnels and deformation zones decreases as the level of grouting increases. This is what could be expected, as the grouting transforms the passages through the deformation zones to be more like the surrounding rock mass. At the higher level of grouting, the affect of the deformation zones are almost negligible.

The results from the numerical calculations show that for the case where no grouting is used, the passages through the deformation zones will dominate the seepage completely. In fact, the seepage to other parts will actually be smaller than when grouting is used, as the deformation zones will "steal" water from the surrounding rock mass.

The results from the analytical calculations of seepage range from 0.2–29 l/s for construction step AC give the highest values. For the numerical simulations, the corresponding values are 19–37 l/s.

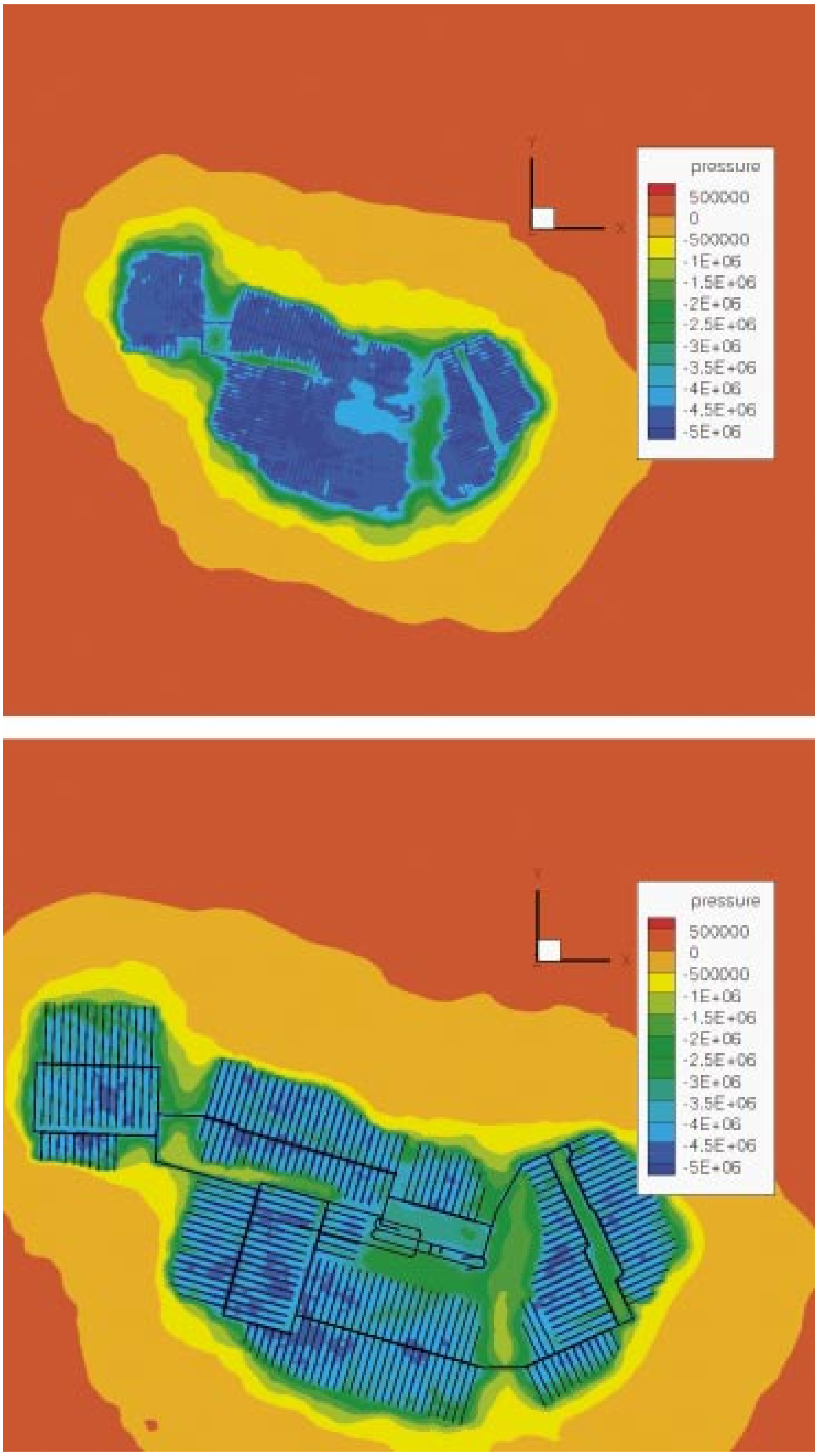


Figure 7-11. *Pressure distribution at the repository depth. Maximum conductivity is set to 10^{-7} m/s (top) and 10^{-9} m/s (bottom) respectively.*

The drawdown area, as calculated using the numerical model, will be significant. For both grouting cases, an area of about 10 km^2 will get a groundwater table that is depressed by 0.3 m or more.

The results from the numerical simulations show small possibilities for inflow of too saline groundwater.

8 Estimation of rock grouting requirements

The purpose of this study is to estimate the grouting quantities for the entire repository complex according to the basic layout in Chapter 5. Before an estimate of the grouting quantities can be given, a number of steps must be described, such as basic assumptions, requirements and the basic construction plan. Grouting quantities will then be estimated by using the gained experience and with analytical calculations.

8.1 Input data and assumptions

8.1.1 Site conditions for grouting

The proposed basic layout, as per Chapter 5, is basically located in two rock domains, A (RSMA) and M (RSMM), see Figure 8-1.

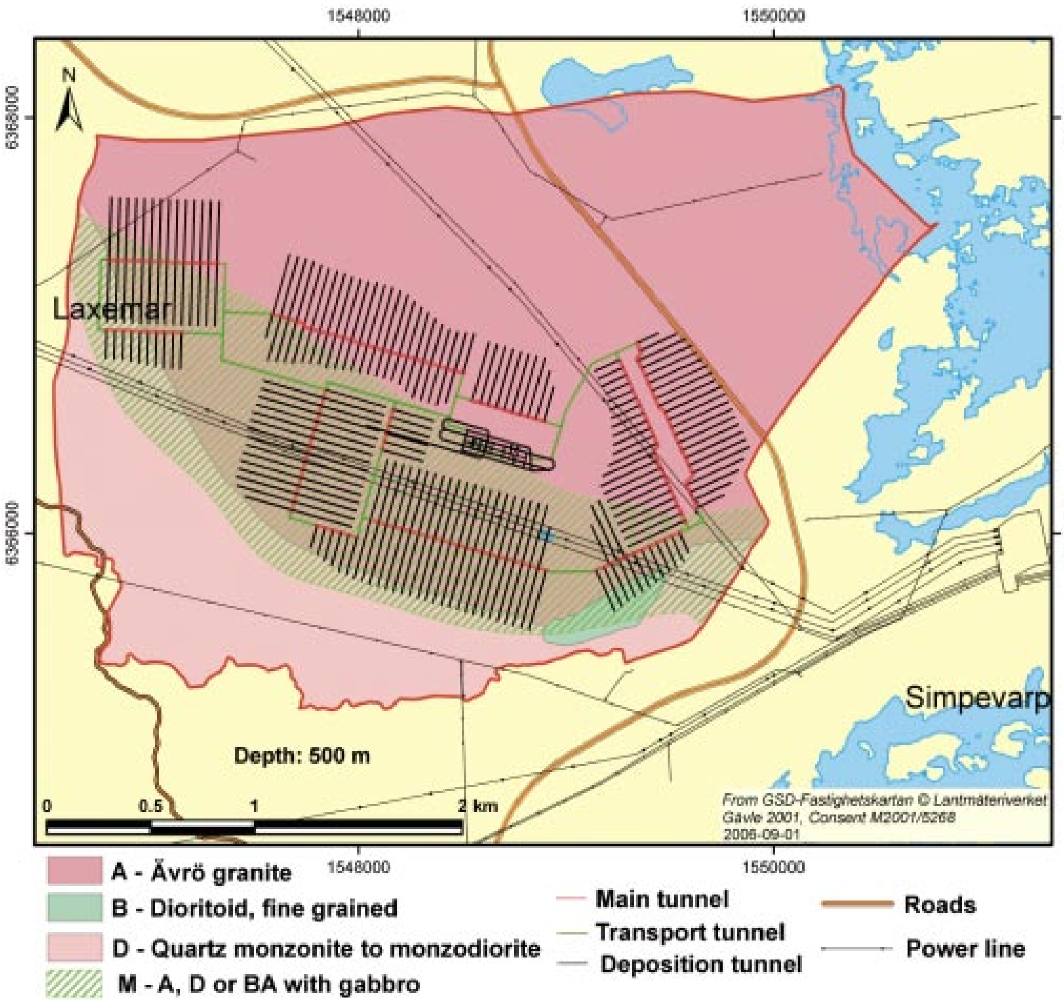


Figure 8-1. *Proposed basic layout for the repository with interpretations from SDM v 1.2.*

The hydraulic conductivity of the two domains is presented in SDM v 1.2 /SKB 2006a/, see Table 8-1.

The conductivity distribution for Rock domain A is given in SDM v 1.2, see Figure 8-2. A corresponding distrubution for Rock domain M is lacking in the site description.

Six transport tunnels will pass through the known deformation zones. Information about the deformation zones and data at the crossing point are presented in Chapter 6.

The proposed repository layout, including the central area, consists of a number of different types of tunnels with different geometries /SKB 2002/. In preparation for the estimation of grouting requirements, the tunnel types have been classified into six different types of tunnels/rock caverns, see Table 8-2.

Table 8-1. Conductivity data for Rock domains A and M, semi-correlated (block 100 m), /SKB 2006a/.

Rock domain	Log10 (Kz*), median value
A	– 8.12
M	– 9.05

* Kz is the vertical flow in the rock mass according to SDM v 1.2.

Table 8-2. The different types of tunnels and rock caverns have been simplified into six main types.

Tunnel type	Tunnel area, m^2	Equivalent radius, m
1. Main tunnel	66	4.6
2. Transport and deposition tunnels	25–46	2.8–3.8 (3.0)*
3. Tunnels in the central area	8–66	1.6–4.6 (3.5)*
4. Rock caverns in the central area	126–233	6.3–8.6 (7.5)*
5. Vertical shafts and silos	5–38	1.3–3.5 (2.0)*
6. Ramp tunnels	30–66	3.1–4.6 (3.5)*

* Selected type radius for tunnel type.

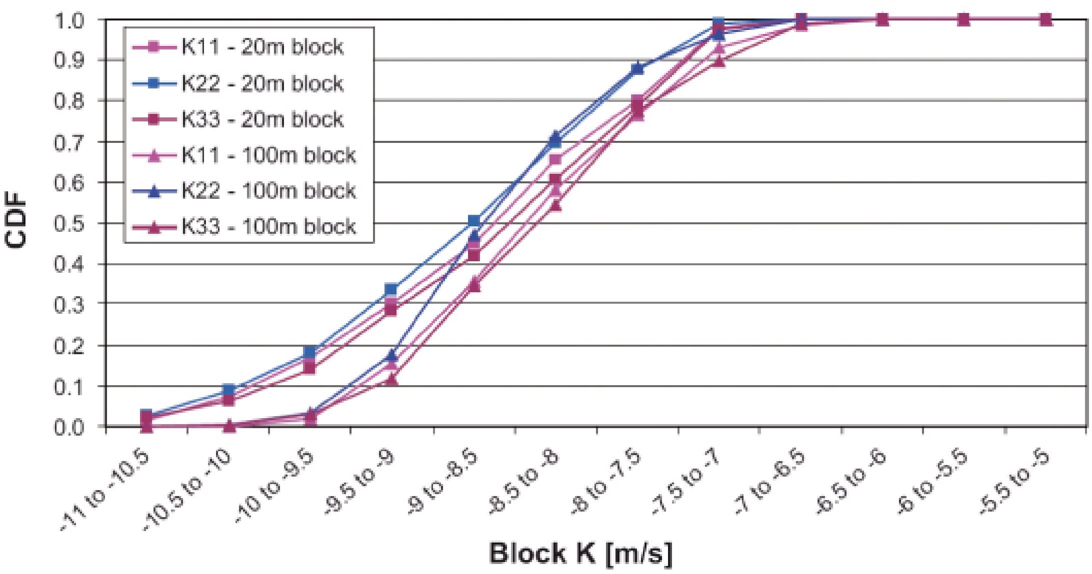


__Figure 8-2.__ The semi-correlated conductivity for Rock domain A and a depth greater than 300 m /SKB 2006a/.

A compilation of the lengths of the different tunnel types is presented in Table 8-3 for respective domains.

Two grouting levels have been obtained in Chapter 7:

1. Grouting level 1 corresponds to a grouted zone with $K = 10^{-7}$ m/s.
2. Grouting level 2 corresponds to a grouted zone with $K = 10^{-9}$ m/s.

In order to assess the grouting input and its level of difficulty, the so-called grouting effect has been calculated in accordance with /Eriksson and Stille 2005/:

$$1 - \frac{q_{grouted}}{q_{ungrouted}} \qquad \text{Equation 8-1}$$

where $q_{ungrouted}$ and $q_{grouted}$ are calculated on the same principles as in Sections 4.3 and Chapter 7.

No detailed recipes for mortar mixes have been made for this work. For the functional requirements of the mixes a number of guidelines are given instead. Pre-grouting is for the most part expected to be performed with cement-based grout. Three types of grout and a hole-filling grout, apply as guidelines for the calculation of grouting quantities, see Table 8-5.

The guidelines for grouting pressure are partly associated with the distribution of the grout in the rock mass and partly with the surrounding groundwater pressure.

Table 8-3. Compilation of the lengths of different tunnel types for Rock domains A and M.

Rock domain A					
Part of repository	**Tunnel type**				
	Deposition tunnels	Other tunnels and rock caverns			
Tunnel type	Deposition	Main	Ramp*	Shaft and silo*	
Total length	**23,165 m**	**4,465**	**6,578 m**	**2,748 m**	
Rock domain M					
Part of repository	**Tunnel type**				
	Deposition tunnels	Other tunnels and rock caverns			
Tunnel type	Deposition	Main	Central**	Rock cavern central**	Shaft
Total length	**47,120 m**	**6,435 m**	**2,090 m**	**789 m**	**509 m**

* All lengths for ramp tunnels, shafts and silos down to the central area are added here in Rock domain A.
** All lengths for tunnels and rock caverns located in the central area and on level 500 m are added here in Rock domain M.

Table 8-4. The level of difficulty of the grouting is assessed in accordance with the guidelines in /Eriksson and Stille 2005/.

Grouting level	The difficulty of grouting		
(m/s)	< 90%	90–99%	> 90%
> 10^{-7}	Easy	Easy	Moderate
10^{-7}–10^{-8}	Easy	Moderate	Difficult
< 10–8	Moderate	Difficult	Difficult

Table 8-5. Assumed guidelines for grout.

Grout	Guidelines for grout properties	Reason
I	Full penetration of fractures to approximately 90 µm. Yield value 5–10 Pa	Basic grout with possibilities for variations so that other specific properties can be fulfilled, for example viscosity, separation stability, gel time and strength
II	Full penetration of fractures to approximately 60 µm. Yield value: no demands	A grout for fine fractures
III	Rapid hardening, i.e. an accelerating grout	A grout for limiting the penetration length and dilution effects
Hole filling	Vct < 0.5	For the plugging of watertight grouting holes

The performance of grouting curtain geometry is primarily dominated by the conditions of the rock mass and the grouting requirements. Grouting curtains are also depending on practical aspects, such as drilling equipment, driving efficiency, etc.

The guidelines have been conventionally and practically elaborated in such a way that the curtain has been concentrated mainly at the higher sealing levels, see Figure 8-3. The borehole diameter is assumed to be 51 mm.

The guidelines for the hole geometry in vertical shafts is basically the same as for tunnel curtains. However, the holes are drilled parallel to the shaft where the distance between grout holes and shaft contours is set to be 2 m.

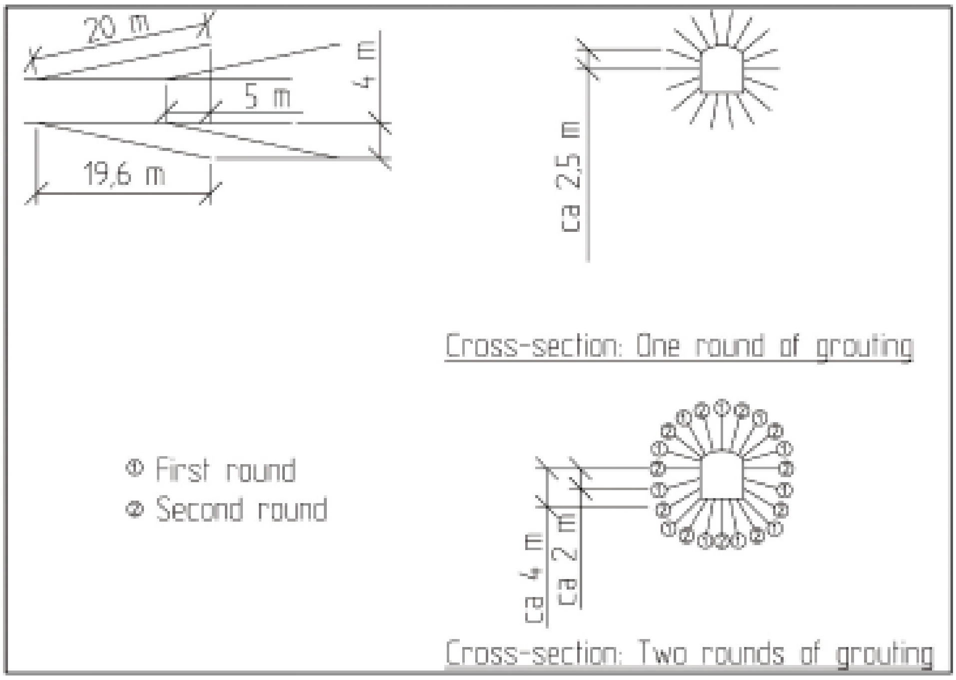


***Figure 8-3.** Basic performance of grouting curtains for deposition tunnels. The lower cross-section shows a concentrated number of holes with two rounds of grouting at a higher level of difficulty.*

8.1.2 Assumed grouting procedures

The grouting procedure will be described so that grouting quantities can be estimated. This means that no detailed descriptions will be given with respect to grouting work criteria, methods, checks, equipment, etc.

The following points will be described in order to be able to estimate the grouting quantities:

- level of grouting difficulty,
- number of grouting rounds, including any check rounds,
- number of probing holes in the curtain,
- decision on grouting and the proportion of the rock excavation (tunnels, rock caverns or shafts) that is expected to be pre-grouted,
- during pre-grouting; number of holes in the curtain that remain unsealed, i.e. that are to be grouted, and vice versa, number of holes that just have to be filled,
- selected grout and pressure,
- possible new round of grouting or check round.

Terms that can be seen in the description are:

- selective pre-grouting: parts of the tunnels, i.e. based on the results of the probing holes and specified criteria for whether or not grouting is necessary, a grouting curtain is made and subsequent grouting carried out,
- continuous pre-grouting: the entire length of the tunnel is grouted, i.e. grouting curtain and grouting are always executed,
- pre-investigation: drilling and investigations in a number of limited grouting holes. Pre-investigation is always carried out for sections with selective grouting, but not for sections with continuous pre-grouting,
- exploratory/probe drilling: drilling and investigation in boreholes that do not belong to the grouting curtain; primarily in connection with the passages of deformation zones,
- drilling information: information on drilling, such as rate of penetration, torque, flushing pressure, rotation pressure, etc,
- plugging: watertight drill holes are filled with a so-called hole-filling grout.

Grouting procedure in Rock domain A

The median conductivity of the rock mass is 7.6×10^{-9} m/s. Areas with individual open fractures and locally higher conductivity will probably need some grouting. Water seepage in unsealed conditions is estimated to be approximately 17 l/min, 100 m.

Sealing level 1

The principle for grouting with sealing level 1 is selective pre-grouting. Brief description of grouting procedure for tunnels and rock caverns respective vertical shafts are present in Table 8-6.

Table 8-6. Description of the grouting procedures.

Grouting procedure	Tunnels and rock caverns	Vertical shafts
1	The required sealing effect is estimated to be 31%, which is defined as “easy” grouting.	Drilling of all grout holes to full length (100–200 m) in a circle around the shaft
2	If grouting is necessary, one grouting round is expected to be enough, see Figure 8-3.	Water-loss measurement in sections of 20 m and drilling information are evaluated against specified criteria for decision whether or not to perform the grouting. If the criteria are met, the sections will be grouted. Some 5% of the sections in Rock domain A are believed to have a higher conductivity than Sealing Level 1, see Figure 8-2.
3	Exploration/probe drilling, i.e. approximately 1/3 of the grout holes in the curtain should be performed.	The required sealing effect is estimated to be 31%, that is defined as “easy” grouting
4	Drilling information and water-loss measurements should be evaluated against established criteria for decision whether or not to perform the grouting. If the criteria are met, the entire curtain should be drilled. Otherwise the exploration drilling should be finished with borehole plugging. Approximately 5% of Rock domain A is believed to have a higher conductivity than sealing level 1, see Figure 8-2.	One round of grouting per section will be enough.
5	After the remaining grouting has been drilled, water loss measurements are performed. The number of drill holes in the curtain, i.e. that are to be grouted, is estimated to be approximately 30% of all drill holes in the curtain. The number of watertight drill holes in the curtain, i.e. holes that will be plugged with hole-filling grout, is estimated to be in the region of 70%.	Processing of the grout holes in 20 m sections starting from the bottom and moving upwards. Approximately 5% of the hole length will be grouted and 95% plugged with filling grout.
6	The selected grout is Recipe 1. The yield value is assumed to be 6 Pa, and the maximum pressure is set at 50 bar over the groundwater pressure.	The selected grout is Recipe 1, the yield value is assumed to be 6 Pa, and the maximum pressure is set at approximately 40 bar over groundwater pressure.
7	No further drilling or grouting is expected	No further drilling or grouting is expected

Sealing Level 2

Continuous pre-grouting should be possible since the sealing requirements are high. Nevertheless, methods with pre-investigation and selective pre-grouting have been selected. This is basically because a large proportion of the grout curtains in a continuous pre-grouting are not expected to give any water losses or grouting quantities. On the contrary it may need a considerably longer time than involving a selective pre-grouting. Brief description of grouting procedures for tunnels and rock caverns as well as vertical shafts is present in Table 8-7.

Table 8-7. Description of the grouting procedure.

Grouting procedure	Tunnels and rock caverns	Vertical shafts
1	The required sealing effect is estimated to be 93%, which is defined as "difficult" grouting.	Drilling of all holes in Grouting Round 1 to full length (100–200 m) in a circle around the shaft
2	If grouting is necessary, two grouting rounds are expected, see Figure 8-3, and check holes with possible post-grouting. Several grouting rounds and check holes in sealing level 2 means that the tunnel driving takes a considerably longer time compared with driving in Sealing Level 1.	Water-loss measurement in sections of 20 m and drilling information to be evaluated against specified criteria to perform the grouting or not. If the criteria are met, the sections will be grouted. Approximately 60% of the sections in Rock domain A are believed to require grouting.
3	Pre-investigation, i.e. approximately 75% of the grout holes in grouting round 1 are drilled	Processing of grout holes in 20 m sections starting from the bottom and working upwards. Approximately 60% of the hole length will be grouted and some 40% plugged with hole-filling grout
4	Drilling information and water-loss measurements should be evaluated against established criteria to perform the grouting or not. If the criteria are met, all holes in Grouting Round 1 should be drilled. Otherwise the exploration drilling should be finished with plugging. Approximately 75% of Rock domain A is believed to have a higher conductivity than sealing level 2, see Figure 8-2.	The selected grout is Recipe 1, the yield value is assumed to be 6 Pa, and the maximum pressure is set at approximately 30 bar over groundwater pressure.
5	Theoretically, this would mean that a selective pre-grouting is needed in approximately 75% of the rock mass. To find such a large proportion as 75% of the finely fractured rock mass that corresponds to low conductivities with exploration/probe drilling, is not considered to be likely with the present production methods for pre-investigation that are at available. With today's pre-investigation methods, it is estimated that grouting will be performed for approximately 60% of the tunnel length.	Drilling of all the holes in Grouting Round 2 to full length (100–200 m) in a circle around the shaft
6	After the remaining grouting holes have been drilled, water loss measurements are performed. The number of drill holes in the curtain, i.e. that are to be grouted, is estimated to approximately 35% of all drill holes in the curtain. The number of watertight drill holes in the curtain, i.e. which will be plugged with hole-filling grout, is estimated to be in the region of 65%.	Water-loss measurement in sections of 20 m and evaluation of drilling information against specified criteria to perform grouting or not. If the criteria are met, the sections will be grouted. Some 25% of the sections in Rock domain A are believed to require grouting.
7	For Grouting Round 1, Recipe 1 will be used, i.e. the yield value is assumed to be 6 Pa, and the maximum pressure is set at 40 bar over the groundwater pressure.	Processing of grout holes in 20 m sections starting from the bottom and working upwards. Approximately 25% of the hole length will be grouted and some 75% plugged with hole-filling grout.
8	Drilling of Grouting Round 2 with water-loss measurements in all holes. The number of drill holes in Grouting Round 2, i.e. that are to be grouted, is estimated to be approximately 25%. The number of watertight holes in the curtain, i.e. which will be plugged with hole-filling grout, is estimated to be approximately 75%.	The selected grout is Recipe 2, the yield value is assumed to be 1 Pa, and the maximum pressure is set at approximately 30 bar over groundwater pressure.
9	For Grouting Round 2, Recipe 2 will be used, i.e. the yield value is assumed to be 1 Pa, and the maximum pressure is set at 30 bar over the groundwater pressure	No further drilling or grouting
10	Drilling of approximately 5 control holes water-loss measurement and possibly grouting. It is assumed that approximately 20% of the holes will be grouted in accordance with Point 8.	–
11	Plugging of watertight grout holes with hole-filling grout.	–

Grouting procedure in Rock domain M

The median conductivity of the rock mass is 0.9×10^{-9} m/s. Areas with individual open joints and locally higher conductivity will probably need some grouting. Water seepage in unsealed conditions is estimated to be approximately 2 l/min, 100 m.

Sealing level 1

The principle for grouting with sealing level 1 is selective pre-grouting.

Since the conductivity distribution for Rock domain M is not available in the SDM v 1.2, the groutable part of Rock domain A's share has been used as the starting point and as the ratio between the conductivities. Approximately 2.5% of Rock domain M, half the value of Rock domain A, is believed to have a higher conductivity than Sealing Level 1.

The description of the grouting approach is the same as for Rock Domain A. The aspect that differs is the length of grouted section due to denser rock mass, such as:

- tunnels and rock caverns: the sections that are grouted are reduced from 5 to 2.5%
- vertical shafts: the sections that are grouted are reduced from 5 to 2.5%

Sealing Level 2

Since the conductivity distribution is not available for Rock domain M in the SDM v 1.2, the groutable part of Rock domain A's share has been used as the starting point and the ratio between the conductivities. Approximately 30% of Rock domain M, half the value of Rock domain A, is believed to have a higher conductivity than Sealing Level 2.

The description of the grouting approach is the same as for Rock Domain A. The aspects that differ, is the length of grouted section due to denser rock mass, such as:

- tunnels and rock caverns: the sections that are grouted are reduced from 60 to 30%
- tunnels and rock caverns: holes that need to be grouted in Grouting Round 1 are reduced from 35 to 30%
- tunnels and rock caverns: holes that need to be grouted in Grouting Round 2 are reduced from 25 to 20%
- vertical shafts: sections that are grouted in Round 1 are reduced from 60 to 30%
- vertical shafts: sections that are grouted in Round 2 are reduced from 25 to 20%

Grouting approach in deformation zones

The required sealing efficiency of the passages through deformation zones has been calculated and the level of difficulty for the grouting has been assessed. The sealing efficiency in the deformation zones varies, from 89 to 99.9%, depending on zone and sealing level. The level of difficulty is mainly assessed as being "difficult".

Sealing Level 1 it could be possible to achieve with continuous pre-grouting, i.e. with 2 grouting rounds in accordance with Figure 8-3 and control holes, with cement-based grout.

Achieving Sealing Level 2 in the deformation zones by means of cement grout will be time-consuming. Sealing works will be carried out with continuous pre-grouting rounds.

Contingency measures for a number of problems and difficulties will be available in connection with passages through deformation zones, see Chapter 6.

General approach for grouting through Passage 1 to 6 at Sealing Level 1:

1. Drilling of Grouting Round 1 as per Figure 8-3. For the two longer passages, the holes will be extended to 30 m.
2. Water-loss measurement, all drill holes are believed to be water leaking.
3. Grouting of all drill holes with Recipe 3, i.e. an assigned yield value of 20 Pa, and maximum pressure over the groundwater table of approximately 20 bar or 40 bar, depending on the level of difficulty.
4. Drilling of Grouting Round 2 followed by water-loss measurement. All drill holes are believed to be water leaking.
5. Grouting of all drill holes with Recipe 3, i.e. an assigned yield value of 20 Pa, and maximum pressure over the groundwater table of approximately 20 bar or 40 bar, depending on the level of difficulty.
6. Drilling of control holes (6pcs) and any re-grouting that is necessary. Grouting of approx. 50% of the control holes with Recipe 1, i.e. assigned yield value 6 Pa, and maximum pressure over the groundwater table of 30 bar.
7. Plugging of watertight control holes with hole-filling grout.

General approach for grouting through Passages 1 to 6 at Sealing Level 2:

1. Drilling of Grout Round 1, hole distance = 5 m, hole length = 20 m or 30 m depending on passage, gauge = 6 m (12 pcs).
2. Water-loss measurement, all drill holes are believed to be water leaking.
3. Grouting of all drill holes with Recipe 3, i.e. an assigned yield value of 20 Pa, and maximum pressure over groundwater level of approximately 40 bar.
4. Drilling of Grout Round 2 as per grout curtain "Sealing Level 2 and Round 1" as well as 4 pcs tunnel face holes (total of 15 pcs), see Figure 8-3.
5. Water-loss measurement, all drill holes are believed to be water leaking.
6. Grouting of all grout holes with Recipe 3, i.e. an assigned yield value of 20 Pa, and maximum pressure over groundwater level of approximately 40 bar.
7. Drilling of Grout Round 3 as per grout curtain "Sealing Level 2 and Round 2", see Figure 8-3.
8. Grouting of water leaking drill holes, approx. 75% of the holes in Grouting Round 3. Recipe 2, i.e. an assigned yield value of 6 Pa and maximum pressure over the groundwater table of approximately 40 bar.
9. Drilling of control holes (11 pcs) and any re-grouting. Grouting of approx. 50% of the control holes with Recipe 2, i.e. an assigned yield value of 6 Pa, and maximum pressure over the groundwater table of approximately 30 bar.
10. Plugging of watertight grout and control holes with hole-filling grout.

8.2 Execution

8.2.1 Estimate of grouting quantities

The estimate of grouting quantities is based on gained experience and is partialy documented. Examples of documented experience are data from the Äspö HRL /SKB 1994/ and /SKB 1997a–b/. Other documented experience from similar depths of approximately 500 m, is extremely limited and reference is often made to the mining industry with entirely other requirements and criteria for a case of a deep-level repository.

Subsequently, grouting quantities shall be verified with analytical methods and the various estimates and calculations of grouting quantities will be compared /SKB 2004a/.

The verification shall be carried out with the aid of analytical methods, one based on "grouting technology", i.e. rheological model, and one on porosity, i.e. porosity model, according to /SKB 2004a/.

Empirical estimate

Estimates are based on a number of parameters for respective domains or zones, sealing level and repository component. The parameters are curtain geometry, number of grout curtains that are grouted, the number of holes that are grouted in the curtain, the consumption of grout per drilling metre and any re-grouting that may be necessary.

Since some of the holes in the grout curtains have been selected for use as exploration/probe holes, there will be two types of curtains. One of the curtain types is called an exploratory curtain and will be plugged if the results of the probing holes from the water-loss measurements do not result in the need for any further drilling and grouting.

The second type of curtain is assigned as grouting curtain and is used when the results of exploration/probe holes from water-loss measurement result in drilling of a grout curtain.

Initially, an estimate is made of the number of grout- or investigation curtains per domain, see Section 8.1.3. For the deformation zones, it is assumed that continuous grouting will be performed along the entire length of the passage.

After this, the number of curtains or investigation drilling metres can be calculated. Grouting curtains contain two types of holes: water leaking holes that are grouted and watertight holes that are only plugged.

The estimate of grouting quantities will be based on the number of drilling metres of grouted holes in a grouting curtain and its ratio to empirical grout volumes per metre of borehole, see Table 8-8.

For Sealing Level 1, a grout amount of 20 l/m is assumed. For Sealing Level 2, which contains more grout holes per curtain, a grout amount of 15 l/m is assumed as an average for both grouting rounds. Hole-filling quantities from investigation holes will be based on the theoretical hole volume multiplied by 0.9, i.e. 10% of the hole is anticipated not to be filled with so-called plug grout, see Table 8-9.

Each grouting curtain length in the zones may contain several grouting rounds depending on the sealing level, as described in Section 8.1.3. This gives a number of drilling metres that relate to empirical grout volumes per metre of borehole, which is assumed to be 25 l/m. The empirical estimate of the grouting quantities for the various zone passages are presented in Table 8-10.

Table 8-8. Grouting quantities in grout curtains for respective repository components, domains and sealing levels based on empirical experience.

Sealing level	Rock domain	Deposition (m^3)	Other tunnels/ rock caverns (m^3)	Total (m^3)
1	A	202	133	**335**
	M	205	51	**256**
2	A	2,678	1,256	**3,934**
	M	2,412	626	**3,038**

Table 8-9. Hole-filling quantities in investigation holes for respective repository components, domains and sealing levels.

Sealing level	Rock domain	Deposition (m^3)	Other tunnels/ rock caverns (m^3)	Total (m^3)
1	A	317	208	**525**
	M	662	162	**824**
2	A	178	113	**291**
	M	633	170	**803**

Table 8-10. Grouting quantities for zone passages based on empirical experience.

Passage	No of curtain lengths (pcs)	Sealing level 1 (m^3)	Sealing level 2 (m^3)
1	4	50	83
2	5	63	103
3	1	13	21
4	1	13	21
5	1	13	21
6	1	13	21
Total		**165**	**270**

Analytical calculation

In order for the two analytical calculations, the rheological and the porosity models, to be comparable with each other, certain assumptions must be similar. In practice, three separate analyses are conducted since the analyses of the porosity model are carried out on the basis of two different approaches with respect to porosity. The common assumptions are the hydraulic conductivity of the rock mass, the distribution of the grout in the rock mass and the proportion of the grouting in the tunnel lengths.

In consultation with SKB it was decided that the following assumptions shall apply for the calculations:

- The mean conductivity of the different domains as per SDM v 1.2 shall be used
- The assumed conductivities of the deformation zones as per Chapter 6
- The grout distribution in the porosity model shall be based on the rheological model, i.e. be calculated by using a selected grouting pressure, grout properties (fluid limits) and the hydraulic fracture width.

Rheological analysis model

The rheological model is based on the fact that the grouting continues to a rheological stop /Hässler 1991/ as per:

$$I = \frac{\Delta P \cdot b}{2 \cdot \tau_o} \qquad \text{Equation 8-2}$$

where I is the grouting distribution, ΔP is the grouting overpressure, τ_o is the fluid limit of the grout and *b* is the width of the fracture plane.

From Equation 8-2, fracture geometry assumptions and the relationship between fracture width and transmissivity, T, we obtain Equation 8-3 /Janson 1998/ for the volume in a borehole:

$$V = \left(\frac{\Delta P}{2 \cdot \tau_o} \right)^2 \frac{12 \cdot T \cdot \mu_w \cdot \pi}{\rho_w \cdot g} \qquad \text{Equation 8-3}$$

Transmissivity T can be expressed as K×L (conductivity and × hole length).

The equation has certain restrictions:

- Unlimited length of fractures, no filtration stop and no practical limitations, such as max volume/time.
- The grout and water give the same measured transmissivity and fracture aperture, i.e. assume a hydraulic aperture.
- The equation applies for a certain specific grouting hole where the transmissivity is known and is not affected by surrounding boreholes.

In order to be able to calculate the grouting quantities with the rheological model, certain assumptions must therefore be made:

1. The yield value and grouting pressure of the grout are constant over time.
2. The ratio between pressure and yield value will be max 1×10^6. If the ratio is greater, the yield value will be adjusted so that the ratio becomes 1×10^6. This will be done in order to simulate a rheological stop before other factors restrict the distribution.
3. All previous grouting holes in the curtain have the same conductivity as the rock mass.
4. At Sealing Level 2, it is assumed that the mean conductivity is reduced by half after each grouting round. The corresponding conductivity reduction in the zones is by a factor of 10.

Calculated grouting quantities for Rock domains A and M are shown in Table 8-11 and for the zone passages in Table 8-12.

Table 8-11. Grouting quantities for respective repository components, domains and sealing levels based on the rheological model.

Sealing level	Rock domain	Deposition (m^3)	Other tunnels/ rock caverns (m^3)	Total (m^3)
1	A	105	60	**165**
	M	58	14	**72**
2	A	1,446	886	**2,332**
	M	941	234	**1,175**

Table 8-12. Grouting quantities for the different zone passages based on the rheological model.

Passage	Conductivity (m/s)	Sealing level 1 (m^3)	Sealing level 2 (m^3)
1	2.0×E–06	56	67
2	2.6×E–06	123	146
3	2.0×E–06	6	7
4	2.0×E–06	6	7
5	2.0×E–06	6	7
6	2.0×E–06	6	7
Total		**203**	**146**

Porosity model

The porosity model is based on the fact that the existing pores in the rock mass are filled with grout to a certain distance from the tunnel periphery.

The grout volume per tunnel metre is calculated in the basis of the following equation

$V_m = p \cdot \pi \cdot R^2$ Equation 8-4

where p is the porosity of the rock mass (–) and estimated with the aid of DFN-data (P_{33}) or with the empirical equation /Brotzèn 1990/:

$\log p = 0.17 \cdot \log K_b - 1.7 \pm 0.3$ Equation 8-5

where K_b (m/s) is the mean conductivity of the rock mass.

R (m) is the sum of the tunnel's equivalent radius and grout distribution, I, that is calculated by using Equation 8-6 where the relation between plane parallel fractures and transmissivity are used:

$$b = \sqrt[3]{\frac{T \cdot 12 \cdot \mu}{N \cdot \rho \cdot g}}$$ Equation 8-6

N is the number of hydraulic fractures in the borehole, which is assumed to be 1 in the rock domains and 4 in the deformation zones.

The distribution of the grouting can then be expressed as:

$$I = \frac{\Delta P}{2 \cdot \tau_o} \sqrt[3]{\frac{K \cdot L \cdot 12 \cdot \mu}{N \cdot \rho \cdot g}}$$ Equation 8-7

Result – based on DFN

The mean porosity, P_{33}, of Rock domains A and M is, according to Section 4.3, 1×10^{-4} and 2×10^{-5} respectively. The results of the calculations of grouting quantities (grout distribution and volume) in the domains are compiled in Table 8-13.

The deformation zones lack DFN data (P_{33}) which means that no calculations can be made of grouting quantities for passages through deformation zones.

Table 8-13. Grouting quantities, grout distribution and volume, for respective domains and sealing levels based on DFN.

Sealing level	Rock domain	Distribution of grouting (m)	Volume, deposition tunnels (m³)	Volume, other tunnels/ rock caverns (m³)	Total (m³)
1	A	26	306	189	**495**
	M	13	17	7	**24**
2	A	21–31 (26)*	3,672	2,273	**5,945**
	M	10–15 (13*)	203	81	**284**

* Selected value for the calculations.

Result – based on empirical porosity ratios

The mean conductivity of Rock domains A and M is 7.6×10^{-9} m/s and 0.9×10^{-9} m/s respectively. This gives a porosity of 8.31×10^{-4} and 5.78×10^{-4} respectively.

The results of the calculations of grouting quantities (grout distribution and volume) in the domains are compiled in Table 8-14.

The results for the calculations of grouting quantities for the different passages through the zones are compiled in Table 8-15.

8.3 Results

8.3.1 Comparison of grouting quantities

Table 8-16 gives a comparison of the rock domains and a corresponding comparison for the zones is given in Table 8-17.

Table 8-14. Grouting quantities, grout distribution and volume, for respective domains and sealing levels based on empirical porosity.

Sealing level	Rock domain	Distribution of grouting (m)	Volume, deposition tunnels (m^3)	Volume, other tunnels/ rock caverns (m^3)	Total, (m^3)
1	A	26	2,544	1,575	**4,119**
	M	13	489	195	**684**
2	A	21–31 (26)*	30,526	18,896	**49,422**
	M	10–15 (13*)	5,871	2,344	**8,215**

* Selected value for the calculations.

Table 8-15. Grouting quantities for respective zones based on empirical porosity ratios (the values have been rounded off to the nearest whole figure).

Passage	Empirical porosity (–)	Distribution, m Level 1	Level 2	Grout volume (m^3) Level 1	Level 2
1	2.14×10–3	4–13 (10*)	4–20 (15*)	57	109
2	2.24×10–3	2–13 (10*)	4–20 (15*)	101	193
3	2.14×10–3	4–13 (10*)	4–20 (15*)	3	7
4	2.14×10–3	4–13 (10*)	4–20 (15*)	3	7
5	2.14×10–3	4–13 (10*)	4–20 (15*)	3	7
6	2.14×10–3	4–13 (10*)	4–20 (15*)	3	7
Total				**160**	**328**

* Selected value for the calculations.

Table 8-16. Comparison between different estimates/calculations of quantities in grout curtains for each domain and sealing level.

Level	Rock domain	Empirical estimation (m^3)	Rheological model (m^3)	Porosity model, DFN (m^3)	Porosity model, empirical (m^3)
1	A	335	165	495	419
	M	256	72	24	684
2	A	3,934	2,332	5,945	49,422
	M	3,038	1,175	284	8,215

Table 8-17. Comparison between different estimates/calculations of quantities for zones/passages and sealing level.

Level	Estimate (m^3)	Rheological model (m^3)	Porosity model, DFN (m^3)	Porosity model, empirical (m^3)
1	165	203	–	170
2	270	241	–	328

The distribution between the values in Table 8-16 and Table 8-17 illustrates the anticipated uncertainty in estimating or calculating grouting quantities in a facility as large as the entire repository complex.

The results of the "empirical" porosity model in Table 8-16 are considered to be unreasonably high. The differences in the grouting quantities for the porosity models are directly proportional to the differences of the porosity values. One interpretation for the differences in the porosity values is that the empirical relation between conductivity and porosity /Brotzèn 1990/ is not developed for low conductivities. This approach can be used for conductivities in the range encountered in the zones in question, but not for low conductivities such as the current values in Rock domains A and M.

8.3.2 Discussion

The estimates and calculations of grouting quantities are very uncertain and are based on a number of assumptions and subjective assessments, which are of great importance for the forecast quantities. In addition, the planned facility is large and complex, which means that individual uncertainties may together be of great importance.

The practical experience of documented projects from similar conditions and requirements are limited. Due to the great depth and the in situ water flow and the pressure conditions, a wrong choice of grouting method technique and associated equipment could have more impact than a facility nearer the surface with "normal" conditions.

Despite all the uncertainties, a so-called "best" assessment has been made concerning grouting quantities. The most important uncertainty for all methods is in estimating the percent of the tunnels/rock caverns that will be grouted. The analytical methods contain more assumptions and uncertainties than the empirical estimates, so the overall assessment has in principle been based on the empirical estimates. The overall assessment is presented in Table 8-18 and Table 8-19.

Table 8-18. Overall assessment of grouting quantities, with minimum and maximum (rectangle distribution), for each repository component and sealing level.

Sealing level	Groutable proportion	Deposition tunnels (m^3)	Other tunnels/ rock caverns (m^3)	Crossings through zones (m^3)
1	Domain A: 5% Domain M: 2.5%	350–650	150–300	100–300
2	Domain A: 60% Domain M: 30%	3,000–6,000	1,000–2,000	200–400

Table 8-19. Overall assessment of hole-filling quantities, with minimum and maximum (rectangle distribution), for investigation holes in areas that are not grouted for respective repository component and sealing level.

Sealing level	Deposition tunnels (m^3)	Other tunnels/ rock caverns (m^3)
1	850–1,250	250–450
2	600–900	150–350

The grouting mix, should have a low pH, according to SKB's safety analysis, in order to avoid an excessively alkaline environment in the rock mass around the final repository. Consideration will be given to this by using a so-called low- pH recipe, pH < 11, which SKB is at present developing and testing. As a part of this study the volume/mass needed of the constituents of the low pH grouting was calculated based on Table 8-18 and Table 8-19. The results have been handed over to SKB's safety analysis.

9 Estimate of rock support requirements

9.1 Input data and assumptions

9.1.1 General

A preliminary estimate for the rock support in rock caverns, tunnels and shafts for the deep repository at a depth of 500 m has been made. The estimate is based on SKB reports /Barton 2003, SKB 2004a, 2006a, Martin 2005, Lanaro 2005/, Chapter 5 and external documents /BV Tunnel 2002, Norsk Concrete Association 2003, Barton 1974, Kaiser and Tanant 1997, Grimstad et al. 2002/.

There are different demands for rock support since the deposition tunnels will be in operation for at least 5 years, whereas the rest of the repository will be designed for a service life of at least 100 years /SKB 2004a/.

The estimate of the rock support requirements in the final repository with corresponding infrastructure is primarily based on the Q-value rock classification system /Grimstad et al. 2002/. In order to determine the Q-value, 6 different constituent parameters must first be determined in the Q-system. The values of the parameters are mainly determined from the logging of drill cores. This applies to the first four parameters: RQD (joint frequency value), J_n (number of joint sets), J_r, (joint roughness value) and J_a (joint filling and weathering). These parameters can be evaluated directly from the drill cores. The last two parameters in the Q-system, the joint water factor, J_w, and stress reduction factor, SRF, are estimated by using conductivity data from the SDM v 1.2 /SKB 2006a/ and the ratio between uniaxial compressive strength and in situ rock stresses.

9.1.2 Q-value

As basic input for determination of the Q-values and estimate of support requirements, the following relationships have been analysed:

- Distribution of the underground repository into the various rock domains A, D, M and B.
- Q-logging of drill cores.
- Initial rock stresses in Stress Domain I and their distribution.
- Uniaxial compressive strength and its distribution in different rock domains.
- Hydraulic conductivity and its variation with depth.
- Fracture sets orientation.
- Calculation of area to be supported in the different tunnel categories.
- Choice of support level and support types for the different tunnel categories in relation to the Q-value interval.
- Calculation of support quantities in the different tunnel categories.
- Environmental requirements and the use of materials for rock support.

In the SDM v 1.2, the Q-values for different boreholes are presented, see Figure 9-1, and the Q-values obtained for the rock domains, see Table 9-1. The graphical overview and table values give only the Q-value in logarithmic scale towards depth and the mean value per rock domain, respectively, the values of the constituent parameters that are the basic input for the Q-values are not presented.

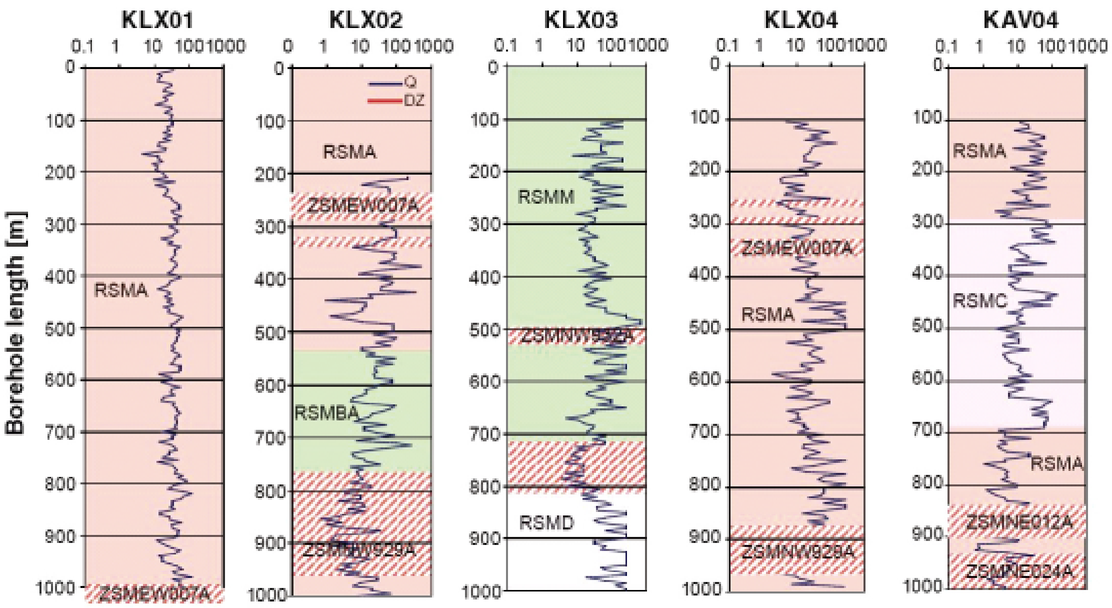


Figure 9-1. *Interpreted Q-values for 5 m sections along Boreholes KLX01-04 and KAV04 /SKB 2006a/. Note that the Q-values are presented in logarithmic scale.*

Table 9-1. Empirical rock mass classification, excluding the deformation zones, in the defined rock domains in accordance with /SKB 2006a/.

Index Based on 5 m sections of drill cores	Rock domain A Mean values (Most freq.)	Rock domain B Mean values (Most freq.)	Rock domain C Mean values (Most freq.)	Rock domain D Mean values (Most freq.)	Rock domain M Mean values (Most freq.)	Uncertainty in the mean estimates
Q (–)	45 (31)	23 (13)	27 (12)	155 (132)	82 (34)	± 3
RMR (–)	75	70	72	85	83	± 1

For the purpose of this work, it has been assumed that the parameters J_w and SRF have been assigned to 1.0 in the calculation of the Q-values that are shown in Figure 9-1 and Table 9-1. As a result of this, the Q-values are reduced when the values for J_w and SRF have been finally evaluated for the assessment of the rock support input data.

The only available documentation of the various parameters in the Q-system has been the Q-logging from borehole cores KSH01A and B at Simpevarp /Barton 2003/. Since the boreholes KSH01A and B have been drilled at Simpevarp, it is possible that the Q-values in Boreholes KSH01A and B are not necessarily representative for the corresponding hole depths and rock domains at Laxemar, even though the same rock domains exist. In the further evaluation of Q-values, the experiences from Boreholes KSH01A and B have been used.

Table 9-2 shows the mean values and most common Q-values, and the variation between typical minimum and maximum values for rock domains A and M in Laxemar /Lanaro 2005/. It is paradoxical that the typical min value for deformation zones is considerably higher than the typical min. for competent rock mass in Rock domain A, in this example. The lower values for Q-min in the competent rock mass can be explained by the presence of small deformation zones.

The Q-values for the deformation zones are only included in smaller parts of the transport tunnels and in parts of the shafts. Furthermore, the anticipated distribution of the parameters joint water (J_w) and stress reduction factor (SRF) will influence the Q-values.

Table 9-2. Estimated Q-values for competent rock mass and deformation zones in Rock Domains A and M in Laxemar /Lanaro 2005/.

	Competent rock mass			Deformation zones		
Rock domain	Typ. Q min	Mean/most common	Typ. Q max	Typ. Q min	Mean/most common	Typ. Q max
Roch domain A Laxemar	0.9	44.7/30.5	529.0	2.7	3.7/3.1	5.4
Rock domain M Laxemar	5.0	81.9/34.1	704.0	3.7	10.7/9.4	19.4

9.1.3 In situ stresses and rock strength

The entire Oskarshamn investigation area has been divided up into two parts with respect to the in situ stresses in the site description. The location of the final repository in Laxemar is planned within stress domain I. The in situ stresses for I are shown in Table 9-3.

As can be seen from the table, major variations in stress level may occur.

Table 9-4 shows the intact strength of respective rock types. Since the majority of the repository facility will be within Rock domains A (1/3), mainly Ävrö granite, and M (2/3), which is a mixture of domain A and D, mainly Quartz monozodiorite, with lenses and bands of gabbro. It can be anticipated that both the stresses and the compressive strength will vary significantly.

Table 9-3. Estimated in situ main stresses in Stress Domain I, where the Laxemar repository is planned to be located, /SKB 2006a/.

Parameters	σ_1	σ_2	σ_3
Mean stress magnitude, z = depth below ground surface (m)	0.058 x z + 5 MPa	0.027 x z MPa	0.014 x z + 3 MPa
Uncertainty, 100–600 m	± 30%	± 30%	± 30%
Spatial variation in rock domains	± 20%	± 20%	± 20%
Spatial variation in or close to deformation zones	± 50%	± 50%	± 50%

Table 9-4. Compressive strength, estimated rock mechanical strength for intact rock in the predominant rock domains /SKB 2006a/.

Parameters for intact rock (drill core scale)	Granite to Quartz monzodiorite (Ävrö granite) Mean/standard dev. Min – max (tunc.) uncertainty in mean	Quartz-monzonite to monzodiorite Mean/standard dev. Min – max (tunc.) uncertainty in mean	Fine-grained dioritoid Mean/standard dev. Min – max (tunc.) uncertainty in mean
Uniaxial compressive strength (UCS)	195 MPa/20 MPa	165 MPa/30 MPa	210 MPa/50 MPa
Minimum-Maximum	150–240 MPa ± 5 MPa	110–200 MPa ± 8 MPa	120–265 MPa ± 10 MPa
Crack initiation stress, σ_{ci}	0.50 × UCS ± 7 MPa	0.47 × UCS ± 9 MPa	0.47 × UCS ± 10 MPa

Since the largest and smallest main stresses are horizontal, the tangential stress in the vertical shafts will be $\sigma_\theta = \sigma_1 \times 3 – \sigma_3 \approx 92$ MPa. In the case of the horizontal tunnels, the relation between the largest main stress, σ_1, and the vertical stress σ_2. The maximum tangential stress in the tunnel roof is oriented at right angles to the largest horizontal in situ stress (approx. 135°) and will then be $\sigma_\theta = \sigma_1 \times 3 – \sigma_2 \approx 89$ MPa, while the minimum tangential stress will be $\sigma_\theta = \sigma_3 \times 3 – \sigma_2 \approx 17$ MPa and is parallel to the largest horizontal in situ stress (approx. 42°).

Since the final repository will mainly be located in Rock Domains A (Ävrö granite) and M (Ävrö + Quartz-monzonite with gabbro). The mean value for compressive strength will be chosen from Ävrö granite in order to determine the ratio between the imposed stress and compressive strength. This ratio gives an expression for the stress value SRF in the Q-system. However, due to major variations in both compressive strength and stress level, this ratio will vary significantly.

The calculated values for SRF are presented in Table 9-5. In this context, certain Q-values, less than 5% of all the values, have been slightly reduced compared to the values in Table 9-1 and Table 9-2. This refers to both domain A and M, and has given Q-values of less than 1.0 for 0–3% of the tunnel lengths, outside the deformation zones, and Q-values of less than 4.0 for 4–6% of the tunnels, outside the deformation zones.

SRF in relation to the ratio between tangential stress, σ_θ and compressive strength, σ_c is shown in Table 9-6. When the mean value for SRF is between 5 and 50, this means that a weak to moderate spalling will probably occur (stress-induced spalling), some time after the rock cavern has been excavated.

Table 9-5. Estimate of stress reduction factor, SRF, based on the ratio between tangential stress, σ_θ and compressive strength, σ_c.

		Ävrö granite	Quartz-monzonite (< 5%)
Compressive strength σ_c (MPa)	Variation	150–240	110–200
	Mean value	195	165
Tangential stress σ_θ (MPa)	Shaft/	≈ 90	≈ 90
	Tunnel // σ_1	≈ 17	≈ 17
σ_θ/σ_c	Variation (// σ_1/ $\perp$ σ_1)	0.60–0.38/0.11–0.07	0.81–0.45/0.15–0.09
	Mean (// σ_1/ $\perp$ σ_1)	0.46/0.09	0.55/0.10
SRF	Variation (// σ_1/ $\perp$ σ_1)	1–30/1	2–100/1
	Mean (// σ_1/ $\perp$ σ_1))	1.5/1	10/1

Table 9-6. Stress reduction factor, SRF, as a function of the ratio between tangential stress, σ_θ and compressive strength, σ_c in competent rock.

b) Component rock, rock stress problems	σ_c/σ_1	σ_θ/σ_1	SRF
Low stress, near surface, open joints,	> 200	< 0.01	2.5
Medium stress, favourable stress condition,	200–10	0.01–0.3	1
High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability	10–5	0.3–0.4	0.5–2
Moderate slabbing after > 1 hour in massive rock	5–3	0.5–0.65	5–50
Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1	50–200
Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1	200–400

Derek Martin states in his study /Martin 2005/ that "the mean uniaxial compressive strength for the Simpvarp and Laxemar sites, which is used to estimate the rock mass spalling strength, is considerably less than the one used for the Äspö diorite, and hence may be underestimated" and may thereby be underestimated in the SDM v 1.2. The distribution in rock mechanical strengths, see Table 9-4, has been considered by using variations in SRF. As can be seen from Table 9-5, it is Quartz-monzonite, which makes up less than 5% of the facility that has the highest values for SRF. Due to major variations in both stress level and compressive strength, there may also be local spalling in Ävrö granite.

The greatest risk of spalling is in the range of $0.4 < \sigma_\theta/\sigma_{ci} < 0.6$ with an expected spalling when the value is higher than 0.6. The values vary in the repository facility depending on the direction of the tunnels in relation to the highest main stress, σ_1, which in Laxemar is oriented N132°. In total, some 62% of the deposition tunnels are oriented at an unfavourable angle to the largest main stress, considering possible stress spalling. This spalling begins at a depth of 450 m where it is marginal /Martin 2005/. However, as mentioned earlier there is a considerable spread in the rock mechanical values, which means that local spalling can be anticipated. Furthermore, a risk of spalling can be expected at greater depth, but it can be controlled since the tunnels are for the most part oriented parallel to the highest main stress /Martin 2005/.

9.1.4 Fracture sets and J_w value

In the Laxemar area, some 5 fracture sets are registered: three global sub vertical sets that follow the regional lineament, one sub-horizontal and one sub-vertical fracture set. One of the fracture sets (Fracture Set B) has an unfavourable angle to the deposition tunnels – an angle of less than 8° to some 80% of the deposition tunnels, see Figure 9-2. This unfavourable angle in relation to the tunnel direction normally gives a reduced stability and a greater need for support, which also is taken in to notice in design task I for the estimate of rock support.

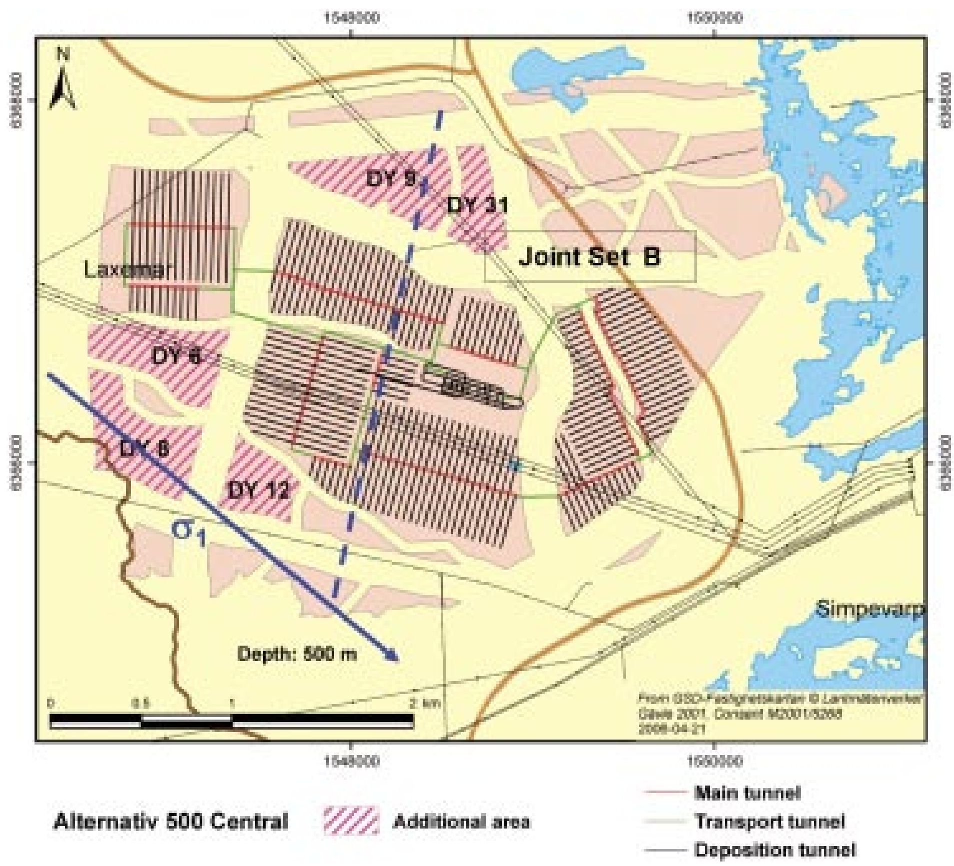


Figure 9-2. Basic layout for Laxemar. Highest main stress, σ_1 and Fracture Set B are marked on the map.

In order to achieve the most possible and realistic Q-value for the assessment of support quantities, the joint water value J_w should be evaluated. This is to avoid the use of the value $J_w = 1.0$ when water seepage of different magnitudes can be expected.

According to Figure 9-2, the conductivity in the rock mass varies between $10^{-7.0}$ and $10^{-9.5}$ m/s at a depth greater than "split" 300 m. Figure 9-3 shows a connection between the conductivity and the water seepage value J_w.

With background in Figure 9-2 and Figure 9-3, J_w is estimated to vary between 0.66–1.0.

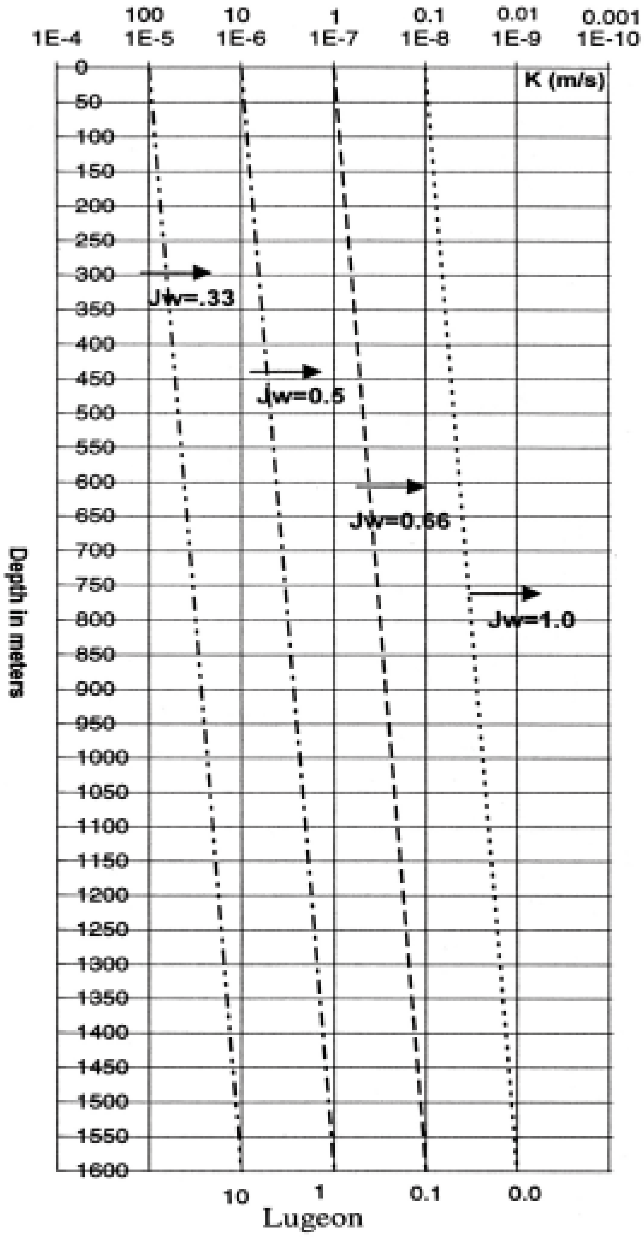


__Figure 9-3.__ Relation between J_w and measured conductivities and depth /Bashin et al. 1999/.

9.2 Execution

Based on the determined Q-values, the empirical ratio of the Q-system between the rock quality and the expected support is used for the estimate of rock support quantities; see Figure 9-4 with explanations in Figure 9-5. From the Q-system, the thickness and energy absorption for the shotcrete and the distance between the rock bolts is determined for each Q-interval along the axis for the Q-values, e.g. 0.1–0.4, 0.4–1.0, 1.0–4, 4–10, 10–40 etc.

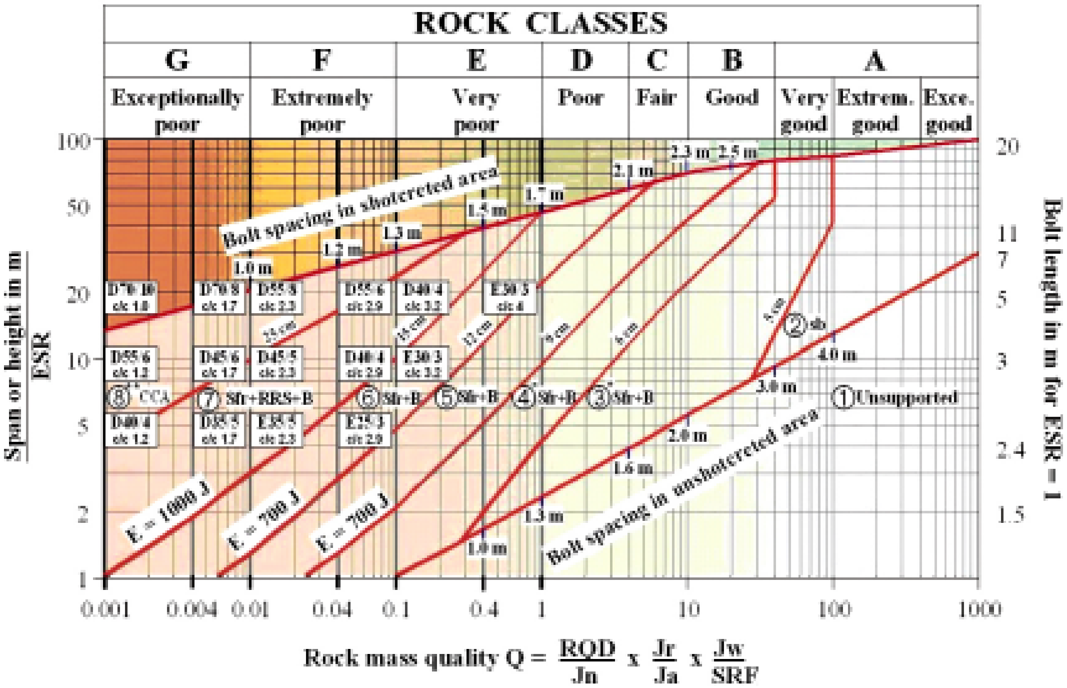


Figure 9-4. Rock mass classification according to the Q-system with appurtenant rock support categories. After /Grimstad et al. 2002/.

Rock mass quality $Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$

ROCK SUPPORT CATEGORIES
1) Unsupported
2) Spot bolting. **sb**
3) Systematic bolting and unreinforced shotcrete. 5-6 cm **B(+S)**
4) Fibre reinforced shotcrete and bolting. 6-9 cm. **Sfr+B**
5) Fibre reinforced shotcrete and bolting. 9-12 cm. **Sfr+B**
6) Fibre reinforced shotcrete and bolting. 12-15 cm. **Sfr+B**
7) Fibre reinforced shotcrete 15 cm+reinforced nbs of shotcrete and bolting. **Sfr+RRS+B**
8) Cast concrete lining. **CCA** or **Sfr+RRS+B**

E) Energy absorbtion in fibre reinforced shotcrete at 25 mm bending during plate testing.

D45 6
c/c 17

=RRS with 6 rock support bars in double layer in 45 cm thick ribs with centre c/c spacing 17 m. Each box corresponds to Q-values on the left hand side of the box (See text for explanation)

* in Figure 9-3) Up to 10 cm in large plans
** in Figure 9-4)Or **Sfr+RRS+B**

Figure 9-5. Explanations for the designations in the Q-graph in Figure 9-4.

In order to be able to estimate the rock support quantities for each tunnel category and shaft, the arch length of tunnel roof, abutments and wall heights are calculated for each category. Furthermore, consideration has been given to the area in the circle arch for the four different diameters for the shafts when estimates are made of the rock support quantities, see Table 9-7. All arch lengths are reduced in the calculations with increasing thickness of shotcrete so that the area will be constantly correct.

In order to get an overview of the tunnel lengths for the different tunnel categories combined with the different rock domains, an account is given in Table 9-8.

For calculations of rock support quantities, based on the respective Q-value, the specified Q-values are used directly in the roof and abutments, whereas they are reduced in the walls in that the Q-values have been multiplied by a predetermined factor for different Q-intervals in accordance with the description of the Q system. These factors are reproduced in Table 9-9.

Table 9-7. Overview of cross-section and area to be reinforced in the different rock caverns.

Tunnel type	Cross-section Area (m^2)	Arch length in cross-section (m) Roof	Wall height
Main tunnels	66.0	11.2	5.6
Transport tunnels	36.0	8.4	5.6
Deposition tunnels	25.0	6.0	4.4
Ramps	30.0	6.6	4.8
Rock caverns	130.0	15.0	8.5
Shafts Ø 2.5 m	4.9	7.9	
Shafts Ø 3 m	7.0	9.4	
Shafts Ø 3.5 m	9.6	11.0	
Shafts Ø 5.5 m	23.0	17.3	

Table 9-8. Amount of tunnel metres in the different rock domains. Only the transport tunnels and some of the shafts are expected to pass through the deformation zones.

Rock domain	A (m)	M (m)	D/M (m)	B/M (m)	B/D/M (m)	Zone (m)	Total length (m)
Main tunnels	2,400	3,500	225	0	0	0	6,205
Transport tunnels	985	3,370	335	0	0	280	4,970
Deposition tunnels	19,085	36,000	3,400	430	560	0	59,475
Ramps	1,350	4,000				50	5,400
Rock caverns	525	1,575					2,100
Shafts	500	2,000	500			100	3,060

Table 9-9. Adjustment of the Q-values in the walls in relation to the observed Q-value for roof and abutments.

In rock masses of good quality	Q > 10	The actual Q-values are multiplied by 5.
For rock masses of intermediate quality	0.1 < Q < 10	The actual Q-value is multiplied by 2.5 (In cases of high stresses the actual Q-value is used).
For rock masses of poor quality	Q < 0.1	The actual Q-value is used.

In order to obtain a Q-value as accurate as possible, a 10% rebound effect on the shotcrete is added in the rock support estimate.

Furthermore, consideration has been given to the description of rough rock surface after blasting by using a roughness factor. This roughness factor or coarseness has been varied with respect to the thickness of the shotcrete since a thicker layer evens out the surface. For calculation purposes, a correction factor has been chosen for mean roughness. This is presented in Table 9-10.

Tables have been created with rock support quantity per metre of tunnel or shaft for each tunnel category combined with each interval in the Q-system. The rock support quantities are base on the Q-system, such as thickness of shotcrete, average bolt spacing and thickness of reinforced ribs of shotcrete and their centre-to-centre distance. The shotcrete thickness is recalculated into m^3 of shotcrete per running metre, divided into three different categories for energy absorption, E500, E700 and E1000 /Norwegian Concrete Association 2003/. Corresponding standards have also been used in several EU countries.

Recalculation into m^3 concrete has been done for shotcreted and steel-reinforced arches, and possible concrete lining. The bolt distance has been converted into number of bolts per tunnel metre. The amount of wire mesh is specified in m^2 for the deposition tunnels, where shotcrete will be replaced by wire mesh support in the lower rock support categories.

These table values have been inserted into the calculation sheet for each tunnel category and shaft, so that the total rock support quantity for each category has been given. The rock support quantities have been specified in m^3 of concrete, number of bolts and m^2 of wire mesh per running metre of tunnel/shaft for each Q-interval. The amounts are summarized for all Q-values for the entire length within the different tunnel categories.

When all of the tunnel categories have been analysed individually, all rock support has finally been added together for each type of rock support, irrespective of tunnel category. The total sum of all rock support is presented in Table 9-11. Furthermore, quantity calculations have been made for the volume and weight of bolts, divided into quantities of different lengths and diameters of bolts, as well as for volume and weight of shotcrete and steel fibre in the concrete. It has been assumed that only Ø25 mm bolts will be used, except in the deposition tunnels that have a short lifetime and in shafts with diameters from 2.5 to 3.5 m, where the bolt length will also be shorter than normal. In the calculations, all bolts are grouted and therefore washers mounted on the bolts have not been included.

The calculated quantities are based on the updated Q-system, with higher demands for safety in the relatively good rock conditions as per current standards.

Table 9-10. Correction factor for coarseness in the tunnel periphery in blasted tunnels.

Thickness of shotcrete (cm)	Correction factor for mean roughness
$t \leq 6$	1.5
$t \leq 10$	1.4
$t \leq 14$	1.35
$t \leq 20$	1.3
$t \leq 24$	1.25
$t > 24$	1.2

Table 9-11. Total rock support for all tunnel categories.

Type of tunnel or shaft	Length (m)	S (m^3) (RRS)	Sfr E500 (m^3)	Sfr E700 (m^3)	Sfr E1000 (m^3)	Bolts (pcs)	Spiling bolts (pcs)	Wire mesh (m^2)
Main tunnels	6,125	0	6,053	0	0	16,408		0
Transport tunnels	4,970	16	3,864	303	0	11,682	270	0
Deposition tunnels	59,475	0	0	975	55	125,485	200	281,950
Ramps	5,400	0	878	18	0	9,598	0	0
Rock caverns	2,100	0	3,115	192		8,359	0	0
Shafts Ø 3 m	1,020	4	174	15	0	1,542	0	0
Shafts Ø 2.5 m	510	0	45	0	0	641	0	0
Shafts Ø 3.5 m	510	0	175	7	0	878	0	0
Shafts Ø 5.5 m	1,020	0	933	22	0	2,797	0	0
Total		**20**	**15,237**	**1,532**	**55**	**177,390**	**470**	**281,950**

Table 9-12. Volume and weight for rock bolts.

Type of tunnel or shaft	Bolts	Bolt length (m)	Total length (m)	Diameter (m)	Volume (m^3)	Weight (tonnes)
Main tunnels	1,6408	3	49,224	0.025	24.2	188.5
Transport tunnels	11,682	2.4	28,037	0.025	13.8	107.4
Deposition tunnels	125,485	2.4	301,164	0.02	94.6	737.6
Ramps	9,598	2.4	23,035	0.025	11.3	88.2
Rock caverns	8,359	4	33,436	0.025	16.4	128.1
Shafts Ø 3 m	1,542	1.5	2,313	0.02	0.7	5.7
Shafts Ø 2.5 m	641	1.5	961	0.02	0.3	2.4
Shafts Ø 3.5 m	878	1.5	1,317	0.02	0.4	3.2
Shafts Ø 5.5 m	2,797	2.4	6,713	0.025	3.3	25.7
Total	**183,240**		**461,476**		**165**	**1,287**

In Table 9-11, a total of 282,200 m^2 of wire mesh support has been calculated, which refers to the deposition tunnels where there will be as little shotcreting as possible and where the operation period has been estimated to last at least 5 years. Of the total quantity of wire mesh support, 111,000 m^2 have been calculated for the purpose of rock support in the area with Q-values between 10 and 40. This is simply in order to prevent smaller outfalling rocks, during the period in question and is completely irrespective of the stress conditions. There is a somewhat greater need for rock support against stress-induced spalling in the deposition tunnels than in the other tunnels. As a consequence of an unfavourable orientation and in some cases due consideration to the proximity of weakness zones, where there is a possibility of uneven stress conditions.

Consideration has also been given to the fact that approximately 10–15% of the deposition tunnels may be influenced by small to moderate spalling. In addition, some 80% of the deposition tunnels are oriented with an angle of less than 8° to Joint Set B. As a consequence of this, consideration has also been given in this context to shotcrete in energy absorption category E700, at lower Q-values than 0.4. This value has also been assigned as the criterion between wire mesh rock support and shotcrete. At lower Q-values there could be a continued development of rock fall with time, so the wire meshes will probably have to be emptied of stones. When using wire meshes, instead of shotcrete, the amount of rock bolt support will also increase.

In Table 9-13, the volume and weight of cement mass (bolt mortar) is estimated. It is assumed that a normal 45 mm large hole is drilled. The space between Ø25 and Ø20 mm bolts and the borehole walls has been calculated as the volume for bolt mortar. The weight has been calculated on the basis that a ready mixed mortar has a density of 1.8 tonne/m^3, which is equivalent to a water/cement ratio of 0.4. For higher water/cement ratios, both the density and the compressive strength of the mortar will be slightly decreased.

Table 9-14 shows the quantity of wire mesh rock support in the deposition tunnels. Three types of wire meshes have been inserted into the table. The most likely answer is a fine-mesh net in order to be able to catch any fallout from stress-induced spalling. Therefore, wire mesh support with a mesh size of 6x8 cm and a wire thickness of 2.7 mm has been chosen. Plastic-coated wire mesh with the same mesh size has also been inserted.

The weight of steel fibre in Table 9-15 is based on the fact that some of the best fibres on the market are used with the aim of giving good energy absorption. The number of kg of steel fibre that is specified here is based on laboratory tests conducted in Norway and Australia.

Table 9-13. Volume and weight of bolt mortar. Volume and weight of bolts with 20 and 25 mm diameters.

Bolt diameter (mm)	Total length of bolts (m)	Volume of bolts (m³)	Weight of bolts (tonnes)	Area of 45 mm bore-hole m²)	Volume of 45 mm borehole (m³)	Volume of bolt mortar (m³)	Weight of bolt mortar (tonnes)
25	140,445	69	538	0.00159	223	154	278
20	305,755	96	749	0.00159	486	390	702
					Total	**544**	**980**

Table 9-14. Weight of wire meshes to be used in the deposition tunnels.

Steel netting Area (m²)	Mesh size (cm)	Wire thickness (mm)	Weight (kg/m²)	Total weight (kg)
286,750	**6x8**	**2.7***	**1.84**	**518,788**
286,750	8x10	3*	1.7	479,315
286,750	6x8	2.2**	1.5	422,925

* Hot-dip galvanised.

** Hot-dip galvanised with a 32 mm coating of plastic.

Table 9-15. Volume and weight of shotcrete divided according to energy absorption categories.

Tunnel type	S (RRS)		Sfr E500 18 kg fibre per m³ concrete			Sfr E700 25 kg fibre per m³ concrete		
	(m³)	Weight concrete (t)*	(m³)	Weight concrete (t)*	Weight fibre (t)	(m³)	Weight concrete (t)*	Weight fibre (t)
Main tunnels	0	0	6,053	14,346	109	0	0	0
Transport tunnels	16	38	3,864	9,158	70	303	718	7.6
Deposition tunnels	0	0	0	0	0	975	2,311	24.4
Ramps	0	0	878	2,081	16	18	43	0
Rock caverns	0	0	3,115	7,383	56	192	455	4.8
Shafts Ø 3 m	4	9	174	413	3	15	36	0
Shafts Ø 2.5 m	0	0	45	107	1	0	0	0
Shafts Ø 3.5 m	0	0	175	415	3	7	17	0
Shafts Ø 5.5 m	0	0	933	2,211	17	22	52	0.6
Total	20	47	15,237	36,113	274	1,532	3,631	38

* Density concrete = 2.37 t/m^3.

In Table 9-16, the total volume and weight are presented for shotcrete and steel fibres, excluding spillage in the deposition tunnels. In the table, the quantity of steel fibres is calculated by two ways first from the functional requirements based on standards in the Norwegian Handbook of Concrete No 7 and the other on the demand for 70 kg of fibre per m^3 of shotcrete, as per the proposed SKB recipe with a low pH.

9.3 Environmental requirements

The demand for the least possible pollution for the groundwater in the deep-level repository, has led to strong environmental requirements on all installed rock support. Attempts will be made to achieve the lowest possible dissolution of foreign chemicals in the bedrock and the groundwater.

In order to prevent the dissolution of steel from leaching out into the groundwater and to take full advantage of the rock support capacity offered by rock bolts over a long time, it is important to have good corrosion protection. In order to achieve the best possible result, all the rock bolts will be properly grouted with mortar. If local spalling rock should occur, which normally gives rise to significant deformations in the tunnel periphery, grouted rock bolts will have a bad effect compared with end-anchored rock bolts with a good elongation capacity. If end-anchored rock bolts are chosen, they should be hot-dip galvanised and epoxy-coated. If necessary, the end-anchored bolts could be grouted after the deformation process has ceased. In such case, the type of bolts used must be fitted with a grouting hose that is cast into the shotcrete. This could be of interest in the deposition tunnels.

BTH rock bolts, except in the deposition tunnels, will meet the requirements of Corrosion Class R3, which is described by /BV Tunnel 2002/, which requires hot-dip galvanisation and > 80µ epoxy coating for fully embedded rock bolts.

If the bolts are provided with washers, they will be covered with an at least 3 cm-thick layer of shotcrete. In the calculations, only grouted rock bolts are included. Therefore, washers are not included in the total sum of steel weight.

In SKB's recipe with a low PH, it is proposed that 70 kg of steel fibre be used per m^3 of shotcrete. According to Swedish standard specifications at least 50 kg of fibres will be added per m^3 of shotcrete.

It is not necessary to use 70 kg of steel fibres/m^3 if we use fibre types that give good ductility. Functional requirements should be set rather than quantity requirements. For E500, which has been assessed to be the predominant energy absorption category for the deep-level repository, approximately 18 kg of good quality steel fibre will be used per m^3, i.e. of the Dramix RC65/35-BN type or similar should be sufficient.

Table 9-16. Volume and weight of concrete and steel fibre in shotcrete when a deduction has been made from the deposition tunnels for spillage.

Total quantity of material in shotcrete	(m^3)	(tonne)
Concrete	16,844	39,921
Steel fibre*	40	314
Steel fibre**	161	1,262

* Fibre quantity as per functional requirement.

** 70 kg fibre/m^3.

The concrete, i.e. both the shotcrete and the bolt mortar, will have a low pH, according to SKB's safety analysis, in order to avoid an excessively alkaline environment in the rock mass around the final repository. Consideration will be given to this by using a so-called low- pH recipe, pH < 11, which SKB is at present developing and testing.

9.4 Discussion

As pointed out previously, the quantity of rock support will depend on both the extent of stress-induced spalling fractures in the facility, even though it has in general been assumed to be of a magnitude less than 5% of the overall tunnel length. If no form of stress-induced spalling occurs, the quantity of shotcrete could probably be significantly reduced, whereas the number of bolts would be reduced to a somewhat lesser extent.

For the purpose of stress relief, it is the fibre rock support in the shotcrete that is absolutely decisive for the rock support effect. However, it is not necessary to add 70 kg of steel fibre/m^3 of shotcrete. On the basis of functional requirements, it is enough with 18 kg/m^3 of shotcrete, in those places where there are no traces of deformations or stress changes, and 25 kg/m^3, where moderate deformations or stress changes can be expected. The total estimated rock support is 177,000 rock bolts, 15,200 m^3 of Shotcrete E500, 1,500 m^3 of Shotcrete E700, 314 tonnes of steel fibre, 282,000 m^2 of wire mesh (520 tonne) and 980 tonne/ 540 m^3 of concrete mortar for embedding bolts. The weight and volume of the rock bolts is estimated to be 1,287 tonne and 165 m^3 of steel.

In the deposition tunnels, there is an estimated demand for 282,000 m^2 of wire mesh. Since the occasional fallout of small rocks is accepted during the period, an approximately 110,000 m^2 of wire mesh can probably be removed. However, the stability above the deposition holes should be reconsidered. Furthermore, there will probably be no need for concrete lining.

The use of steel-fibre-reinforced shotcrete in relatively good rock categories, Q > 10, is a result of modern rock support philosophy with a high demand for safety in public spaces (road and railway tunnels), compared with older rock support philosophies where a certain acceptance of minor stone fallouts and periodical maintenance clearing is permitted.

If occasional fallout of small rocks were accepted (except above the deposition holes) over a longer period of time, or periodic clearing were to be carried out, the quantity of fibre-reinforced shotcrete and wire mesh rock support could probably be reduced considerably, i.e. by approximately 40–50%. The number of bolts would not be affected by the same conditions, but if shotcrete and wire mesh support are not applied, it usually means shorter distance between the bolts.

10 Technical risk assessment

10.1 Critical issues

A technical risk assessment has been made of the preconditions that have served as a basis for the designed layout alternatives. The risk assessment has above all concerned issues involving the design conditions and the extent of the repository. The area analysed corresponds to the entire area that is subject to investigations pending the construction of a possible final repository in Laxemar. Only one depth has been studied, 500 m, which has been considered the most suitable to accommodate a repository, see Chapter 5.

The analysis has been conducted in three main stages:

1. Analysis of the impact of different parameters on whether or not there is accommodation for the repository, by means of simulations according to the Monte Carlo method.
2. Sensitivity analysis of the results of Stage 1 in order to find out which parameters having the greatest importance as to whether or not there will be enough accommodation for the repository.
3. Analysis with the purpose of "testing and evaluating the design methods", with a focus on events that affect the layout or design.

The principle of the Monte Carlo simulation has in this case been to compare the deposition area that needs to be utilised with the entire area in Laxemar, minus the calculated loss areas. Loss areas are areas that consist of deformation zones and their respect distance or margin for excavation, space for the central area and communication tunnels (main or transport tunnels) and spaces between different limit lines that are too small to meet the demands for the smallest deposition area. The parameters that are considered in the simulations are:

1. Existence and length of deformation zones
2. The dip of the deformation zones.
3. Respect distance and margin for excavation of the deformation zones.
4. Distance between deposition holes (as a consequence of thermal properties).
5. Loss of deposition holes (as a consequence of fractures, hydrogeological and rock mechanical properties).

The sensitivity analysis has been carried out on the basis of the Monte Carlo simulation, where the mutual uncertainties of the constituent parameters are ranked. Further, the consequence of a reduced water seepage criterion to the deposition holes is considered, see Section 4.4.

In order to test and evaluate the design methods, events have been identified that could cause undesirable damage and consequences for the design. The evaluation covers design methods in Chapters 3 to 9. The analysis does not include events that are linked to the construction and operation stages or to the stage after encapsulation. Preventive measures that are connected to the design stage have been subsequently proposed.

10.2 Approach for risk assessment

10.2.1 Introduction

The method that has been used to estimate the likelihood of the repository having sufficient space means that the requisite deposition area ($A_{deposition}$) is compared with the total area of the Laxemar area (A_{total}) minus the calculated loss areas. Those spaces that are defined as loss areas within the area are:

- Any deformation zones and their respect distance or margin for excavation from the deposition area ($A_{zone\ areas}$).
- Areas between deformation zones and limit lines that do not satisfy the requirements for the minimum requisite deposition area, i.e. holding at least five deposition tunnels with a minimum length of 100 m ($A_{small\ areas}$).
- Area for the central area ($A_{central}$).
- Areas for main tunnels and transport tunnels, excluding those tunnel sections that are intersected by deformation zones. (A_{tunnel}).
- Areas in the deposition areas, shortened DA in Section 3.2, that could not be used in the layout design, caused by geometrical problems (A_{geom}).

Mathematical conditions

The mathematical condition for the calculations and simulation are expressed as follows:

$$A_{deposition} < A_{total} - A_{zone\ areas} - A_{small\ areas} - A_{central} - A_{tunnel} - A_{geom} \qquad \text{Equation 10-1}$$

In the Monte Carlo simulation, a separation around zero is obtained if the expression is rewritten as follows:

$$0 < A_{total} - A_{zone\ areas} - A_{small\ areas} - A_{central} - A_{tunnel} - A_{deposition} - A_{geom} \qquad \text{Equation 10-2}$$

The calculations are repeated many times through so-called Monte Carlo simulation in which the calculation parameters are varied at random on the basis of set divisions. As long as Equation 10-2 is satisfied, there is sufficient space for the repository.

10.2.2 Description of calculated areas

In this section, a description is given of the areas in question and how they are treated in the simulation

Requisite deposition area

The smallest areas needed in order to meet the demand for deposition can be calculated as:

$$A_{deposition} = \frac{N \cdot A_S}{(1-K)} \qquad \text{Equation 10-3}$$

where N is the number of canisters that are to be stored, A_s, is the required specific area per deposition hole and K is the proportion of loss. N is a constant and is assumed to be 6,000. A_s is the area that each individual deposition hole requires in order to satisfy the thermal gradient from the canister and into the rock mass, i.e. the product of the distance between the deposition holes (C_H) and the distance between the deposition tunnels (C_T):

$$A_S = C_H \cdot C_T \qquad \text{Equation 10-4}$$

The distance between the deposition tunnels, (C_T), is given as a constant of 40 m.

The hole distance (c/c) between the deposition holes (C_H) varies depending on the thermal conductivity of the rock domains. In Section 4.2, an analysis is made of the variation in hole distance as a consequence of uncertainties in the determination of the thermal conductivity. In Table 10-1, a presentation is given of the mean distance arrived at, the largest and smallest distance for the respective subarea in the basic layout.

For the purpose of the Monte Carlo simulation, the weighted hole distance between the subareas has been used. A list of the weighted minimum, mean and maximum values is presented in Table 10-2.

The Monte Carlo simulation use a triangular distribution with the minimum and maximum values that are equivalent to the smallest weighted min and the largest weighted max value according to Table 10-2, i.e. 6.27 and 8.57 m respectively. The likeliest value is the weighted mean distance for alternative 2, i.e. 7.77 m, since this alternative applies as the main alternative in the interpretation in the SDM v 1.2.

In addition to hole distance, the loss of deposition holes is of importance for the requisite size of the deposition area. The loss is described by the parameter *K*, which is dependent on four factors: elongated fractures/fracture zones, water seepage, wedge breakout and the spalling rock phenomenon. The four factors are described in Section 4.4. The parameter *K* is described with a triangular distribution where the most probable value is 20% and the min and max values are 15 and 30% respectively, see Section 4.4 and Chapter 5. Here, the basic design value, 10.0 l/min, for water seepage has been used.

Available total area in Laxemar

The available total area, A_{total}, for repository depth 500 m is limited in Laxemar by the deformation zones ZSMNS001C, ZSMEW007A and ZSMNE005A and by the national interest for final repository deep, see Chapter 3.

In order to investigate how the total area is affected as a consequence of changes in the dip of the limiting deformation zones, a simplified area has been assumed in which the boundaries are represented by straight lines, see Figure 10-1.

Table 10-1. Hole distance within the three subareas "south", "central and west" and "north and east".

Subarea within Laxemar	Total deposition area* (km²)	Mean distance (m)	Min. hole distance (m)	Max. hole distance (m)
South	~ 0.71	7.6	5.9	8.2
Central and west, Alt. 1**	~ 2.85	7.4	7.0	7.8
Central and west, Alt 2**	~ 2.85	8.1	6.7	8.7
North and East	~ 1.42	7.2	5.6	8.5

* The areas are illustrated in Chapter 5.

** The site description /Sundberg et al. 2006/ shows two alternative interpretations of the thermal conductivity in rock domain M (corresponding to subarea "central and west"). For the basic layout, alt. 1 is chosen.

Table 10-2. Results of the weighted hole distances for the min, mean and max hole distance.

Weighted hole distance	Min (m)	Mean (m)	Max (m)
With Alt 1	6.44	7.37	8.06
With Alt 2	6.27	7.77	8.57

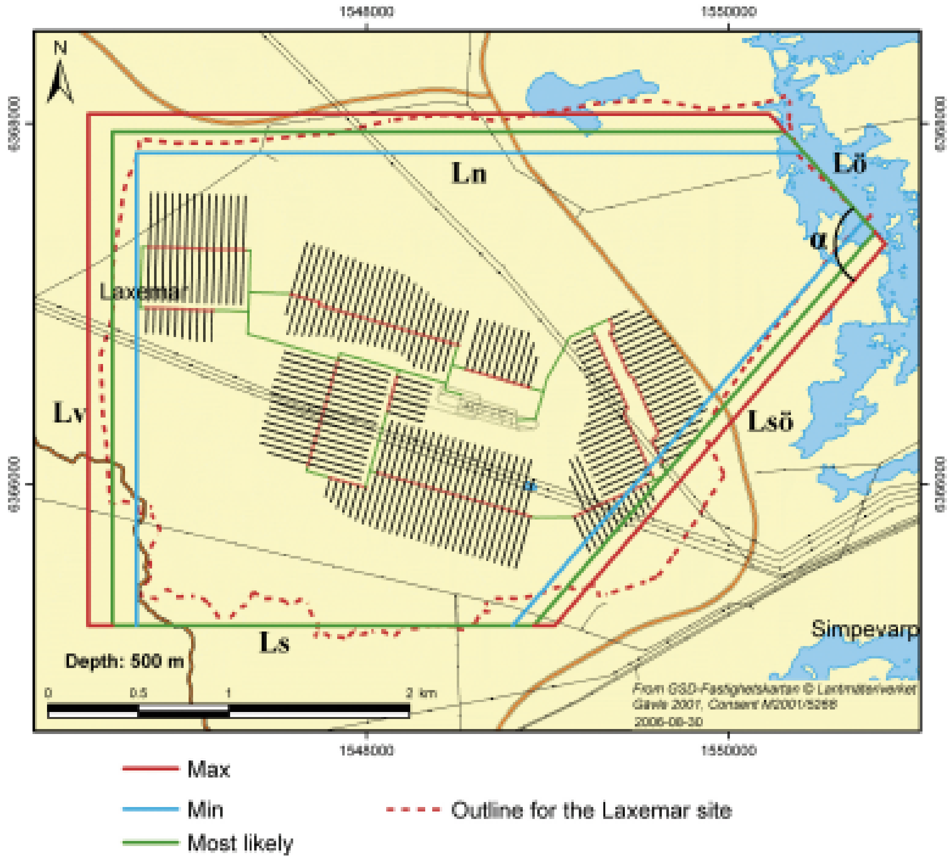


Figure 10-1. *Simplified boundaries for the total area with max, min and most likely area shown along with the angle α.*

The boundaries are:

- In the south: "national interest for deep repository" (Ls).
- In the west: deformation zone ZSMNS001C (Lv).
- In the north: deformation zone: ZSMEW002A (Ln).
- In the east: deformation zone ZSMNE005A (Lsö) and "national interest for deep repository" (Lö).

The boundaries (Lv, Ln and Lsö) consist of three deformation zones with a dip uncertainty, i.e. the positions of the lines at the storage depth are not locked, see Table 10-3.

The total simulation area, i.e. A_{total}, is a pentagon, see Figure 10-1, and can be calculated with the following equation:

$$A_{total} = \frac{(Ln \cdot Lv)}{2} + \frac{(Ls \cdot Lv)}{2} + \frac{(Ls\ddot{o} \cdot L\ddot{o} \cdot \sin\alpha)}{2} \qquad \text{Equation 10-5}$$

where α is the constant angle between line Lsö and Lö, which is measured at 97 degrees.

Table 10-3. The dip of the limiting deformation zones.

Deformation zone	Dip in basic layout	Dip interval
NS001C	90°	± 15°
EW002A	65° to the south	± 10°
NE005A	90°	± 10°

The distribution of the boundaries has been assumed to be triangular. The min and max values are obtained by alternating the dip of the limiting deformation zones. The most probable is the length that is obtained from the basic layout.

Deformation zones and their extent

Within the boundaries of the repository there are a number of deterministic deformation zones. The total area of these zones, including respect distance or margin for excavation, is referred to as $A_{zone\ areas}$.

The deterministically interpreted deformation zones are divided into three classes: those with a high degree of confidence, those with a medium degree of confidence and those with a low degree of confidence. For Monte Carlo simulation, it has been assumed that those with a high degree of confidence are likely to exist at 95% probability at the repository depth 500 m. The corresponding assumption for those with a medium degree of confidence is 70% and for low level of confidence 20%.

Deformation zones with a length of 3 km or more are given a respect distance (RD), while zones shorter than 3 km are given a margin for excavation (MFE). Since the length of the zones is associated with a certain degree of uncertainty, there is also a degree of uncertainty whether the RD or MFE should be assigned in the layout. The principle for assessment of whether RD or MFE shall apply has been:

- For a length < 2 km the probability for RD/MFE distance is 5/95%.
- For a length of 2–2.7 km: 25/75%.
- For a length of 2.7–3.3 km: 50/50%.
- For a length of 3.3–4 km: 75/25%.
- For a length of > 4 km: 95/5%.

To clarify the uncertainty, which exists in connection with the length of the deformation zones, was not included in the Monte Carlo simulation. The length was regarded as constant provided that the zone exists. It was only to decide whether RD or MFE should apply the rules above was used.

In order to calculate the effective area of the available deposition area, the following principles were applied:

- If the deformation zone were found to exist, its length was assumed to be given.
- The RD was set at 100 m. In those cases with RD, the width of the deformation zone was estimated to be 2×RD.
- For zones with a defined width, the MFE was set at 20 m. For zones with an undefined width (assumed as lines) the MFE was set at 30 m. In those cases with a MFE, the width was calculated as the width of the zone + the 2×MFE.

$A_{zone\ areas}$ was calculated as the sum of the areas of the deformation zones. In order not to calculate overlapping zones several times, the area of each zone was reduced by the area of crossing zones. Thus, no intersected area was calculated more than once for any of the crossing deformation zones.

Each calculation of $A_{zone\ areas}$ in the simulation can be summarised in the following calculation steps:

1) Does the zone in question exist? If so – what is its width and length? Is there a respect distance or margin for excavation?
2) Does the zone intersect with any other zone or zones? If so – what is its/their width and angle of intersection?

3) Calculate the area of the zone in question as the difference between its own area and the intersected area. If the zone does not exist, the area is set at 0.

4) Repeat steps 1 to 3 for all zones and add the areas together to obtain $A_{zone\ areas}$.

Small areas

$A_{small\ areas}$ is the sum of the deposition areas that do not fulfil the requirements that each individual deposition space shall contain at least five deposition tunnels with a min length of 100 m.

In the analysis, areas smaller than 0.025 km^2 in size was determined to be too small for deposition. To obtain a triangular distribution in the Monte Carlo simulation, the total sum of all small areas was divided between a minimum area, a most probable area and a maximum area. The minimum area was obtained on the assumption that only deformation zones with a high degree of confidence exist, which gives a small number of small areas. The max value was obtained when all deformation zones were assumed to exist, i.e. those with a high, medium and low degree of confidence, which consequently gives a higher number of small areas. The most probable total area of all the small areas was assumed to be the one that is received in the basic layout. Total areas obtained for min, probable and max are presented in Table 10-4.

Area for central area, main tunnels and transport tunnels

The area for the central area ($A_{central}$) and the area for transport and main tunnels (A_{tunnel}) is constant. The area of the central area is 0.228 km^2, which includes the safety area for the central area.

The area for main tunnels and transport tunnels is based on the basic layout, excluding tunnel sections intersected by deformation zones, and is in total 0.0867 km^2. Smaller areas such as at the beginning and at the end of the deposition tunnels, have been ignored (see Figure 4-2 and Figure 4-3).

Area for geometrical problems

The entire part of the deposition area can not be used because they are too narrow or are of irregular shape or to meet the demands of distances between tunnels and so on. How much of a deposition area that could not be used will of course differ for each area. For a large size area of regular shape, the part that will not be usable will be small whereas the opposite will be true for a small area of irregular shape. To differentiate this for every available area would not be feasible. Instead an approximation of the percentage of lost area due to geometry was made. All the deposition areas that had been used for any of the layout alternatives (see Figure 5-4 to Figure 5-7) were looked at, and an estimate of "rest area" was made (Figure 10-2). Based on this study, it was estimated that as much as 25% of the total area would be lost due to geometrical problems.

Table 10-4. Min value, most likely and max value used in triangular distribution in the Monte Carlo simulation for small areas.

Assumption	Total area (km^2)
Only zones with a high degree of confidence exist	0.012
Zones with a high and medium degree of confidence exist	0.114
All zones (high, medium and low) exist	0.152

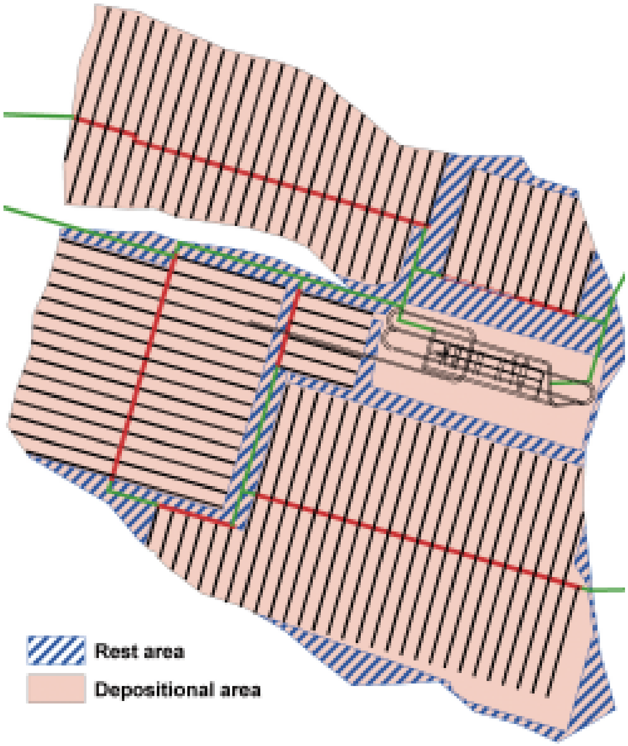


Figure 10-2. *Example of how the part of a deposition area that could not be used for deposition.*

This area was defined as A_{geom} and was calculated as a fraction of the total area that is available for the central area, tunnels and deposition holes:

$A_{geom} = k_{geom} \times (A_{total} - A_{zone\ areas} - A_{small\ areas})$

k_{geom} was defined by a triangular distribution with minimum value of 0.20, a maximum value of 0.30 and a most likely value of 0.25 due to the uncertainty of the study.

10.3 Results

10.3.1 Results of the Monte Carlo simulation

The Monte Carlo simulation was carried out using the software Crystal Ball. 10,000 calculations were performed for all equations. The main results according to Equation 10-2 are summarised in Table 10-5. In Figure 10-3, the final results of the simulation are presented and Figure 10-5 to Figure 10-7 show various partial results.

Table 10-5 and the line "Reserve area" show that the repository can be accommodated with sufficient space, i.e. the reserve area is on mean 2.38 km^2 when all conceivable loss area in the Laxemar area are included. None of the 10,000 simulated cases gave a negative reserve area, i.e. there is accommodation for the repository in 100% of the cases.

Figure 10-3 shows the calculated reserve area following a compilation of all simulations. The graph shows the relationship between calculated reserve area and the probability of the calculated reserve area. If the probability is 0.05, it means that 5% of the calculations (i.e. 500 of 10,000 simulations) gave the reserve area in question. As long as the reserve area is positive, there is enough room for the repository. The graph also shows the cumulative distribution function (CDF), which illustrates the probability that the reserve area is below a certain value. None of the calculations gave a negative reserve area, which means that there is enough accommodation for the repository in 100% of the calculation cases.

Table 10-5. Summary of the results of the Monte Carlo simulation for different parameters.

Parameter	Unit	Mean	Median	Lowest value	Highest value
Reserve area	km^2	2.38	2.37	0.92	4.00
A_s*	m^2	301	303	251	343
$A_{deposition}$	km^2	2.31	2.32	1.80	2.88
A_{total}	km^2	9.34	9.34	8.39	10.4
$A_{zone\ areas}$	km^2	2.57	2.58	1.01	3.94
$A_{small\ areas}$	km^2	0.093	0.096	0.013	0.152
$A_{central}$+ A_{tunnel}	km^2	0.315	0.315	0.315	0.315
A_{geom}	km^2	1.67	1.66	1.13	2.40

* Specific area of deposition holes.

Figure 10-4 shows the total deposition area that is required in order to accommodate 6,000 canisters.

According to Table 10-5 and Figure 10-4 the maximum area required to accommodate the repository is 2.88 km². The chosen "basic layout" from Chapter 5 is situated within four different deposition areas DA05, DA11, DA26 and D27 (see chapter 3 and 5). The total area of these areas is 3.42 km² (see Table 3-5 and Table 3-6) at the depth of 500 m. Thus these four areas offers sufficient space for the repository and further investigations of rock mass properties and deformation zones can be concentrated to these deposition areas.

Figure 10-5 shows the results from calculation of the total available area. The graph is a compilation of all simulations and shows the probability of different sizes of the total available area.

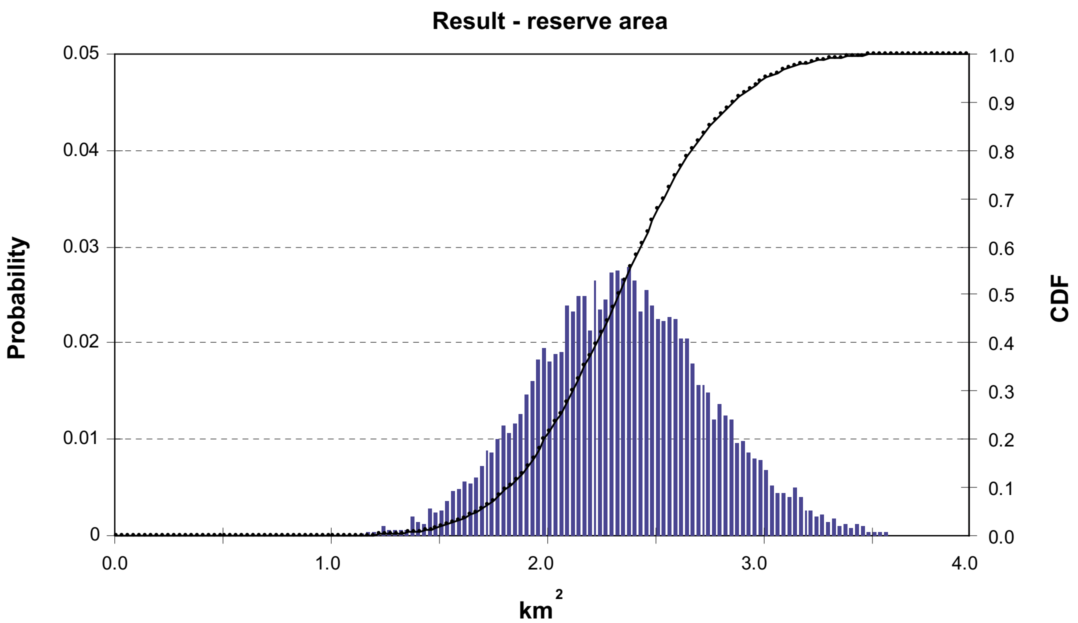


Figure 10-3. Forecast of the reserve area as result of the Monte Carlo simulation. All simulations gave a result of reserve area > 0, i.e. there is accommodation for the repository in 100% of the cases.

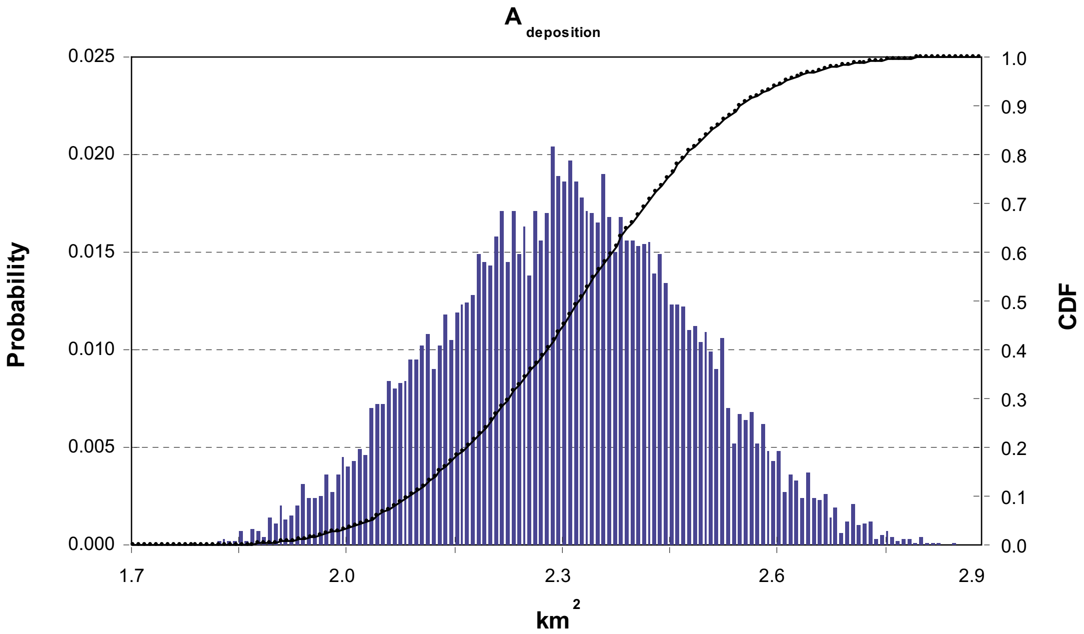


Figure 10-4. *Simulation of the necessary deposition area. Each bar represents an interval of 0.008 km^2 and the probability applies for an area within each interval. The CDF shows the probability that the necessary deposition area is below a certain value.*

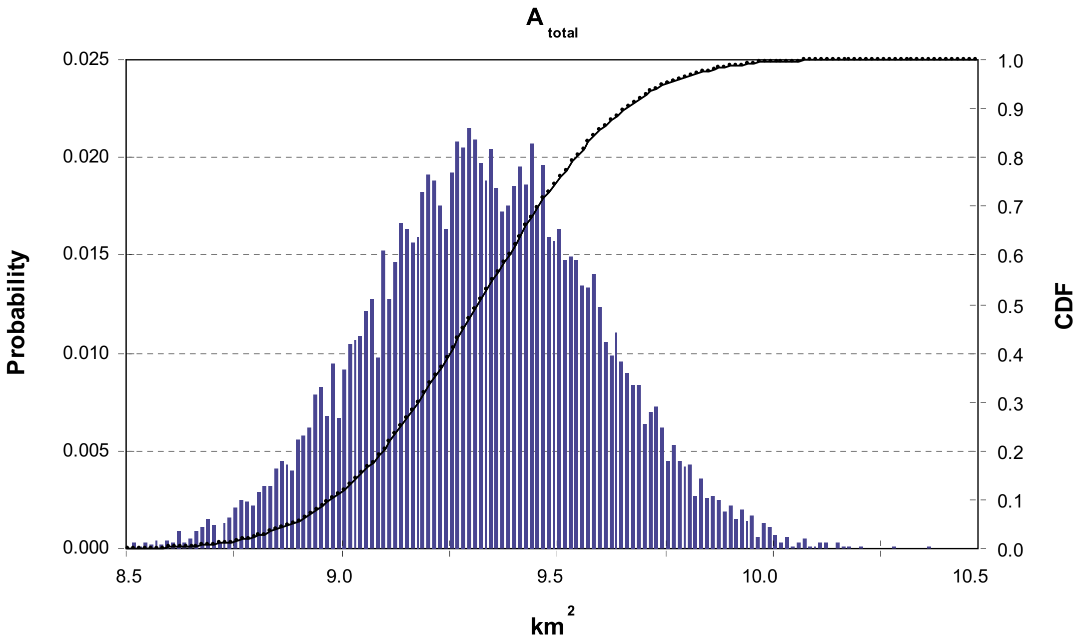


Figure 10-5. *Simulation of the total available area. Each bar represents an interval of 0.013 km^2 and the probability applies for an area within each interval. The CDF shows the probability that the total available area is below a specific value.*

Figure 10-6 shows the results from calculation of the total deformation zone area. The graph shows the probability of different sizes of the zones' total area.

Figure 10-7 shows the calculation results of small areas that are not sufficient for deposition area. The graph shows the probability of different sizes of the small areas' total area.

Figure 10-8 shows the result of areas that for geometrical reasons are not suitable for deposition areas.

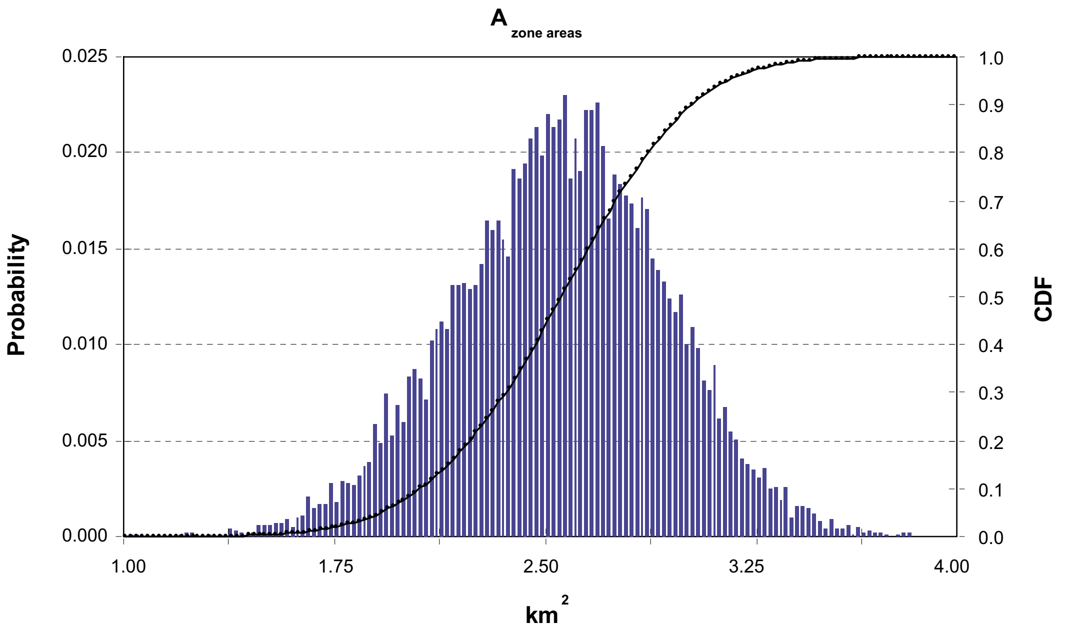


Figure 10-6. Simulation of the deformation zones identified and their total area. Each bar represents an interval of 0.030 km² and the probability applies for an area within each interval. The CDF shows the probability that the total deformation zone area is below a specific value.

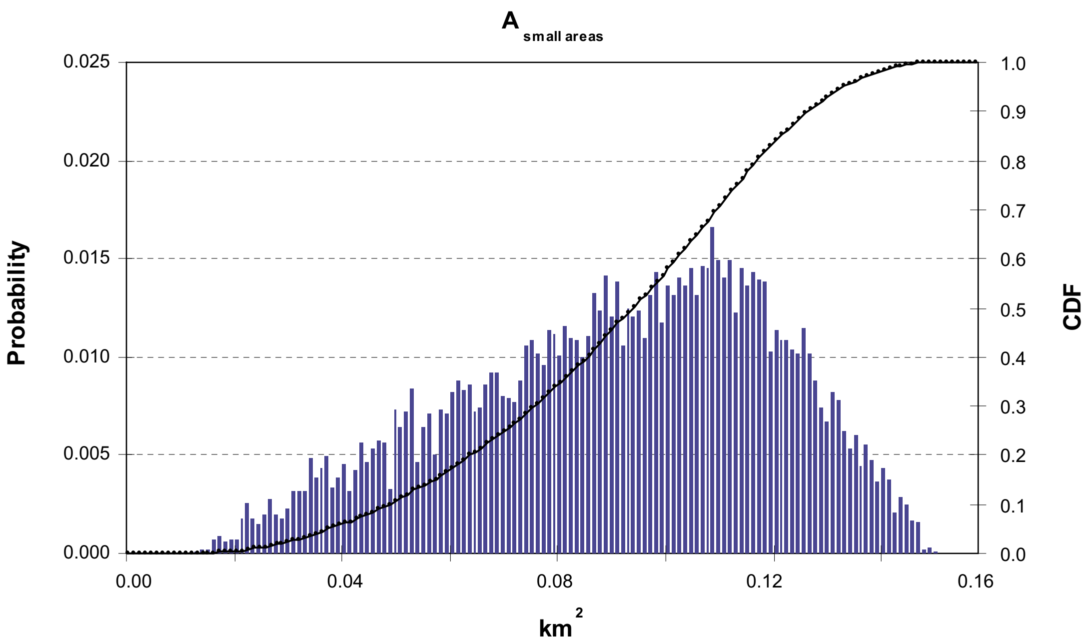


Figure 10-7. Simulation of small areas that are not sufficient for deposition area. Each bar represents an interval of 0.0016 km² and the probability applies for an area within each interval. The CDF shows the probability that the small areas' total area is below a specific value.

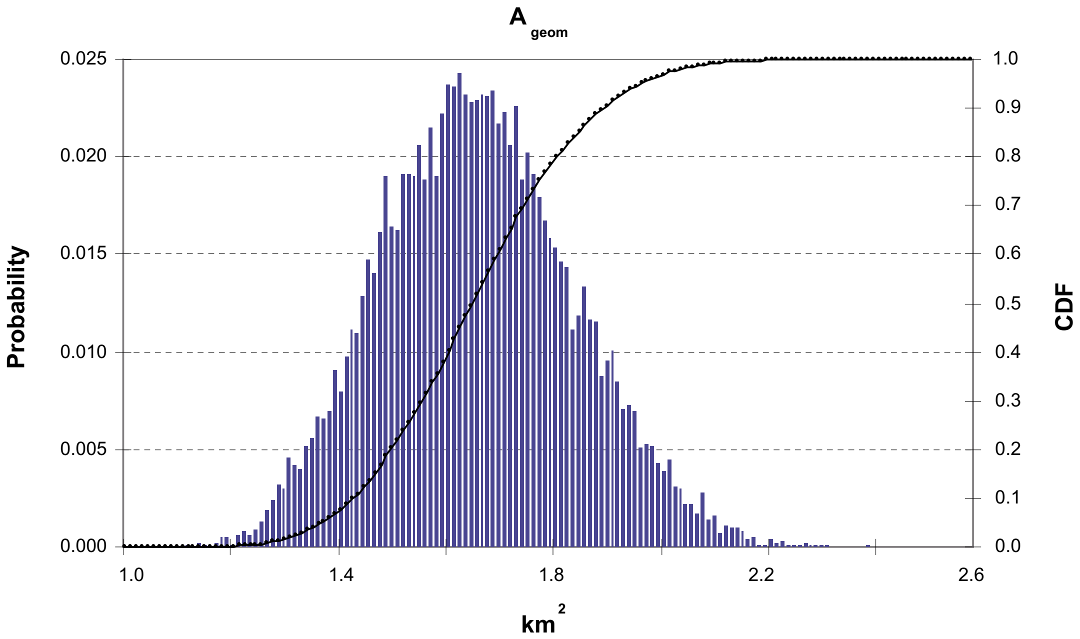


Figure 10-8. *Simulation of areas that for geometrical reasons are not suitable for deposition area. Each bar represents an interval of 0.008 km² and the probability applies for an area within each interval. The CDF shows the probability that the area is below a specific value.*

10.3.2 Sensitivity analysis

In order to illustrate the importance of the constituent parameters for uncertainty (variance) in the final results, a sensitivity analysis was conducted. Figure 10-9 shows a ranking of the 15 parameters that are of greatest importance for the uncertainty in calculation of the size of the reserve area.

Figure 10-9 shows that the uncertainty of the distance between deposition holes is most important in the simulation for the question "Is there enough accommodation for the repository". The dips of the limiting deformation zones, see Figure 10-1, also explain much of the uncertainty. As *Ls* is made up by the national interest for a final repository it has no variable dip. However, the length can vary depending on the dip of ZSMNS001C (Lv) and ZSMNE005A (Lsö). The presence of deformation zone ZSMN046A has also large influence on the simulation result.

As a part of the sensitivity analysis, a simulation was performed in which the proportion of deposition holes that cannot be used (loss parameter *K*) have been defined on the basis of the water seepage criterion as max 0.1 l/min instead of 10 l/min. This means a greater loss in which the most likely value is a loss of 45%, which is a weighted value of the A (41%) and M (59%) domains (Section 4.4.3). To get a triangular distribution the min and max value was set at 15% and 95 % respectively.

Table 10-6 summarises the results that differ compared with previous calculations. Figure 10-10 shows the calculated reserve area on condition that the seepage criterion 0.1 l/min is used. The alternative simulation, with changed loss proportion, shows that with a probability of 71% the repository accommodates 6,000 canisters. The sensitivity analysis showed that this simulation was almost fully dependent on the loss criterion (92 % of the uncertainty). The second most important parameter was the distance between the deposition holes (3%). All other parameters had an influence < 1% on the uncertainty.

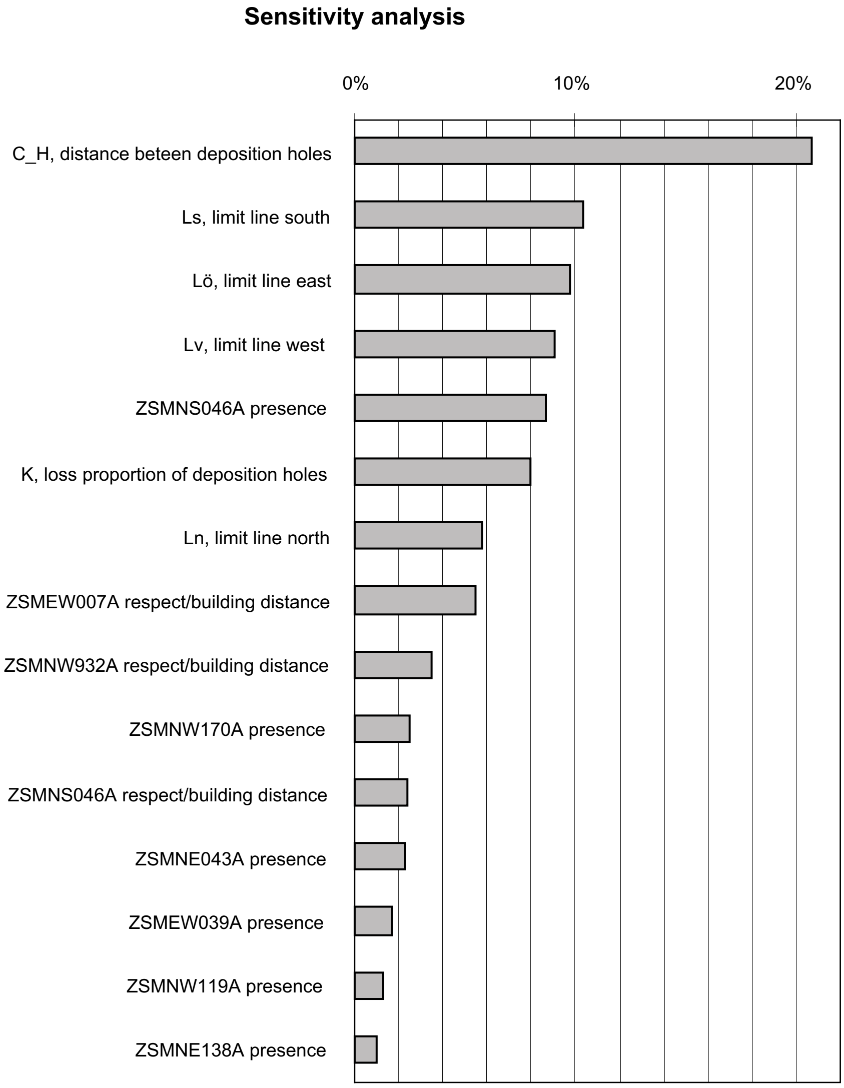


Figure 10-9. *Parameters ranked according to their percentage contribution to uncertainty (variance) in the size of the reserve area. Only the 15 most important parameters are included.*

Table 10-6. Summary of results of the alternative Monte Carlo simulation. Only the deposition area and reserve area are affected by the change in loss proportion.

Parameter	Unit	Mean	Median	Lowest value	Highest value
Reserve area	km^2	0.18	1.01	–31.8	3.70
$A_{deposoition}$	km^2	4.53	3.64	1.87	31.9
A_{geom}	km^2	1.67	1.66	1.13	2.40

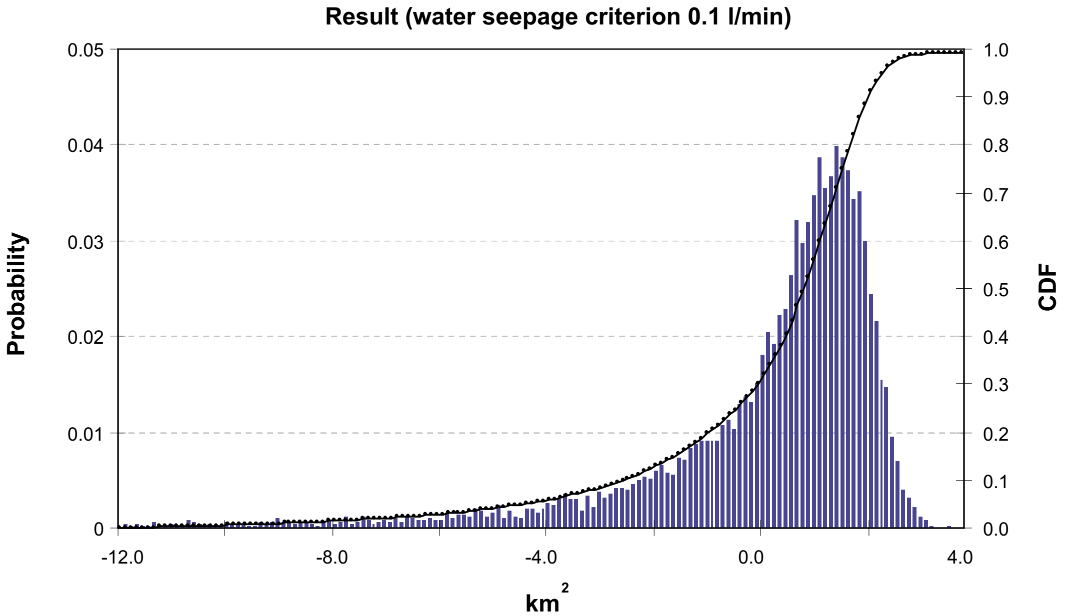


Figure 10-10. *Forecast reserve area as a result of the alternative Monte Carlo simulation. The loss proportion has been assumed to be greater according to the water seepage criterion of max 0.1 l/min. 71% of the simulations gave a result area > 0, i.e. there is accommodation for the repository in 71% of the cases. The CDF gives the probability that the reserve area is below a specific area.*

10.4 Design risks

The selected approach to "test and evaluate the design methods" has been for each section, from 3 to 9, to identify events in the design that could initiate a possible undesirable damage incident and propose measures for it, linked with the design stage.

The following steps have been carried out:

- Object, i.e. the purpose of the design question.
- Event that affects the object.
- Damage.
- Impact on the layout.
- Preventive measure.

After this, a risk matrix was prepared with respect to the probability of the event and the extent of the impact. The evaluations in the risk matrix will be mainly subjective, but give an indication of the principal design risks.

Those events that can be considered to have the main design risks are the ones connected with co-ordination of different project groups regarding preconditions, results and design criteria, i.e. communication issues. Clear examples of this is the design D1 (this design work) where the separate studies, "elongated fractures/fracture zones" (Section 4.4), "spalling rock" (Section 4.3 and 4.4) and "numerical seepage simulation" (Chapter 7) were to be worked up into the rest of the design work. Other examples are:

- revised geometries and locations of rock caverns in central and above-ground facilities,
- deadlock of other above-ground facilities such as ventilation buildings,
- insufficient space in certain critical passages where installations, etc will be collected,
- design criteria for installations and transportation that affects the space and driving,
- overall parameters for the repository safety such as the depth interval.

The proposed measures are to identify and check off the various design conditions and given design criteria between the different design groups. This is followed by problems of a more technical character where excavation through deformation zones, underestimation of the water situation and also the handling of the different hydro data/analysis/simulations and seepage criteria constitute the greatest design risks.

10.5 Discussion and conclusions

Calculation by means of Monte Carlo simulation indicates that there is enough accommodation space for the repository with a probability of 100%. The mean reserve area is *2.38* km^2 which can be compared with the required deposition area of 2.31 km^2 (i.e. the total available area is 4.69 km^2). Areas that are problematic to use for designs reason were taken into consideration.

The four depositional areas used for the basic layout (Alternative 500 Central) holds enough space to accommodate the repository even in cases where a larger area is needed for deposition for different reasons. The parameters that affect the area needed for deposition are the required distance between deposition holes and the percentage loss of deposition holes. Those are also the parameters that have a considerable impact in the sensitivity analysis (Figure 10-9).

The size of the four depositional areas used for the basic layout is affected by four parameters in the Monte Carlo simulation (Figure 10-9). These parameters are:

- whether the medium confidence deformation zone ZSMNS046A exists or not,
- the position of two of the limiting lines for the area, Lv and Ln, which are directly depending on the dip of the deformation zones ZSMNS001C and ZSMEW002A,
- whether respect distance (100 m) or margin for excavation (20 m) should be applied to the deformation zones ZSMEW007A and ZSMNW932A, i.e. whether the zones are longer than 3 km or not,
- whether the low confidence deformation zones ZSMNW170A, ZSMNE043A and ZSMNE138A exists at a depth of 500 m or not.

Further parameters that could have an impact on the four deposition areas used for the basic layout, but were not included in the sensitivity analysis at this stage, are uncertainties in the dip of some high confidence deformation zones. The deformation zones in question are:

- ZSMEW007A, ZSMEW900A and ZSMNW932A.

For the alternative design, i.e. Alternative 500 West, the deposition tunnels are placed in five different deposition areas. In spite of this the total area is somewhat smaller (3.22 km^2) compared to the four areas of the basic layout (3.43 km^2). Please note that these figures and all other total areas in Section 3.2 are calculated on the basis that all high and medium confidence zones exist and that no low confidence zones exist. This implies a larger loss area compared to the result (median and medium) from the Monte Carlo simulation, where existence of the different confidence zones are calculated individually in each simulation.

There are also more parameters to take into account for this alternative as it is affected by the high confidence deformation zone ZSMNW042A. This alternative has access to more extra area than does the basic layout. However, it is not of great importance to have a large extra area for the basic layout.

The conclusion of the discussion above is that the chosen basic layout (Alternative 500 Central) is the most suitable and that there is enough space to accommodate the repository with a good margin. Furthermore the parameters listed above should have high priority for further investigations.

An alternative sensitivity analysis was carried out where the used loss of deposition holes were based on a more conservative criterion for seepage to the deposition holes. Instead of the 10 l/min that were used for the former case a maximum seepage of 0.1 l/min was used. This means that a larger percentage of deposition holes are lost due to too high seepages. The result of this analysis was that the probability that the area is large enough to accommodate the repository is 71%.

The events, during the design stage, that entail the greatest design risks and influence over continued survey work are those associated with co-ordination of different project groups regarding preconditions and design criteria.

Finally, the following "feedback" is obtained from this chapter:

- *"Design in connection with continued survey work"* in which the questions that are critical for construction can be divided into both co-ordination matters between technical/design groups and separate surveys. Here the risks of communication problems must be identified. Furthermore the technical issues such as more knowledge on the handling of deformation zones at great depths and the interpretation of the hydrogeological situation from hydro data (DFN, analytical, numerical and connection to seepage criteria).
- *"Site organisation with respect to continued surveys"* where the knowledge of a number of critical deformation zones (occurrence, length and dip, whether respect distance or margin for excavation should be used) should be of high priority for further investigations in order to provide data for the parameters that control the loss proportions.
- *"Safety analysis with respect to what parameters control the extent of the repository"* It is crucial to decide the seepage criteria for the analysis of loss proportions.

11 Conclusions

11.1 Outcome of design task

11.1.1 Layout

As a whole, the results from design step D1 show that several alternative locations of the deep repository are possible within the Laxemar area. This becomes the conclusion after analysis of a number of parameters including deformation zones, thermal properties of the rock mass, seepage to the repository and rock mechanical stability issues such as unstable wedges, rock spalling and the affect of elongated fractures (Chapter 3–5). The key conditions for the layout are presented in Table 11-1.

The key data for the layout are presented in Table 11-2.

Table 11-1. Summary of key conditions for the layout work.

Conditions	Description of conditions
Possible location	The limits for the accessible area are: • To the east, the deformation zone ZSMNE005A and the outer limit of national interest for a final repository. • To the north, the deformation zone ZSMEW002A. • To the west, the deformation zone ZSMNS001C. • To the south, the boundary corresponds with the southern limit of the national interest for a final repository. The total accessible area is between 5.30 and 5.58 km^2, depending on depth, the 31 deformation zones and their respect distance or margin of excavation excluded.
Distance between deposition tunnels and deposition holes	The distance between deposition tunnels should be 40 m. Between the deposition holes, at depth 500 m, the distance varies between 7.0 and 8.1 m depending on rock domain.
Orientation of deposition tunnels	The obtained optimal orientation was N132°, i.e. parallel to the main horizontal stress. However, it was shown that the differences between different orientations was so small that it was decided that the design should not take this into account but focus on optimisation with respect to the accessible areas instead.
Loss of deposition holes	The loss of deposition holes varies between 17.5–22% at the depth of 500 m.
Depth of the repository	The repository should be located at the depth of 500 m.

Table 11-2. Summary of key data for the basic layout.

Key data	Value
Enclosed area for deposition (m^2)	32,355,490
Total length of main tunnels (m)	6,500
Total length of transport tunnels in the deposition area (m)	4,600
Number of deposition tunnels	244
Total length of deposition tunnels (m)	60,620
Number of canister positions, excluding loss of deposition holes	6,000
Number of canister positions, allowing for 20% loss of deposition holes	7,498
Excavated volume including central area, ramp and shaft but excluding deposition holes (m^3)	2,534,880

11.1.2 Hydrogeological results and rock support system

Regarding the issues of seepage to the repository and the extent of grouting and rock support, the uncertainties of volumes and volume intervals are large for the entire repository (Chapter 7–9).

In general the equations given in the UDP /SKB 2004a/ for analytical solutions of "Seepage and hydrogeological situation around repository" are not fully applicable in this context which made the analytical calculations very uncertain.

The results from the numerical calculations of seepage gave that the repository will be in the range of 4–19 l/s for the case where the grouting level is set to 10^{-9} m/s depending of construction step. When the grouting level is set to 10^{-7} m/s the seepage is between 14–37 l/s.

The drawdown area, as calculated using the numerical model, will be significant. For both of the grouting cases an area of about 10 km^2 will get a groundwater table that is depressed by 0.3 m or more. The results from the numerical simulations show little risk of inflow of too saline groundwater.

A summary of the grout quantity is presented in Table 11-3.

The practical experience of documented projects from similar conditions and requirements are limited. Due to the great depth and the in situ water flow and the pressure conditions, a wrong choice of grouting method technique and associated equipment could have more impact than a facility nearer the surface with "normal" conditions.

The total quantity of bolts, wire mesh and shotcrete for the basic layout is presented in Table 11-4 and special for the deposition tunnels in Table 11-5.

Table 11-3. Summary of grout quantities injected into the rock mass.

Sealing level	Grout quantity (m^3)
1 (10^{-7} m/s)	Total repository: 600–1,250
2 (10^{-9} m/s)	Total repository: 4,200–8,400
1 (10^{-7} m/s)	Deposition tunnels: 350–650
2 (10^{-9} m/s)	Deposition tunnels: 3,000–6,000

Table 11-4. Summary of rock support for the basic layout.

Item	Quantity Min	Mean	Max
Shotcrete, un-reinforced (m^3)	0	20	29
Shotcrete, fibre reinforced* (m^3)	11,690	16,820	19,420
Bolts, pcs	145,220	177,390	188,330
Bolt mortar, (m^3)	450	540	580
Wire mesh, (m^2)	219,050	281,950	293,250
Spiling bolts, pcs	0	470	730

* Included all energy absorption categories.

Table 11-5. Summary of rock support for deposition tunnels.

Item	Quantity Min	Mean	Max
Shotcrete, un-reinforced (m^3)	0	0	0
Shotcrete, fibre reinforced* (m^3)	375	1,030	1,985
Bolts, pcs	102,000	125,490	132,730
Wire mesh, (m^2)	219,050	281,950	293,250
Spiling bolts, pcs	0	200	400

* Included all energy absorption categories.

The required quantity of rock support will depend on the extent of stress-induced spalling in the facility, even though it has in general been assumed to be less than 5% of the overall tunnel length. If no form of stress-induced spalling occurs, the quantity of shotcrete could probably be significantly reduced. The number of bolts would also be reduced, though to a somewhat lesser extent.

Furthermore it is estimated that it will be technically feasibly to deal with the most demanding deformation zones within the site during construction and operation (Chapter 6).

11.1.3 Technical risk assessment

The result from the sensitivity analysis (Chapter 10) showed that there is a good margin for the Laxemar area to accommodate the repository. In fact the probability that the area is large enough came out as 100% from the analysis. The analysis also gave that the mean value for extra area would be 2.38 km^2 compared to a mean value of 2.31 km^2 for the area needed to accommodate the repository. This means that the available extra area is as large as the area needed for the repository. The analysis was performed taking the uncertainties and input data into consideration.

The four depositional areas used for the basic layout are large enough to accommodate the repository even in cases where a larger area is needed for deposition for different reasons. The parameters that affect the area needed for deposition are the required distance between deposition holes and the percentage loss of deposition holes. Those are also parameters that have a considerable impact in the sensitivity analysis.

The size of the four depositional areas, used for the basic layout, is affected by four main parameters in the Monte Carlo simulation. These parameters are:

- whether the medium confidence deformation zone ZSMNS046A exists or not at repository depth,
- the position of two of the limiting lines for the area, Lv and Ln, which are directly depending on the dip of the deformation zones ZSMNS001C and ZSMEW002A,
- whether respect distance (100 m) or margin for excavation (20 m) should be applied to the deformation zones ZSMEW007A and ZSMNW932A, i.e. whether the zones are longer than 3 km or not,
- whether the low confidence deformation zones ZSMNW170A, ZSMNE043A and ZSMNE138A exists at a depth of 500 m or not.

Further parameters that could have an impact on the four deposition areas used for the basic layout but that were not included in the sensitivity analysis for the whole Laxemar area, in Chapter 10, are uncertainties in the dip of some high confidence deformation zones. Deformation zones in question are:

- ZSMEW007A, ZSMEW900A and ZSMNW932A

11.2 Critical issues

The most critical issues identified concern deformation zones and the expected loss of deposition holes.

Regarding deformation zones the most critical issues are:

- whether several zones exists at repository depth or not,
- if some of the zones have a length of more than 3 km,
- to establish the dip of zones.

To get further information on these issues is critical to make it possible to further optimize the design and location of the repository.

For a more accurate estimation of the loss of deposition holes it is necessary to get a better picture of elongated fractures/fracture zones as well as water seepage. The latter is also connected to what seepage criterion value should be used.

11.3 Recommendations

Communication issues and problems in coordination within SKB and between SKB and the external groups working with the design of the repository could impose a risk for errors in the design work. This regards the whole process from site investigation to the design of the repository and in the end the construction. Problems in communication and coordination could mean a risk that different analysis is carried out based on inadequate information which in the end could lead to errors in the design.

To find ways minimising these possible problems is an important task for the subsequent design work. One purpose is to identify and check off the various design conditions and given design criteria between the different design groups.

The following "feedback" is obtained:

To site organisation:

Further investigations should be concentrated on the tasks listed under critical issues (Section 11.2).

To design:

Those questions that are critical for construction can be divided into both co-ordination matters between technical/design groups and separate studies, where the risks of communication problems and the more directly technical issues must be identified. The technical issues are more knowledge on the handling of deformation zones at great depth and not to underestimate the water situation. Also a more overarching approach to handle hydrogeological data is needed to reach concordance between the different analysis methods and between different seepage criteria (e.g. seepage to deposition holes connected to grouting levels).

Furthermore it is recommended that a sensitivity analysis, using Monte Carlo simulation, should be done where only the deposition areas used for the basic layout is taken into account. This way the probability that any extra areas need to be used can be evaluated.

Feedback on the UDP /SKB 2004a/ and the working strategy method used by SKB:

In part the UDP /SKB 2004a/ should be reviewed for the next design step. Especially the section regarding “Seepage and hydrogeological situation around repository” is unclear and need a somewhat different approach. The sections handling “Estimation of rock grouting need” and “Estimation of rock support need” are too extensive and detailed for design step D1. Furthermore, the outline of the work reports should be more consistent with the outline of the final report.

As a whole, the experience of the methodology for design step D1 as described in UDP i.e. the division into the steps of design work – presentation – conceptual report – check -revision of report – final report has been positive. The part that has worked the least satisfyingly, from the design team point of view, is the presentation of the design work and the subsequent check of the conceptual report. In many cases the comments following the check of the report differ a great deal from the issues that have been discussed at the presentation meetings. This is, of course a natural part of the process but has, at times, led to unnecessary extensive revision work that could have been avoided if the presentation meetings had worked more efficiently.

To safe analysis:

The work for safe analysis is to decide the seepage criteria for the analysis of loss proportions.

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