R-06-35

Final repository for spent nuclear fuel

Underground design Simpevarp, Layout D1

Svensk Kärnbränslehantering AB

April 2006

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Preface

This report is a compilation of the results of the underground design work carried out in design phase D1 of the Repository Design Project within the Deep Repository Project for the Simpevarp site. Similar reports are also being produced for the Laxemar and Forsmark sites. The design phase coincides with the initial site investigation phase.

The main purpose of phase D1 is to answer the question “Can a final repository be accommodated within the designated site”, but also to test the design methodology and provide feedback to the modelling project.

The design work for Simpevarp was carried out by FB Engineering AB in cooperation with subcontractors for certain areas. For two sections, Computer-aided Fluid Engineering AB and Prof. Derek Martin of the University of Alberta conducted studies in separate assignments for SKB.

Design was carried out in accordance with the methodology described in UDP (Underground Design Premises), SKB Report R-04-60, and was based on preliminary data from various disciplines in the site modelling project. The preliminary input data used were then cross-checked against data in the final Site Descriptive Model SDM v 1.2 and significant differences were integrated in the design work.

The design results from each design topic were presented by the designer at presentation meetings for SKB’s design management and the reviewers engaged by SKB for the specific topic. After the presentation meeting the designer wrote up the work reports for the topic in question. The work reports were then reviewed by SKB’s review team. The results of the review were compiled in a statement that was submitted to the designer to be dealt with. In the statement the designer documented which comments were dealt with and how. This report is a compilation of the entire design phase D1 for Simpevarp.

The 3D layout with coordinate lists for deposition holes and tunnels that was drawn to illustrate a possible layout was used in the Preliminary safety evaluation of the Simpevarp subarea and the hydromodelling of the Open Repository, both activities within the Deep Repository Project.

Stockholm, 30 March 2006

Eva Widing

Summary

Objectives

This report summarises the work performed within design step D1, which is based on the Site Descriptive Model Simpevarp v 1.2 (SDM v 1.2). In design step D1, three different sites for the repository − Simpevarp, Forsmark and Laxemar − are investigated. After design step D2 is completed, the most suitable site will be selected for the application for concession stipulated by Swedish environmental laws and regulations.

According to current plans for the Swedish nuclear programme, the minimum required number of canister positions in the repository is estimated to be 4,500. However, in order to accommodate the uncertainty entailed by possible future extensions of the operating periods of the nuclear power plants, the deposition area should, according to SKB, be designed for a capacity of 6,000 canisters.

SKB has published guidelines entitled "Underground Design Premises" (UDP) /SKB 2004a/ for the design of the repository, and from these guidelines the following basic objectives for the Layout D1 design can be summarized:

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

During the course of the studies, consequences of the applied design methodology, findings in other parallel ongoing studies, R & D work, etc have occasioned certain deviations from the UDP, which are further summarized and explained in Chapter 2 of the report.

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

The possible location of a tentative Deep Repository has been defined by SKB as lying within the *Simpevarp interest area* /SKB 2003/. This area has been further restricted by allowing deformation zones ZSMNE005A and ZSMNE024A to mark the western and eastern boundaries, respectively. The study area is shown in Figure 3-1 of the report.

The bedrock in the Simpevarp subarea is divided into four rock domains: domain A (Ävrö granite), B (fine-grained diorite), C (mixture of Ävrö granite and quartz monzodiorite), and D (quartz monzodiorite). Rock domain A dominates the main part of the area, followed in size by rock domains B and C. Rock domain D lies outside the study area.

The preliminary assessment made in Chapter 3 demonstrates that the site has a clear potential to accommodate a storage facility. The factor *P* varies between 1.78 and 2.07, taking into account high confidence zones and all study depths. A *P* value of 1 indicates the site has sufficient capacity.

If “possible” as well as high confidence deformation zones are taken into account the *P* value falls by 12%.

Design of deposition areas

The studies of the design of deposition areas are reported in Chapter 4 and include the design of layout features for all tunnels, deposition holes, orientation of tunnels, calculation of anticipated losses of deposition holes due to applied design criteria and recommendation regarding repository depth.

For design step D1, tunnel geometries and dimensions were recommended in accordance with Layout E /SKB 2002a/ and /SKB 2002b/.

The studies of thermal properties show that for the rock domains relevant for design, namely domains A to C, the minimum allowable canister spacing varies from 7.1 to 7.3 m at a depth of 400 m and from 8.3 to 8.5 m at a depth of 700 m.

The analytical analysis shows that water inflow to the deposition tunnel is in principle independent of tunnel orientation, while an orientation that minimizes water inflow can be identified by DFN simulation.

It should be noted that transmissivity measurements in the spiral ramp at the Äspö HRL are in sharp contrast to the results presented here. These measurements show a clear anisotropy with a factor of one hundred between the direction of minimal and maximal transmissivity. The highest transmissivity was recorded in fractures in a NW-SE direction /Rhén et al. 1997/.

In stress domain I water inflows reach a minimum with a tunnel orientation of approximately 60° to the maximum horizontal stress (N015). This orientation is not optimal for minimizing the risk of spalling. However, this orientation is unlikely to result in spalling at a repository depth of 400 m to 500 m. The orientation of deposition tunnels in the layout has been chosen with respect to water inflow.

Intersection of deposition holes by large fractures is analyzed by two different models, analytical and numerical. The loss of deposition holes due to large fractures is 13% according to the analytical model and approximately 10% according to the numerical model. Both model results showed independence from the tunnel orientation.

Deposition hole losses due to unacceptably high water inflows are expected to be in the order of 1% for an inflow criterion of 10 l/min per deposition hole and 2–5% for an inflow criterion of 1 l/min per deposition hole, depending on the tunnel orientation.

Overall the results show that there is essentially no risk of spalling in stress domain I at depths of 400 m to 500 m. However, the risk increases significantly at greater depths. Results for a depth of 700 m indicate there is a clear risk that the majority of deposition holes located in stress domain I would be lost due to spalling.

In summary, it is concluded that the total combined deposition hole losses are 13% according to the analytical method or 10% according to the numerical method for the theoretical total. Losses are minimized by placing the repository at a depth of 400 m to 500 m.

In general, the shallower the storage level the more favourable the overall storage conditions. The results of the current study from the various design tasks all indicate that for the depth range considered, a storage depth of 400 m is most advantageous. There are however factors of importance for long-term safety that are not considered in the UDP, and according to SKB several of these factors will result in a deeper placement of the repository. Since the benefits of placing the repository at 400 m as compared to the initial reference level of 500 m /SKB 2002c/ are marginal, the reference level of 500 m has been maintained for the purposes of the current Simpevarp D1 layout.

Layout studies

The layout studies are reported in Chapter 5.

The siting process needs to consider both current land use and long-term environmental issues. A site within or near an existing industrial area, exemplified by locating the Operations Area in close proximity to Clab, would give access to an already established transport and utilities infrastructure, avoiding the exploitation of a completely new undisturbed site. A possible alternative to an industrial area would be Hålö, a site located within an area currently used for commercial forestry, with little other land use interests.

The D1 design layout at level 500 m shows sufficient space and volume are available at the site for the anticipated number of 6,000 canisters. The anticipated volume of the underground facilities is approximately 2.2 million m³, including 65 km of tunnels and deposition holes.

Identification of passages through deformation zones

The proposed repository layout involves eleven passages through deformation zones, reported in Chapter 6. Seven of these deformation zones are classified as high confidence and four as “possible”. The total tunnel length of passages is approximately 415 m. Individual passage lengths vary from 10 to 70 m.

There is a risk of potentially *high* water inflows for two of the passages. Of the total passage tunnel length, approximately 125 m has a risk of *high* water inflows.

The proposed rock support is to a large degree based on recommendations from the Q system. The proposed grouting activities are focused on a strict programme of probing, grouting and control holes of sufficient length using cement-based grouts. Freezing and the installation of a local concrete lining is proposed as an alternative method for passages with potentially high water inflows and very to extremely poor rock conditions /Chang et al. 2005/.

Seepage and hydrogeological situation around the repository

The study of seepage and the hydrogeological situation is reported in Chapter 7. The largest water inflows are associated with the passage of the deformation zones where hydraulic conductivities may be 1,000 times higher than in the surrounding rock mass. However, the tunnel sections intersecting deformation zones are relatively short. Major reductions in groundwater pressure due to local seepage at the zone passages are not expected.

Both the analytical and numerical methods indicate that inflow into the repository will be dominated by the deformation zone passages and the larger fractures. Total seepage to the repository for grouting level 0 (no grouting) is estimated to be 300–350 l/s.

Grouting efficiency clearly has an effect on the quantity of seepage. If grouting to a resulting hydraulic conductivity of 10^{-7} m/s (level 1) is achieved, this will result in a significant reduction in total seepage. However, the analytical estimation of the necessary seepage reduction in the passages indicates that grouting to a hydraulic conductivity of 10^{-9} m/s (level 2) will be difficult to achieve for the zones with high hydraulic conductivities.

Groundwater table drawdown due to the development of the repository is moderate and localized. The lateral extent of the depressed groundwater table is essentially limited to the area directly above the tunnels.

Saline water is drawn into the repository, particularly if grouting is limited to the higher grouting factors, which results in an estimated salinity of 2–4% TDS around the repository.

Estimation of rock grouting need

Estimation of rock grouting need is reported in Chapter 8. The total grout quantity injected into the rock mass, including plugged volume, is estimated to be 3,350 to 5,380 m^3 for grouting level 1 ($K = 10^{-7}$ m/s) and 15,380 to 18,615 m^3 for grouting level 2 ($K = 10^{-9}$ m/s). The deposition tunnels, with a total length of 54 km, dominate the grouting need with 1,590 to 2,690 m^3 for grouting level 1 and 9,750 to 11,250 m^3 for grouting level 2, all including plugged volume.

It is important to limit the pH in the rock mass around the repository and in the KBS-3 concept /SKB 2000a/, and in the safety analysis it is assumed that grout with a $pH < 11$ is used. It is assumed that the work is based on a standard pre-grouting programme and cement-based grouts are used.

The presented estimates should be taken as an initial attempt to assess the scale of the grouting work associated with the development of the repository. The number of existing excavations of a similar type and depth from which experience can be drawn is limited.

Estimation of rock support need

Estimation of rock support need is reported in Chapter 9. A preliminary estimate has been made of required support quantities in the repository. Due to uncertainties in the underlying parameters it seems reasonable to assume a range of variation of –15 to +40% of the calculated quantities.

The total quantity of bolts in the complete facility is calculated to be 45,000, of which approximately 20,000 are in the deposition tunnels. The area in the deposition tunnels supported by mesh is calculated to be approximately 46,000 m^2. This may be compared with the total area in other parts of the facility that are supported by shotcrete of 201,000 m^2, based on a theoretical rock contour.

The total amount of unreinforced shotcrete, based on a realistic rock contour including rebound (5,090 m^3), is approximately 60% greater than the amount of fibre reinforced shotcrete (3,160 m^3). The calculation of quantities results in a total weight of cement for rock support of approximately 2,645 tonnes and a total weight of cement for tunnel sealing of approximately 8,105 tonnes.

Technical risk assessment

The strategy for answering the question "Can the repository be accommodated within the assigned area" is to build an appropriate risk model.

The most important results obtained from the calculations are:

- There is a very high (> 99%) probability that 6,000 canisters can be accommodated within the studied area at a depth of 500 m.
- Total deposition hole loss, loss factor (1–*k*), is 13% on average.
- The average area needed to host the 6,000 canisters at a depth of 500 m is 3.5 km^2, with a range of 2–5.6 km^2. This is within the limits of what was found in the layout studies in Chapter 5, where an area of 4.5 km^2 was found to be needed. It should be noted that the layout study was not optimized.
- The three factors that have the greatest impact on uncertainty are:
  - Hole spacing due to thermal properties.
  - Loss percentage due to fractures with R > 100 m.
  - Dip of external boundary deformation zones.

Supplementary update based on new Site Description Model

The design D1 Simpevarp is based on the site conditions presented in SDM 1.2 Simpevarp. During the time the design work preceded a similar site description task was carried out for the adjacent area in Laxemar. This resulted in a remodelling of several deformation zones in the Simpevarp area, which gave some rather important changes to the base for the layout in the Simpevarp area. Some deformation zones were reclassified from "possible" zones to "high confidence" zones and new deformation zones are added. An additional study of the possibility to accommodate the repository at the Simpevarp location based on the remodelled deformation zones in SDM 1.2 for Laxemar has been carried out. The result of the study showed that the repository can be accommodated if the eastern border of the available area is the same as the *Interest area*, but not if the eastern border is the same as deformation zone ZSMNE024A. The study is reported in Appendix D.

Sammanfattning

Denna rapport utgör en sammanställning av den bergprojektering som utförts under skede D1 och som baseras på Platsbeskrivning Simpevarp v 1.2 (SDM v 1.2). Under projekteringssteg D1 tas motsvarande rapport fram för de tre platserna Simpevarp, Laxemar och Forsmark. Efter projekteringssteg D2 så kommer den lämpligaste platsen att väljas för ansökan om koncession enligt svensk miljölagstiftning.

Enligt nuvarande planer för det svenska kärnkraftprogrammet är det minsta antal kapslar i slutförvaret bedömt till 4 500 stycken. Men på grund av osäkerheten i möjlig framtida utökning av kärnkraftsverkens driftsperiod ska slutförvaret, enligt SKB, bedömas för 6 000 kapslar.

SKB har tagit fram en handledning "Underground Design Premises" (UDP) /SKB 2004a/ för projekteringen av slutförvaret. Från den kan följande huvudsakliga målsättning för projektering av Layout D1 summeras:

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

Under arbetets gång har avvikelser från UDPn gjorts på grund av den applicerade projekteringsmetodiken, resultat från parallella studier och FoU m m. Avvikelserna beskrivs och summeras i kapitel 2 i rapporten.

Möjliga platser och preliminär bedömning att rymma slutförvaret

Möjlig placering av ett tänkt djupförvar har definierats av SKB att vara inom *Simpevarps intresseområde* /SKB 2003/. Området har begränsats ytterligare genom att ange ZSMNE005A och ZSMNE024A till västlig respektive östlig gräns. Det studerade området visas i figur 3-1 i rapporten.

Bergmassan inom Simpevarps delområde är indelad i fyra bergdomäner: domän A (Ävrögranit), B (finkornig diorit), C (blandning av Ävrögranit och kvartsmonzodiorit) och D (kvartsmonzodiorit). Bergdomän A dominerar större delen av området och följs av bergdomänerna B och C. Bergdomän D ligger utanför det studerade området.

Den preliminära bedömningen som görs i kapitel 3 visar att området har en tydlig möjlighet att rymma förvarsanläggningen. Faktorn P varierar mellan 1,78 och 2,07, med hänsyn tagen till högkonfidenszoner och alla studerade djup. Ett P-värde på 1 anger att platsen har tillräcklig kapacitet.

Om hänsyn tas till såväl "möjliga" som deformationszoner med hög konfidensgrad så minskar P-värdet med 12 %.

Beräkning av deponeringsytor

Beräkningarna av deponeringsytorna är redovisade i kapitel 4 och omfattar beräkning av layoutegenskaper för alla tunnlar, deponeringshål, tunnlarnas orientering, beräkning av uppskattat bortfall av deponeringshål på grund av använda beräkningsförutsättningar och rekommendation avseende förvarsdjup.

För projekteringssteg D1 har tunnelgeometrier och dimensioner rekommenderats i enlighet med layout E /SKB 2002a/ och /SKB 2002b/.

Studierna av de termiska egenskaperna visar att för de bergdomäner som är av betydelse för projekteringen, nämligen domäner A till C, så varierar det minsta tillåtna kapselavståndet mellan 7,1 och 7,3 m på djupet 400 m och mellan 8,3 och 8,5 m på djupet 700 m.

Den analytiska beräkningen visar att vatteninläckaget i deponeringstunneln i princip är oberoende av tunnelns orientering, medan en orientering som minimerar vatteninläckaget kan identifieras genom en DFN-simulering.

Det bör noteras att transmissivitetsmätningarna i spiralrampen på Äspö HRL står i skarp kontrast till de resultat som presenteras här. Mätningarna visar en tydlig anisotropi med en faktor ett hundra mellan riktningen för minimal och maximal transmissivitet. Den högsta transmissiviteten uppmättes i sprickor i NV-SÖ riktning /Rhén et al. 1997/.

I spänningsdomän I når inläckaget ett maximum med en tunnelorientering på ca 60° mot maximal horisontell spänning (N015). Denna orientering är inte optimal för att minimera risken för smällberg. Emellertid är det osannolikt att denna orientering resulterar i smällberg på förvarsdjup 400 m till 500 m. Deponeringstunnlarnas orientering i layouten har valts med hänsyn till vatteninläckage.

Korsning av deponeringshål och stora sprickor har analyserats med två olika modeller, analytisk och numerisk. Bortfallet av deponeringshål på grund av stora sprickor är ungefär 13 % enligt den analytiska modellen och 10 % enligt den numeriska modellen. Båda modellerna visade sig oberoende av tunnelorienteringen.

Bortfall av deponeringshål på grund av oacceptabelt höga vatteninläckage bedöms vara i storleksordningen 1 % för läckagekriteriet 10 l/min per deponeringshål och 2–5 % för läckagekriteriet 1 l/min per deponeringshål, beroende på tunnelorientering.

Sammantaget visar resultaten att det finns väsentligen ingen risk för smällberg i spänningsdomän I på förvarsdjup 400 m till 500 m. Emellertid ökar risken signifikant på större djup. Resultat för djupet 700 m antyder att det finns en tydlig risk för att en majoritet av deponeringshålen i spänningsdomän I skulle falla bort på grund av smällberg.

Sammanfattningsvis kan konstateras att det totala sammanlagda bortfallet av deponeringshål är 13 % enligt den analytiska metoden eller 10% enligt den numeriska metoden för den teoretiska totalsumman. Bortfallet minimeras genom att placera förvaret på djupet 400 m till 500 m.

Allmänt kan sägas att ju grundare förvarsnivå, desto förmånligare förvarsförhållanden. Resultaten från den aktuella studien av de olika projekteringsstegen visar alla att vad avser förvarsdjup så är djupet 400 m fördelaktigast. Det finns emellertid betydande faktorer för långtidssäkerheten som inte beaktas i UDP och enligt SKB kommer flera av dessa faktorer att resultera i djupare placering av förvaret. Eftersom fördelarna med att placera förvaret på 400 m i jämförelse med den ursprungliga referensnivån på 500 m /SKB 2002c/ är marginella, har referensnivån 500 m behållits i den förhandenvarande Simpevarp D1 layouten.

Layoutstudier

Layoutstudierna är redovisade i kapitel 5.

Processen att bestämma platsen behöver ta hänsyn både till nuvarande landanvändning och till långsiktiga miljöfrågor. En plats inom eller i närheten av ett existerande industriområde, exemplifierat genom lokaliseringen av driftområdet i nära förbindelse med Clab, skulle ge tillträde till en redan etablerad infrastruktur för transporter och annan service och därmed undvika en utbyggnad av ett helt nytt och ostört område. Ett möjligt alternativ till ett industriområde vore Hålö, en plats inom ett område som för närvarande används för kommersiellt skogsbruk och med få andra landanvändningsintressen.

Nivån 500 m för layouten i projektering D1 visar att det finns tillräcklig plats och volym på platsen för det förutsedda antalet 6 000 behållare. Den förutsedda volymen av underjordsanläggningen är ungefär 2,2 millioner m^3, inklusive 65 km tunnlar och deponeringshål.

Identifiering av passager genom deformationszoner

Den föreslagna layouten för förvaret omfattar elva passager genom deformationszoner som redovisats i kapitel 6. Sju av dessa deformationszoner är klassificerade som hög konfidens och fyra som "möjliga". Den totala tunnellängden i passager är ungefär 415 m. Enskilda tunnelpassager varierar mellan 10 och 70 m.

Det finns en risk för potentiellt *höga* vatteninläckage i två av passagerna. Ca 125 m av den totala tunnellängden i passager löper en risk för *höga* vatteninläckage.

Den föreslagna bergförstärkningen är till stor del baserad på rekommendationer från Q-systemet. De föreslagna tätningsinsatserna fokuserar på ett noggrant program för undersökning, injektering och kontrollhål med tillräcklig längd och användning av cementbaserade injekteringsmedel. Frysning och installation av lokal betonglining föreslås som en alternativ metod för passager med potentiellt höga vatteninläckage och mycket till extremt dåliga bergförhållanden /Chang et al. 2005/.

Inläckage och hydrogeologisk situation kring förvaret

Undersökningen av vatteninläckage och den hydrogeologiska situationen är redovisad i kapitel 7. De största inläckagen finns i anslutning till passage av deformationszoner där de hydrauliska konduktiviteterna kan vara 1 000 gånger högre än i den omgivande bergmassan. Tunnelsektionerna som korsar deformationszoner är emellertid ganska korta. Större reduktioner av grundvattentrycket på grund av lokalt vatteninläckage kan inte förväntas.

Både de analytiska och numeriska metoderna visar att inläckaget till förvaret kommer att domineras av deformationszonernas passager och av de större sprickorna. Totalt inläckage till förvaret för tätningsnivå 0 (ingen injektering) uppskattas till 300–350 l/s.

Injekteringens effektivitet har en tydlig påverkan på inläckaget. Om injektering till en hydraulisk konduktivitet på 10^{-7} m/s (nivå 1) uppnås så kommer detta att resultera i en signifikant reducering av det totala inläckaget. Dock visar den analytiska uppskattningen av den nödvändiga reduktionen av inläckage i passagerna att injektering till en hydraulisk konduktivitet på 10^{-9} m/s (nivå 2) är svår att uppnå för zoner med höga hydrauliska konduktiviteter.

Sänkningen av grundvattenytan på grund av utbyggnaden av förvaret är måttlig och lokal. Utbredningen av den sänkta grundvattenytan är huvudsakligen begränsad till området direkt ovanför tunnlarna.

Saltvatten kan komma in i förvaret, särskilt om tätningen är begränsad till de högre tätningsnivåerna, vilket resulterar i en uppskattad salthalt på 2–4 % TDS runt förvaret.

Uppskattat behov av tätning

Uppskattning av behov av tätningsinsatser redovisas i kapitel 8. Den totala injekteringsmängden som injekteras i bergmassan, inkl. pluggad volym, uppskattas till 3 350 till 5 380 m^3 för tätningsnivå 1 ($K = 10^{-7}$ m/s) och 15 380 till 18 615 m^3 för tätningsnivå 2 ($K = 10^{-9}$ m/s). Deponeringstunnlarna, med en total längd på 54 km, dominerar tätningsbehovet med 1 590 till 2 690 m^3 för tätningsnivå 1 och 9 750 till 11 250 m^3 för tätningsnivå 2 inklusive pluggad volym.

Det är viktigt att begränsa pH i bergmassan runt förvaret och i KBS-3 konceptet /SKB 2000a/, och i säkerhetsanalysen förutsätts det att injekteringsmedel med pH < 11 används. Det förutsätts att arbetet grundas på ett standard förinjekteringsprogram och att cementbaserade injekteringsmedel används.

De presenterade uppskattningarna skall uppfattas som ett första försök att uppskatta storleksordningen av de tätningsinsatser som är förknippade med utbyggnaden av förvaret. Antalet existerande bergrum av liknande typ och djup och från vilka erfarenheter kan erhållas är begränsat.

Uppskattning av bergförstärkningsinsatser

Uppskattning av bergförstärkningsinsatser redovisas i kapitel 9. En preliminär uppskattning av nödvändiga förstärkningsmängder i förvaret har gjorts. På grund av osäkerheter i de underliggande parametrarna kan det vara rimligt att anta en variationsbredd på –15 till +40 % av de beräknade mängderna.

Den totala mängden bultar i hela anläggningen är beräknad till 45 000 varav ca 20 000 i deponeringstunnlarna. Ytan i deponeringstunnlarna som är förstärkt med nät är beräknad till ca 46 000 m^2. Detta kan jämföras med den totala arean på 201 000 m^2, baserat på teoretisk bergkontur, i andra delar av anläggningen som är förstärkt med sprutbetong,

Den totala mängden oarmerad sprutbetong, baserat på verklig bergkontur inklusive spill (5 090 m^3), är ca 60 % större än mängden fiberarmerad sprutbetong (3 160 m^3). Beräkningen av mängder resulterar i en total vikt av cement för bergförstärkning till ca 2 645 ton och en total vikt av cement för tunneltätning på ca 8 105 ton.

Teknisk riskbedömning

- Strategin för att besvara frågan ”Kan förvaret rymmas inom anvisat område?” är att tillämpa en lämplig riskmodell.

De mest betydande resultaten som erhållits från beräkningarna är:

- Det är en mycket stor (> 99 %) sannolikhet att 6 000 kapslar kan rymmas inom det studerade området på ett djup av 500 m.
- Det totala bortfallet av deponeringshål, faktor (1-k), är i medeltal 13 %.
- Den genomsnittliga area som behövs för att rymma 6 000 kapslar på ett djup av 500 m är 3,5 km², med en spännvidd på 2–5,6 km². Detta är inom de gränser för vad som erhölls i layoutstudierna i kapitel 5 där en area på 4,5 km² befanns nödvändig. Det bör noteras att layoutstudien inte optimerades.
- De tre faktorer som har den största inverkan på osäkerheten är:
  – Hålavstånd på grund av termiska egenskaper.
  – Bortfallsprocent på grund av sprickor med R > 100 m.
  – Stupning av yttre gränsens deformationszoner.

Kompletterande uppdatering baserat på ny Platsbeskrivning

Projekteringen av Simpevarp i steg D1 baseras på platsförhållandena som presenteras i platsbeskrivningen SDM 1.2 Simpevarp. Under tiden som projekteringsuppdraget utfördes för Simpevarp så pågick arbetet med att ta fram en platsbeskrivning för den angränsande platsen Laxemar. Det resulterade i ommodellering av flera deformationszoner i Simpevarpsområdet, vilket innebar viktiga förändringar av underlaget som layouten baseras på. Vissa deformationszoner klassificerades om från ”possible” zoner till ”high confidence” zoner och nya deformationszoner tillkom. En kompletterande studie har utförts för att bedöma möjligheterna att rymma förvaret i Simpevarpsområdet baserat på ommodelleringen av zonerna enligt SDM 1.2 Laxemar. Resultatet av studien visat att förvaret kan rymmas förutsatt att den östra gränsen av området utgörs av intresseområdesgränsen, men inte om den östliga gränsen går i deformationszonen ZSMNE024A. Studien redovisas i appendix D.

Contents

1 Introduction

1.1 Objectives

SKB is currently planning for the construction of a final repository for disposal of spent nuclear fuel from the Swedish nuclear power plants. Geological investigations are ongoing at the municipalities of Oskarshamn and Östhammar. This design study has been carried out by a design team, including FB Engineering AB, Kemakta Konsult AB, Tyréns AB, Naturgasteknik AB, Geostatistik AB, and Team uStation AB, to meet the goals for design step D1 of a final repository at the Simpevarp subarea.

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

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

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

SKB has developed guidelines entitled "Underground Design Premises" (UDP) /SKB 2004a/ for the design of the repository, and from these guidelines the following 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 need to be done,
  - the site investigation and modelling organization regarding further investigations required,
  - the safety assessment team.
- Provide illustrative tentative layouts for public consultations as required by Swedish environmental laws, comprising:
  - the location of surface facilities,
  - the location and extent of underground facilities,
  - baseline data for the environmental impact assessment.

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

1.2 Strategy

The site investigations for the final repository started in 2002 and are scheduled to continue until 2007. The design procedures will proceed in parallel 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 three key issues:

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

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

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

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

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

The site investigation data are submitted in consecutive batches ("data freezes") and each part is evaluated and assessed into a site descriptive model (SDM). However, in order to gain time the design team has worked in close co-operation with the 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 possibility 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 fully reviewed and approved 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.

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

1.4 Organisation

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

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

FB Engineering:

- Per Tengborg – Project leader, hydrogeological analyses and editor
- Rune Glamheden – Rock engineering and rock mechanics
- Philip Curtis – Geology, translation and review
- Claes Danling – Layout and design studies, CAD operator
- Mauritz Altahr-Cederberg – Project administrator and general engineering
- Ingemar Markström – CAD support and reviewer
- Maria Olsen – Reviewer
- Lars Clemensson – Reviewer

Naturgasteknik: Jan Johansson – Technical risk assessment

Geostatistik: Lars Olsson – Technical risk assessment

Tyréns: Thomas Janson – Rock grouting

Kemakta Konsult: Björn Gylling – Discrete Fracture Network (DFN) hydro analysis

Team uStation: Chris Zakrewsky – Software developer

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

1.5 Definitions and abbrevations

Definitions and abbreviations described in this section are mainly based on the UDP /SKB 2004a/.

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 Simpevarp area – version 1.2. /SKB 2005a/.

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”.

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 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.
Canister	Load bearing steel container with copper shell in which spent nuclear fuel is placed for deposition.
Permanent plug	Facility part that is used to permanently separate or seal various underground openings in the hard rock facility.
Backfill	Backfill refers to the material that is placed in deposition tunnels and the rock caverns in the central area as deposition proceeds.
Backfilling	Backfilling refers to the activity.

1.5.4 Underground openings

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

Rock cavern	Underground opening intended to contain caverns for personnel and visitors, technical systems, other equipment or for loading/unloading that is required for construction and operation.
Rock silo	Cavern for interim storage of rock spoil from blasting.
Central area's rock caverns	Caverns necessary for operation of the final repository.
Deposition hole	Hole for deposition of canisters containing spent nuclear fuel. Besides canisters, deposition holes also contain the buffer.

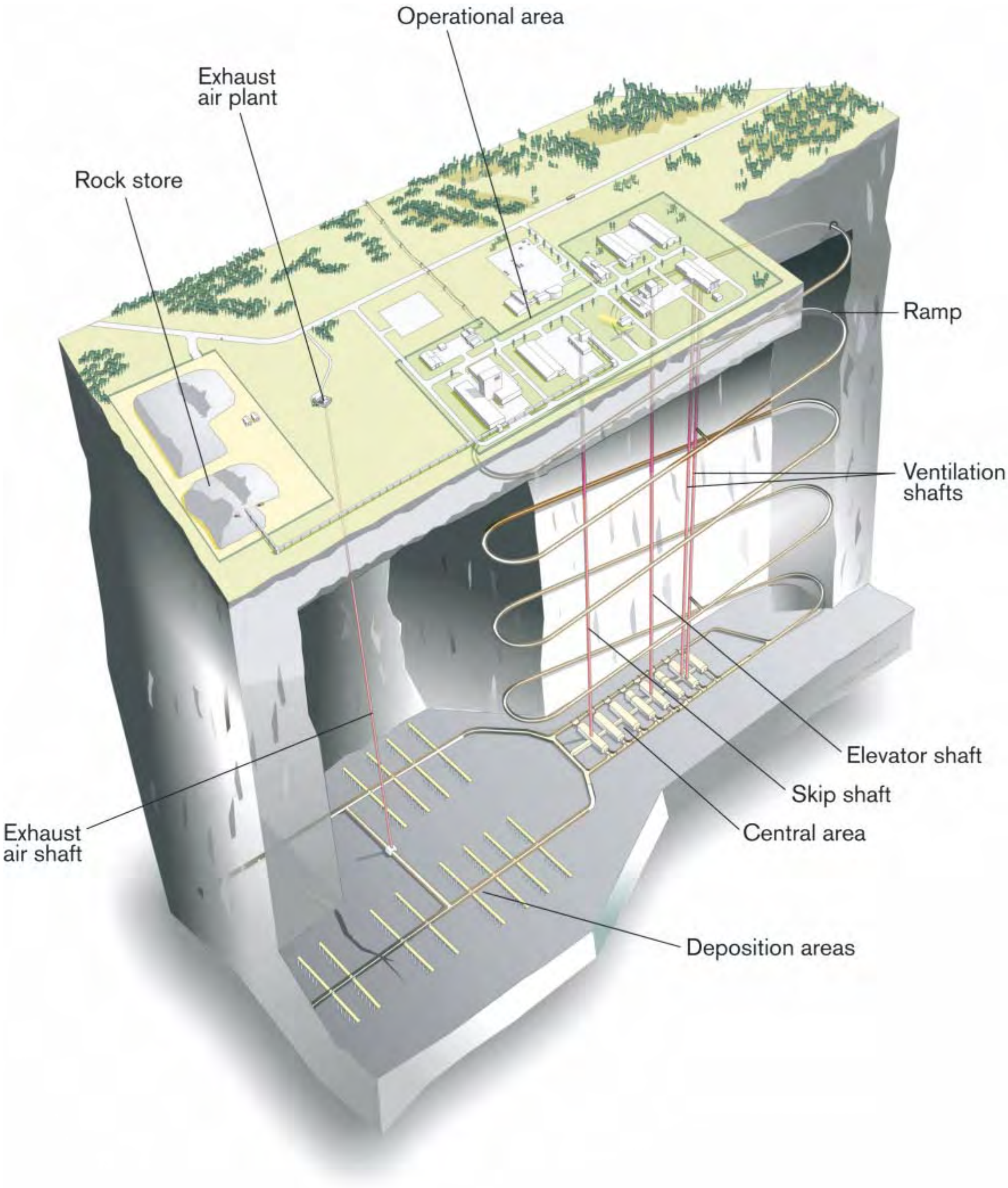


Figure 1-1. *3D-illustration of surface and underground facilities.*

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.

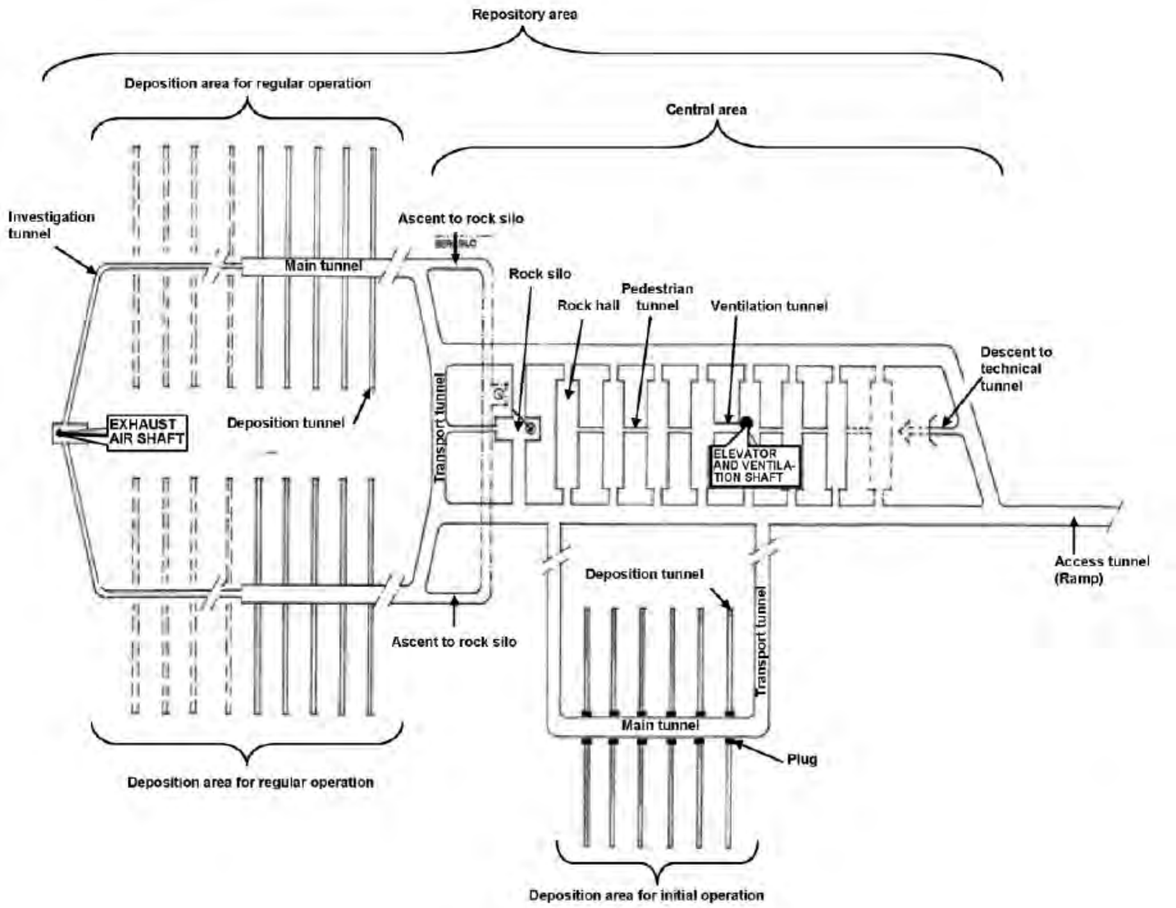


Figure 1-2. *Schematic plan showing certain parts and underground openings.*

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 3,000 m or more, due to anticipated future seismic events on canister integrity /SKB 2004c/.
Margin for Excavation (MFE)	The minimum distance a deposition tunnel or cavern excavation should be from a particular deformation zone from the point of view of ease of construction.
Rock Block (RB)	A rock volume bounded by deterministic deformation zones.
Deposition Block (DB)	The rock volume that is available for deposition after reduction of the rock block volume due to respect distance and margin for excavation.
Deposition Unit (DU)	A group of parallel deposition tunnels within a deposition block.
Rock contour	Actual rock surface surrounding a tunnel, rock cavern, shaft, etc, i.e. outside support, drains, etc.
Internal contour	Actual envelope surrounding the free space in a tunnel, rock cavern, shaft, etc, i.e. inside concrete structure, support, drains, etc.
Theoretical internal contour	Theoretical envelope surrounding the free space in a tunnel, rock cavern, shaft, etc, i.e. inside concrete structure, support, drains, etc.
Theoretical rock contour	Theoretical rock surface surrounding a tunnel, rock cavern, shaft, etc, i.e. outside support, drains, etc.
Design working life	The assumed period for which a structure is to be used for its intended purpose with anticipated maintenance and repair.

2 Design premises and site conditions

2.1 Design methodology

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

For each site the design methodology calls for answering 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, considering assumed preliminary respect distances to deformation zones and loss of deposition holes?

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

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

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

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

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

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

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

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

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

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

I. How much grouting might be required?

J. How much rock support might be required?

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

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

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

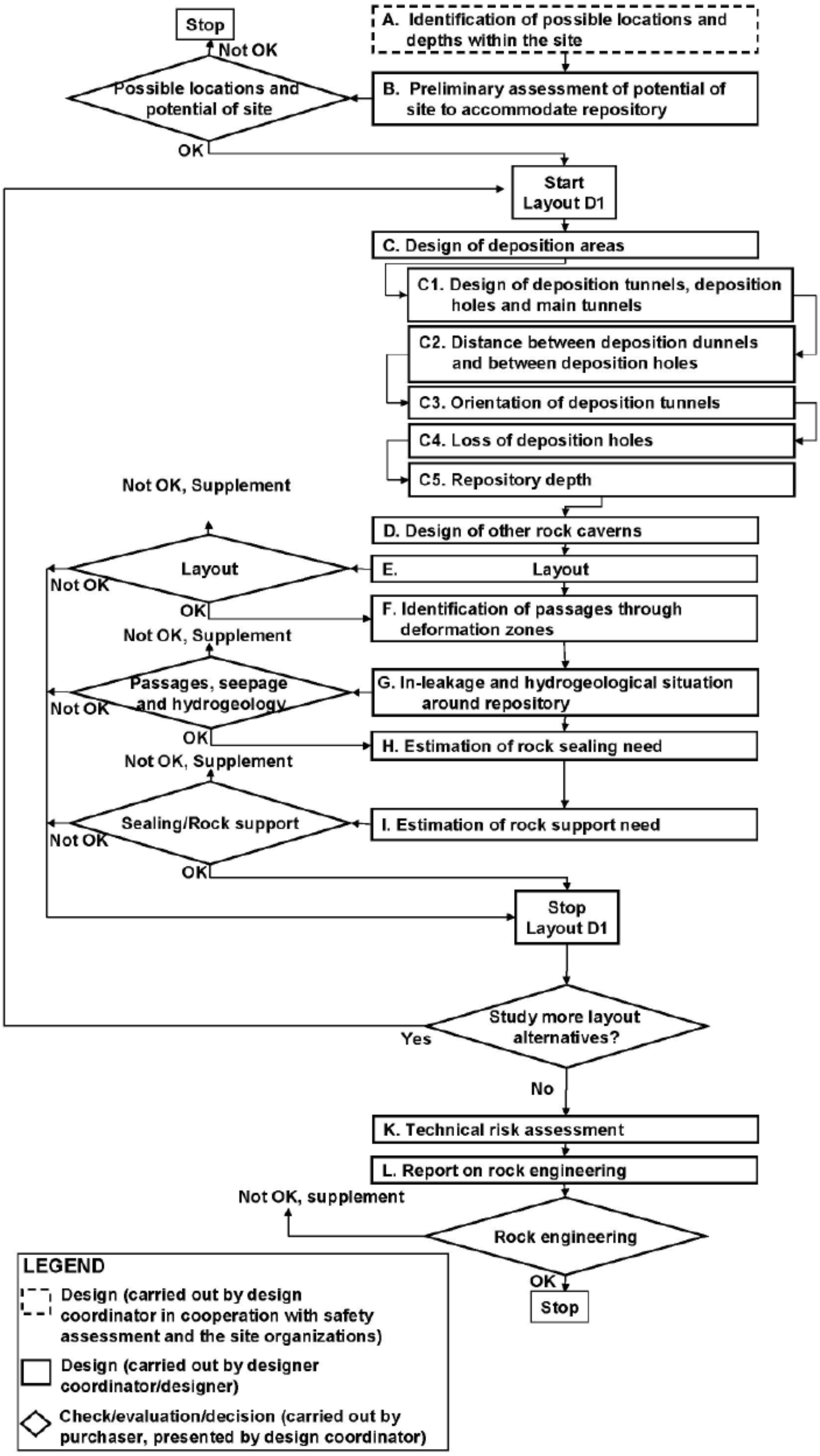


Figure 2-1. *Design methodology, logical framework.*

2.2 Site specific key issues

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

- the site was expected to be intersected by several deformations zones that reduce the available area,
- the rock mass thermal conductivity was expected to be low in deposition units dominated by fine grained diorite,
- the rock mass hydraulic conductivity was expected to be relatively high in deposition units in the northern part of the site in the vicinity of Äspö HRL,
- the rock mass strength was expected to be relatively poor in deposition units dominated by fine grained diorite, compared to other rock types in the area, due to high fracture frequency and great number of fracture sets.

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 above part of the design work proposed 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 preliminary results used. Identified discrepancies are listed in Table 2-1, and it was intended to rectify the analysis only if it was estimated that the final SDM v 1.2 results would not be conservative. The influence on the respective design task concerning new data not applied in analyses are assessed and shown in Table 2-1.

Table 2-1. Major differences between "preliminary data" used in design step D1 and input data from the SDM v 1.2 /SKB 2005a/.

Design task	Chapter in SDM v 1.2	Preliminary data used	Final SDM v 1.2 data	Estimation of influence from new data	Analysis rectified Yes/No
B	App. 4	ZSMEW009A width span 6 m.	ZSMEW009A width span 5–20 m.	Minor	No
C2	7.3	Rock mass thermal conductivity	Rock mass thermal conductivity		
		Standard deviation 0.25 to 0.28 W/mK	Standard deviation 0.20 to 0.28 W/mK	Minor	Yes
		Low and upper confidence interval 2.04 to 3.29 W/mK	Low and upper confidence interval 2.04 to 3.35 W/mK	Minor	Yes
C3–C4	App. 6	Density of dominant rock type 2,663 to 2,783 kg/m^3	Density of dominant rock type 2,681 to 2,803 kg/m^3	Minor	Yes
C3–C4	6.3	Rock mass spalling strength σ_{sm} = 100 to 108 MPa	Rock mass spalling strength σ_{sm} = 76 to 96 MPa.	Moderate	No

Design task	Chapter in SDM v 1.2	Preliminary data used	Final SDM v 1.2 data	Estimation of influence from new data	Analysis rectified Yes/No
C3–C4	6.4	In situ stresses, Domain I σ1 = 0.058z+5 MPa σ2 = 0.027z MPa σ3 = 0.014z+3 MPa	In situ stresses, Domain I σ1 = 0.058z+3 MPa σ2 = 0.028z MPa σ3 = 0.019z MPa	Minor	No
		Uncertainty ± 25%	Uncertainty ± 30%		
		Domain II σ1 = 0.032z+3 MPa σ2 = 0.025z MPa σ3 = 0.01z MPa	Domain II σ1 = 0.032z MPa σ2 = 0.018z MPa σ3 = 0.011z MPa	Minor	No
		Uncertainty ± 25%	Uncertainty ± 30%		

2.3.2 Input from SKB

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

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

SKB have previously carried out studies to identify a general suitable storage depth for the repository KBS-3-system /SKB 2002c/. These studies have concluded that the depth of interest lies within the –400 m to –700 m depth range and that –500 m was determined as being the standard reference level. SKB have prescribed that the reference level should be maintained for the purposes of the current Simpevarp D1 layout even if the results of the current study should indicate that shallower storage depth is more advantageous.

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

The above ground facility currently SKB has targeted to a location in close proximity to Clab. Hålö was considered as an alternative location during the earlier preliminary design work. This design requirement has an effect on the repository layout since the location of shafts and the central area of the repository is restricted by this prerequisite.

During completion of this design report, findings in parallel ongoing studies, Safety Assessment SR-Can /SKB 2006/, revealed that the temperature criteria for the canister and buffer could be changed from 100°C at the canister surface to max 100°C inside the buffer. This indicated the possibility to allow for 10°C higher temperature when evaluating the canister spacing according to Figure 5-4 in UDP /SKB 2004a/. However, SKB decided not to utilise this opportunity, and consequently not to revise the study at this late stage.

2.4 Deviations from the Design Premises

The design works in design step D1 presented in this report has primarily been based on the UDP /SKB 2004a/. However, some amendments have for various reasons been introduced. For example the ongoing R&D work within SKB has given new insight and understanding of studied tasks, such as the analytic method for estimating the probability of canister/fracture intersections in a KBS-3 repository /Hedin 2005/ that overrule suggestions on this matter in the UDP /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 in /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 UDP /SKB 2004a/ is not meaningful, or ought to be carried out differently. All deviations from the strategy outlined in the UDP /SKB 2004a/ are summarised in Table 2-2.

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

Design task	Chapter in this report	Premises according to /SKB 2004a/	Deviation from /SKB 2004a/	Justification
Orientation of deposition tunnels.	4.3	Risk of spalling in deposition tunnels should be analysed by calculations using a 3D finite element or difference analysis.	Calculation of the tangential stresses on the deposition tunnel periphery analytically.	Parallel studies /Martin 2005/ have given additional and valuable information. Decision by SKB.
Orientation of deposition tunnels.	4.3	Volume of unstable wedges should be analysed by generating a stochastic fracture network.	Analysed by kinematic block analysis.	Simplified analysis which is relevant since the parameter is not expected critical for the orientation of tunnels.
Loss of deposition holes.	4.4	Numerical DFN-method for stochastically generated fractures with R > 100 m.	Additional results from analytical method as proposed in Preliminary Safety Evaluation for Simpevarp /SKB 2005b/ for stochastically generated fractures with 100 m < R ≤ 200 m.	Additional less time consuming calculation method. Re-evaluated limits for fractures discernible during the construction period. These results came late in the design work, after assessment of loss and the following layout. Decision by SKB.
		The minimum permissible distance between a deposition hole and a stochastically determined fracture should be 2 m for fractures 100 < R ≤ 200 m and 0.01R for fractures R > 200 m.	For the fractures R > 100 m the minimum distance is assessed to 2 m.	Simplifications that are not judged to have a large influence and on the conservative side.
Loss of deposition holes	4.4	Volume of unstable wedges should be analysed by generating a stochastic fracture network.	Analysed by kinematic block analysis.	Simplified analysis which is relevant since the parameter is not expected critical for loss of deposition holes.

Design task	Chapter in this report	Premises according to /SKB 2004a/	Deviation from /SKB 2004a/	Justification
Loss of deposition holes	4.4	Risk of spalling in deposition tunnels should be analysed by calculations using a 3D finite element or difference analysis.	Calculation of the tangential stresses on the deposition tunnel periphery analytically.	Parallel studies /Martin 2005/ have given additional and valuable information. Decision by SKB.
Identification of passages through deformation zones	6	Identification shall be based on the Simpevarp SDM v 1.2.	Supplementary information has been taken from local excavation experience.	As agreed by SKB additional information can be utilized if site specific input as long as this information is well documented and described.
Seepage and hydrogeological situation around repository	7	Numerical analyses should be made with both of the software tools Darcy Tools and Connect Flow.	Analyses have been made only with Darcy Tools.	Decision by SKB.
Estimation of rock grouting need	8	A) Mortar mixtures should be assessed.	A) Low-pH cement is the only alt presented.	A) SKB decision.
		B) Rock mass porosity should be according to P_{33}.	B) This value was missing at the time of the study, why the porosity was estimated from the hydraulic conductivity.	B) Due to timing issues, the alternative analysis was carried out.

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

3.1 Possible location

The possible location for a tentative Deep Repository has been defined by SKB to lie within the *Simpevarp interest area* /SKB 2003/. The area has been further restricted by taking deformation zones ZSMNE005A and ZSMNE024A to mark the western and eastern boundaries respectively. The study area is shown in grey in Figure 3-1.

3.2 Division of bedrock into rock domains

The bedrock in the Simpevarp subarea is divided into four rock domains, domain A – Ävrö granite, B – fine-grained diorite, C – mixture of Ävrö granite and quartz monzodiorite, D – quartz monzodiorite. The area taken up by each rock domain on level –400 m is presented Figure 3-2. From the map it is clear that rock domain A dominates the main part of the area followed in size by rock domains B and C. Rock domain D lies outside the study area.

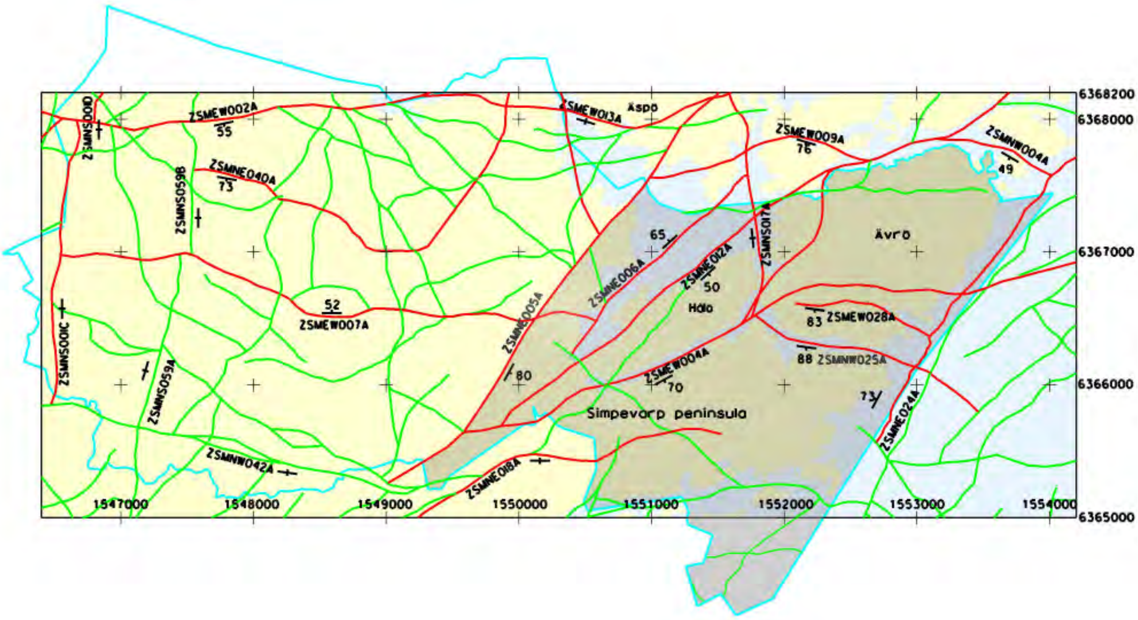


Figure 3-1. Local model area Simpevarp v 1.2, Interest area for the storage facility and the current study area shown in grey. The red and green lines represent the deformation zones. (Section level at z = 0 m).

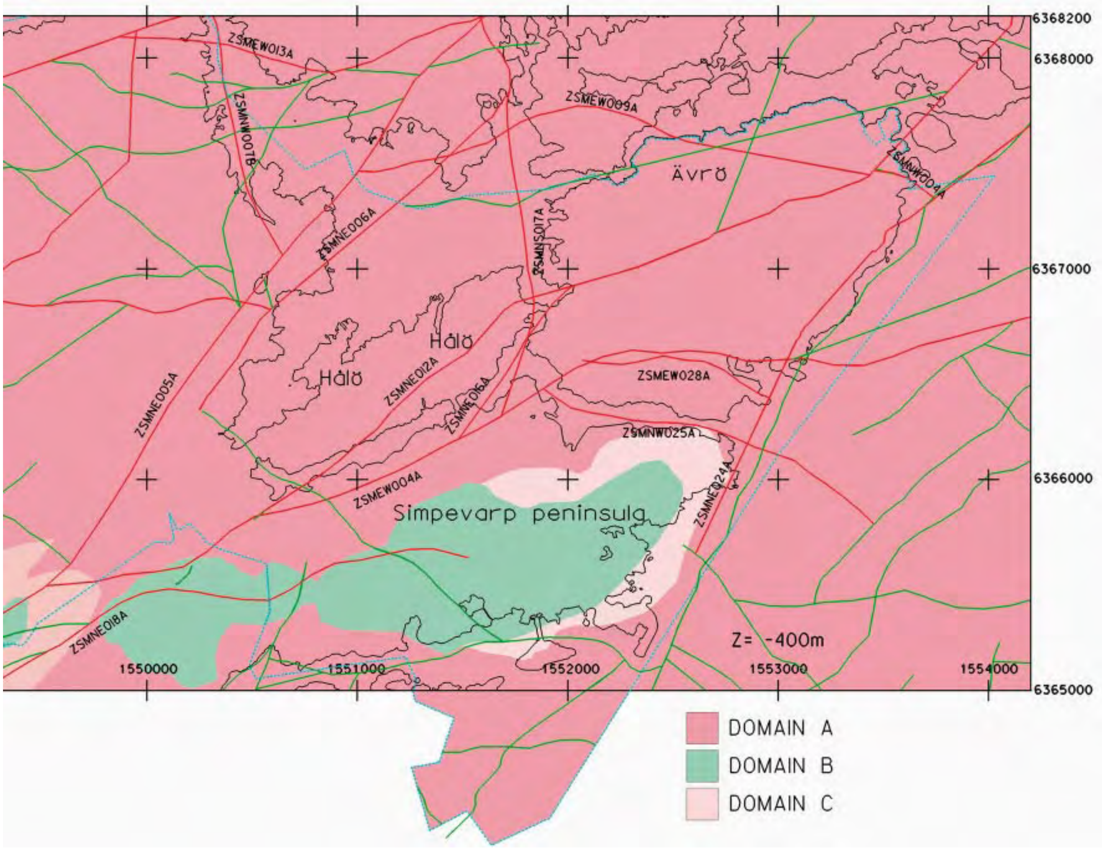


Figure 3-2. *Map of the bedrock division into rock domains in the Simpevarp subarea on level –400 m. Domain A – Ävrö granite, Domain B – Fine grain dioritoid, Domain C – Mixture of Ävrö granite and quartz monzodiorit. (Modified after SKB, 2005a).*

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

3.3.1 Input data and assumptions

The initial basis for the work is the geometrical framework defined by the deformation zones. The zone geometries have been taken from the Simpevarp site descriptive model version 1.2 /SKB 2005a/. The study area is criss-crossed by such zones, which need to be taken into account by the deposition and cavern excavations. The transport tunnels will have to pass through these zones and the likely rock conditions and excavation problems need to be assessed as well as potential for water inflows and long term stability. This has resulted in the definition of two types of safe working distances: *Respect Distance* (RD) based on assessed earthquake susceptibility and *Margin for Excavation* (MFE) based on rock quality.

The concept *Respect Distance* is defined in SR-Can Interim /SKB 2004c/ and is the minimum permissible distance between a deposition hole and a deformation zone with a trace length of 3,000 m or more, due to anticipated future seismic effects on canister integrity.

The concept *Margin for Excavation* has been developed and introduced by the designer during the current study and represents the minimum distance a deposition tunnel or cavern excavation should be from a particular zone from the point of view of ease of construction.

The presence of high confidence zones were taken into account for levels –400, –500, –600 and –700 m. The inclusion of “possible” zones was confined to levels –400 and –500 m.

A deformation zone is classed as high confidence if evidence for its existence has been confirmed by geophysical ground control, borehole, tunnel or field mapping. A “possible” zone is based solely on topography and the results from aerial geophysical surveys. In order for a potential deformation zone to be included in the models it must have a surface trace length of at least 1,000 m. Structures shorter than this cut-off length are dealt with by stochastic modelling.

Twelve high confidence deformation zones have been identified within the study area. The location of these zones at –400 m and –700 m are presented in Figures 3-3 and 3-4. The lateral shift in some of the zone positions with depth is due to the dipping nature of the structures.

3.3.2 Execution

The work has been carried out in accordance with SKB’s design guidelines Deep Repository – Underground design premises, Edition D1/1 /SKB 2004a/ and criteria for “respect distance” and partly for “margin for excavation” from SR-CAN report /SKB 2004c/.

The aim of the study is to make a preliminary assessment of the site’s ability to accommodate a total of 6,000 canisters at –400, –500, –600 and –700 m depths.

The work was based initially on the geometrical framework defined by the deformation zones. The deformation zone geometries have been taken from the Simpevarp preliminary site description model S1.2 /SKB 2005a/. The design layout principles allow for any transport tunnel to be excavated through any such zone. An assessment of the potential problems of excavating a tunnel through the different zones has been based on information currently available and details are presented in Chapter 6 of this report.

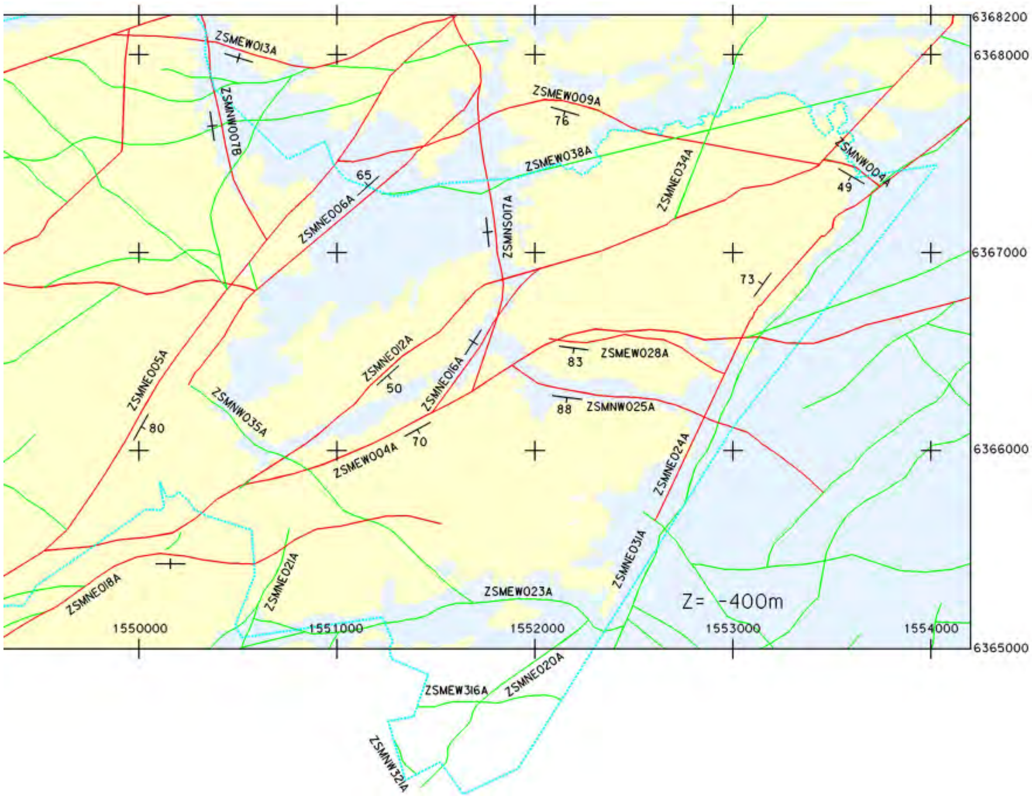


Figure 3-3. *Deformation zones, level –400 m. High confidence zones are shown in red, “possible” zones in green /SKB 2005a/.*

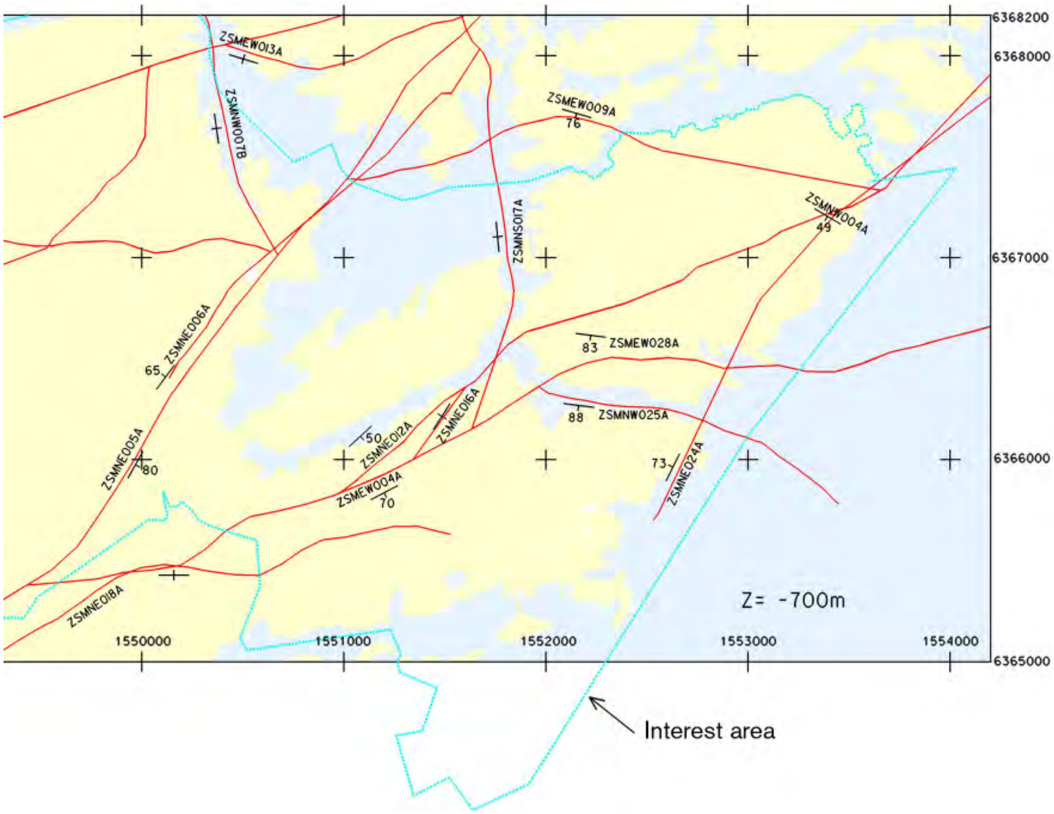


Figure 3-4. *Deformation zones level –700 m. High confidence zones are shown in red, /SKB 2005a/.*

Each zone was assessed on its potential earthquake susceptibility and its associated rock quality resulting in a quantified Respect Distance (RD) according to SR-Can Interim /SKB 2004c/ and a Margin for Excavation (MFE). RD and MFE are applied to both high and "possible" zones. For any zone with a trace length of 3,000 m or more the larger of the two limits, whether it be RD or MFE, is applied, see Figures 3-5 and 3-6.

The deformation zones divide the rock volume up into a series of *rock blocks* (RB). Since deposition holes can only be placed outside the deformation zone RD and MFE envelopes the available deposition volume is reduced. The available rock volumes within these smaller defined blocks are termed *deposition blocks* (DB). Further, it was assumed that an additional 25% of the potential deposition hole sites are unavailable due to unfavourable local rock conditions.

A preliminary assessment of the site's potential to accommodate 6,000 canisters at a particular depth is presented based on the calculation of a *P* value. P is an SKB defined measure of site potential. A *P* value of 1 indicates the site has sufficient capacity; P < 1 indicates insufficient capacity whilst P > 1 indicates over capacity.

Respect distance

Respect distances are only applied to those zones with surface trace lengths of 3,000 m or more. The respect distance is applied to both sides of a zone and measured perpendicularly from a defined central plane. For zones with traces of 3,000 m or more the RD is taken to be equal to the zone's width with a minimum value of 100 m applied to any deposition hole /SKB 2004c/.

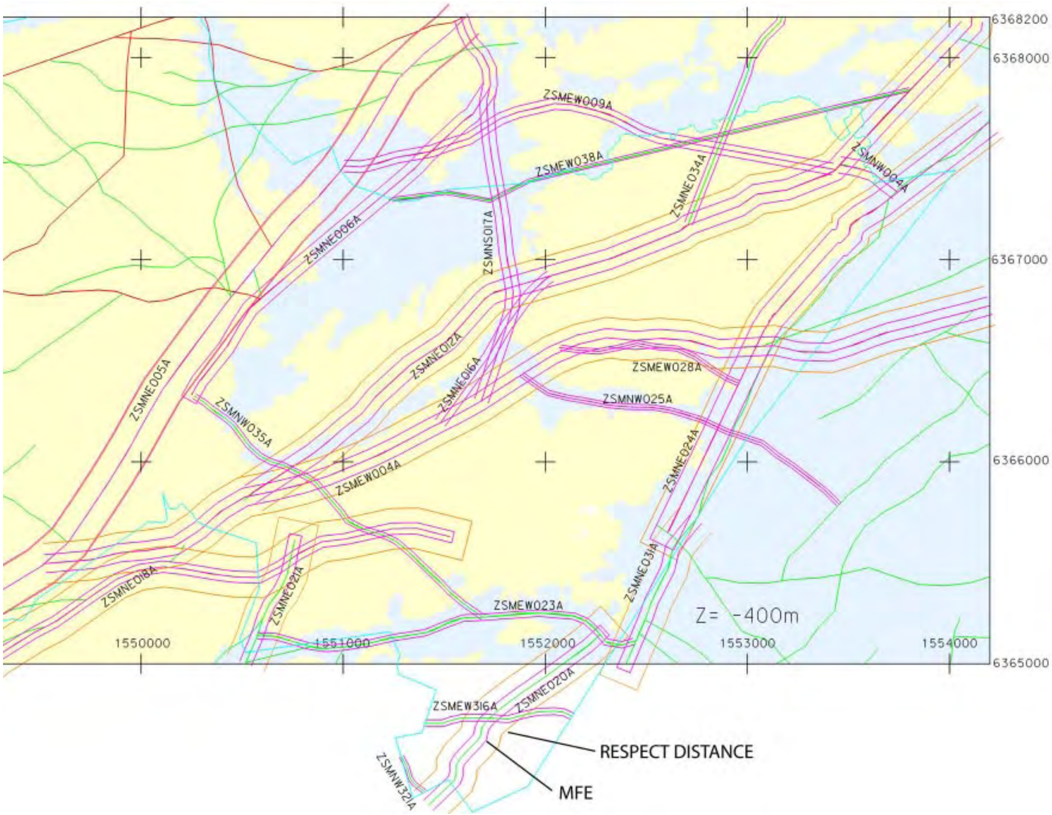


Figure 3-5. *RD and MFE for deterministic deformation zones lying within the study area. Level –400 m.*

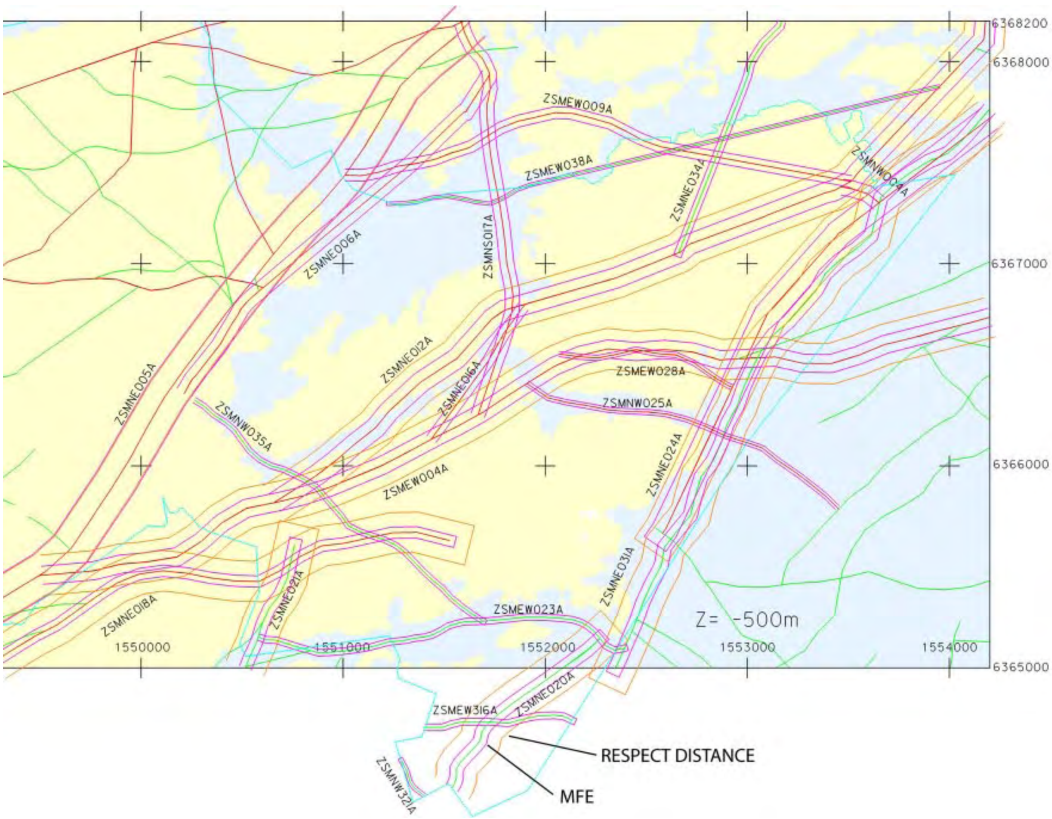


Figure 3-6. *RD and MFE for deterministic deformation zones lying within the study area. Level –500 m.*

Margin for excavation

Margin for excavation is the minimum distance a deposition tunnel or cavern excavation should be from a particular deformation zone from the point of view of ease of construction.

MFE is based on the zone's width including assumed variations and the application of a further safety margin (SM).

Where information was available, deformation zone widths have been taken from the site descriptive model /SKB 2005a/. If no information was available concerning variation then half the zone width has been used.

The safety margin (SM) is based on the deformation zone properties and an assessment of likely construction and maintenance issues. An SM associated with an inferred stability problem is taken to be in the order of two blasting rounds (10 m). An SM associated with water problems is taken to be in the order of a standard grouting section length (20 m). Where no information was available a default value of 5 m was used. The following equation was applied to high confidence zones:

$$MFE = \left(\frac{Width + |Variation|}{2} \right) + SM \qquad \text{Equation 3-1}$$

where,

SM equals to 20 m in the case of an inferred water problem, 10 m in the case of an inferred stability problem and 5 in the case of no available information.

"Possible" zone lengths were taken from the site descriptive model /SKB 2005a/, however, no information was available concerning their widths or likely variation. In the case of "possible" zones the width was assumed to be equal to 1% of the zone's trace length /SKB 2004c/. An assumed SM value of 5 m was applied in such cases.

The following equation was applied to "possible" zones:

$$MFE = \left(\frac{0.01 \cdot (Zone\_length)}{2} \right) + 5 \qquad \text{Equation 3-2}$$

The resulting RD and MFE values have been applied to the model zone geometries and are presented in Section 3.3.3.

3.3.3 Results

The resulting RD and MFE values have been applied to the model zone geometries and are presented in Figures 3-5 and 3-6.

Available rock volume for deposition

The position and volume of the *Deposition blocks* (DB) is depth dependent due to the dipping nature of some of the modelled deformation zones. The distribution of DB's for –400 and –500 m levels are presented in Figures 3-7 and 3-8.

The total available rock volume at any one level is represented by the sum of all of the individual DB areas for that level, seen in a 2D horizontal plane. This summed area is defined as a level's A_T value /SKB 2004a/. Taking into account the existence of only high confidence deformation zones, A_T values for the current study levels vary between 4.73 and 4.82 km².

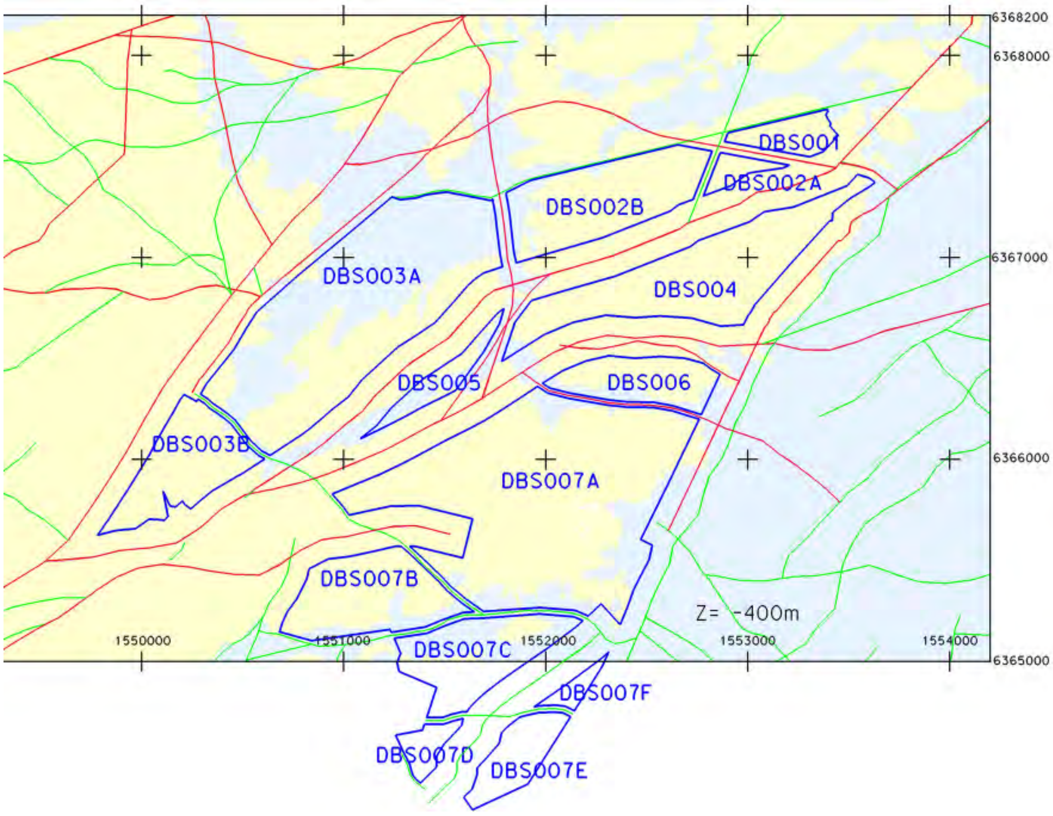


***Figure 3-7.** Distribution of available rock volumes, DB's outlined in blue, –400 m level.*

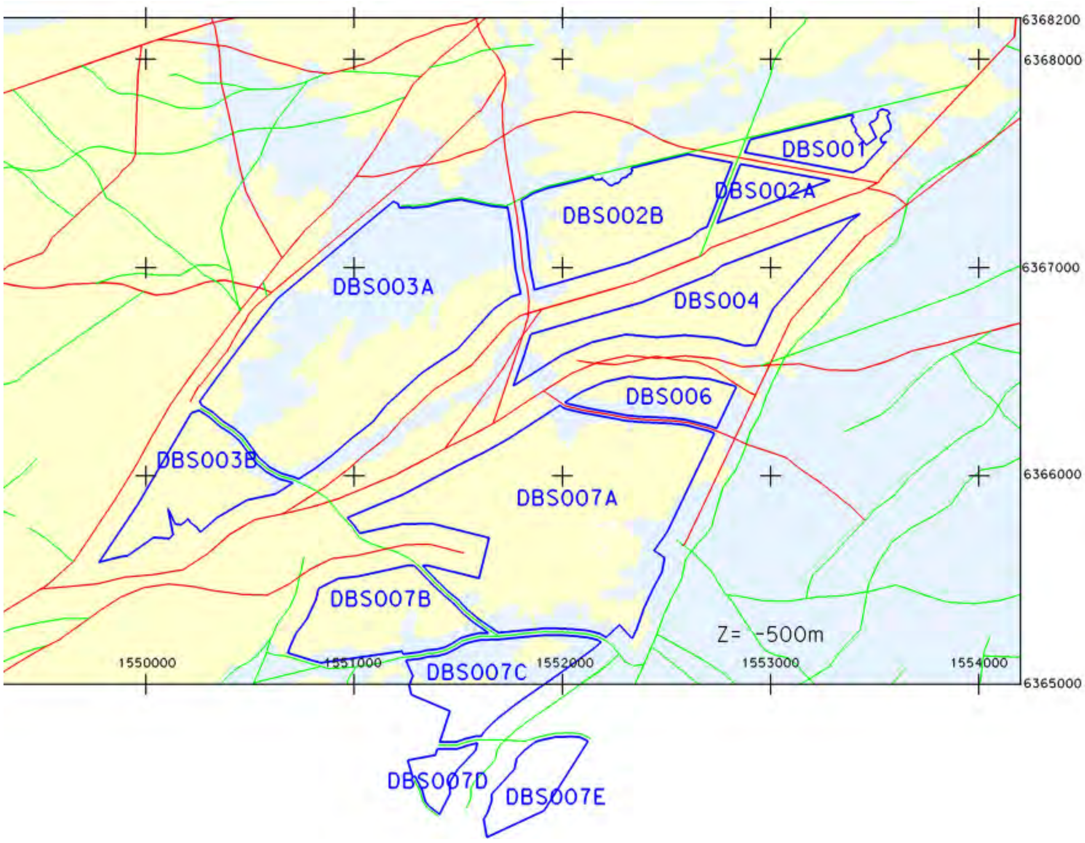


***Figure 3-8.** Distribution of available rock volumes, DB's outlined in blue, –500 m level.*

This 2% degree of variation with depth can be said to be marginal and therefore, from the view of available rock volumes, the positioning of the storage facility is not depth sensitive within the limits of the current study. Results are presented in Figure 3-9.

A more detailed focused assessment was made at –400 m and –500 m levels where the existence of "possible" zones was also taken into account. The A_T values were calculated to be 4.16 km² (–400 m) and 4.12 km² (–500 m), see Table 3-1. This represents a variation of 1% between the two levels. Results are presented in Figure 3-10.

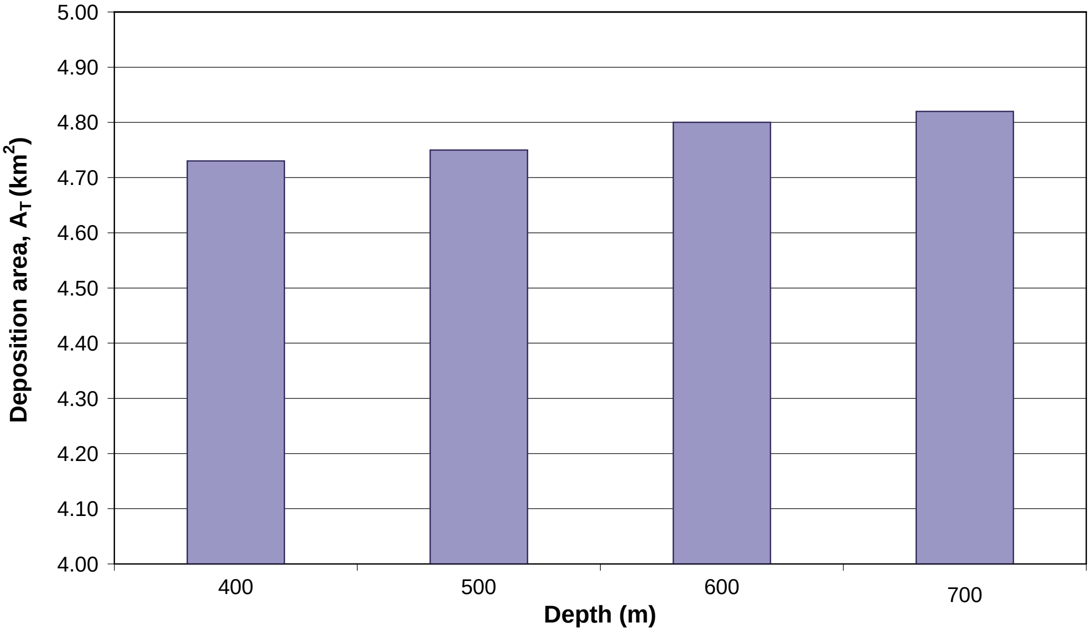


Figure 3-9. *Variation of A_T with depth considering high confidence zones.*

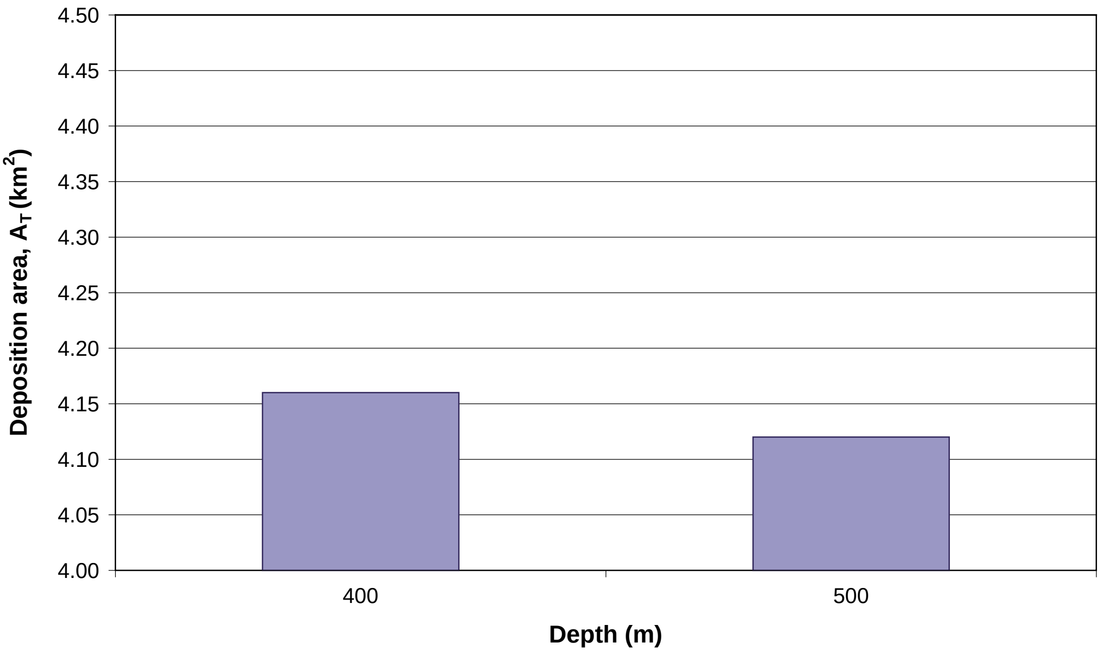


Figure 3-10. *Variation of A_T with depth considering both zones with the confidence level high and possible.*

Table 3-1. Deposition block area and potential for deposition according to Figures 3-7 and 3-8.

Deposition block (DES)	Area (km²)		Potential for deposition	
	Level –400 m	Level –500 m	Level –400 m	Level –500 m
001	0.08	0.11	0.04	0.05
002A	0.04	0.06	0.02	0.03
002B	0.33	0.39	0.15	0.17
003A	0.90	1.08	0.40	0.46
003B	0.24	0.26	0.11	0.11
004	0.46	0.33	0.20	0.14
005	0.07	0.00	0.03	0.00
006	0.16	0.13	0.07	0.06
007A	1.15	1.07	0.50	0.45
007B	0.28	0.27	0.12	0.11
007C	0.25	0.25	0.11	0.11
007D	0.05	0.05	0.02	0.02
007E	0.12	0.12	0.05	0.05
007F	0.03	0.00	0.01	0.00
Total	4.16	4.12	1.82	1.75

The inclusion of "possible" zones in the assessment resulted in a 12–13% drop in overall A_T values.

It should be noted that A_T values take no account of the geometrical form of the available area. Clearly some forms may be utilized more efficiently than others, whilst some narrow forms are essentially unusable and should be discounted.

Preliminary assessment of the available rock volume to accommodate the storage facility

The key question in the current study is whether there is a sufficient rock volume, with suitable characteristics, to safely contain the specified 6,000 deposition canisters.

A preliminary assessment of the answer to this question is assisted by the calculation of parameter *P*, (Potential for deposition).

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

where,

K = Assumed percentage of loss of deposition locations within each deposition block. Defined by UDP to be 25% for the current study.

N = Preliminary number of required canister positions for the study purposes. Taken as 6,000 for the current study.

A_T = Available deposition area, in accordance with /SKB 2004a/.

A_S = Specific surface area per deposition hole (Canister spacing × 40 m^2). The canister spacing varies depending on thermal properties of the surrounding rock mass see Table 4-4, Section 4.2.3.

A *P* value of 1 indicates the site has sufficient capacity; *P* < 1 indicates insufficient capacity whilst *P* > 1 indicates over capacity.

The calculated Preliminary potential, *P*, varies between 1.78–2.07 considering high confidence zones and all study depths. This result indicates the site has clear potential to accommodate a storage facility, see Figure 3-11.

If "possible"as well as high confidence deformation zones are taken into account the *P* value falls by 12%.

3.3.4 Discussion

Generally the shallower the level the more favourable it appears to be from the point of view of rock properties generally and in particular beneficial for thermal, hydrogeological and in situ stress conditions.

The in situ rock mass temperature increases with depth and resulting in an increase in the required spacing between canisters.

Groundwater pressures increase with depth leading to a higher potential for increased water inflows to the tunnels and canister sites. This in turn increases the costs of excavation.

3.3.5 Conclusions and recommendations

To a large extent it is simply the relative size and shape of the individual deposition blocks that determines whether or not they are recommended for further study. The number and location of deterministic deformation zones that are modelled in the area has a large impact on the area available for deposition.

DBS003A and DBS007A are judged to be the most attractive deposition blocks, with any additional expansion directed towards the northern and the southern boundary of the studied area. The available deposition area in these two blocks is A_T = 2.15 km² for level –500 m, which corresponds to a calculated potential for deposition P = 0.9.

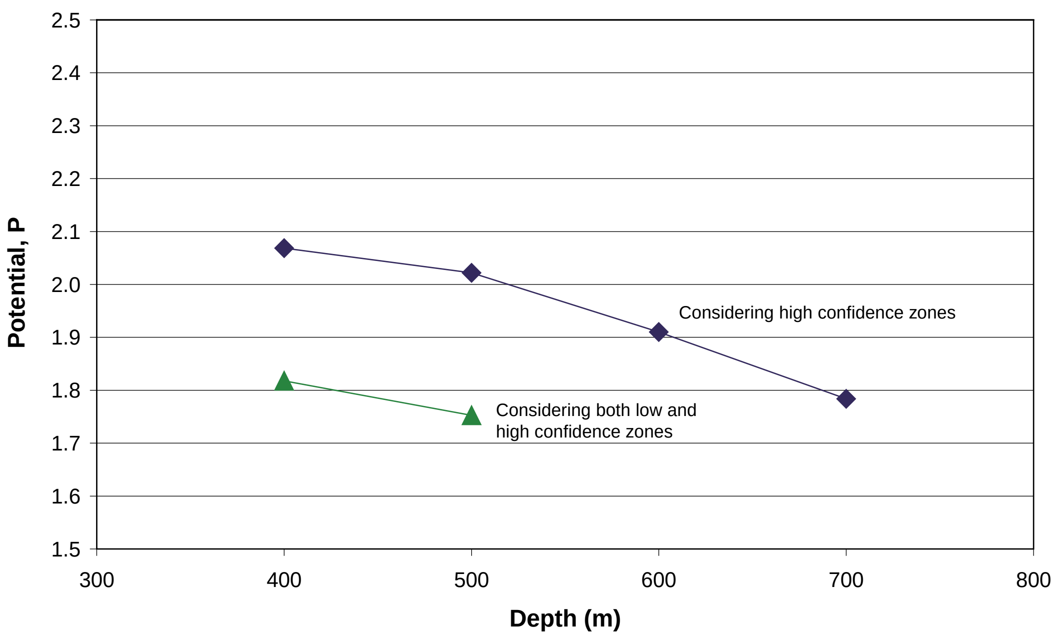


Figure 3-11. *Variation of site storage potential P with depth.*

4 Design of deposition areas

4.1 Design of tunnel geometries

4.1.1 Demands

For design step D1, no requirements are made on specific documentation of the design of tunnels, since this is largely done in accordance with Layout E /SKB 2002a/ and /SKB 2002b/.

The tunnel sizes and associated measurements are described in the UDP and take into account:

1. the required space for the equipment and installations required for ventilation, transport of rock spoil, investigation of the rock, preparation and cleaning of deposition holes, deposition of buffer and canisters, backfilling and temporary plugging,
2. the possibility of canister retrieval,
3. the minimum required distance between deposition holes and main tunnel with a view towards:
   - the stress state around the first deposition hole due to stress redistribution around the main tunnel,
   - the position of the concrete plug in relation to the fracturing in the rock mass due to unilateral water pressure on the concrete plug,
4. minimum required distance between deposition holes and tunnel end,
5. stability.

Points 3 and 4 provide information on how much of the deposition tunnels that cannot be utilized for deposition holes due to the location of the temporary plug and space requirements for the deposition equipment.

Design of deposition holes presented in the UDP and Layout E take into account:

6. the required space for deposition of buffer and canisters,
7. the possibility of canister retrieval,
8. stability.

Design of main tunnels presented in the UDP and Layout E take into account:

9. the required space for the equipment and installations required for ventilation, transport of rock spoil, investigation of the rock, preparation and cleaning of deposition holes, deposition of buffer and canisters, backfilling and temporary plugging,
10. stability.

4.1.2 Theoretical tunnel geometries and dimensions

In the previous work carried out by SKB in the so-called Layout E /SKB 2002a/ and /SKB 2002b/ tunnel geometries and dimensions were recommended for the design work. Figures 4-1 to 4-3 shows the tunnel sections and distances used for the design tasks.

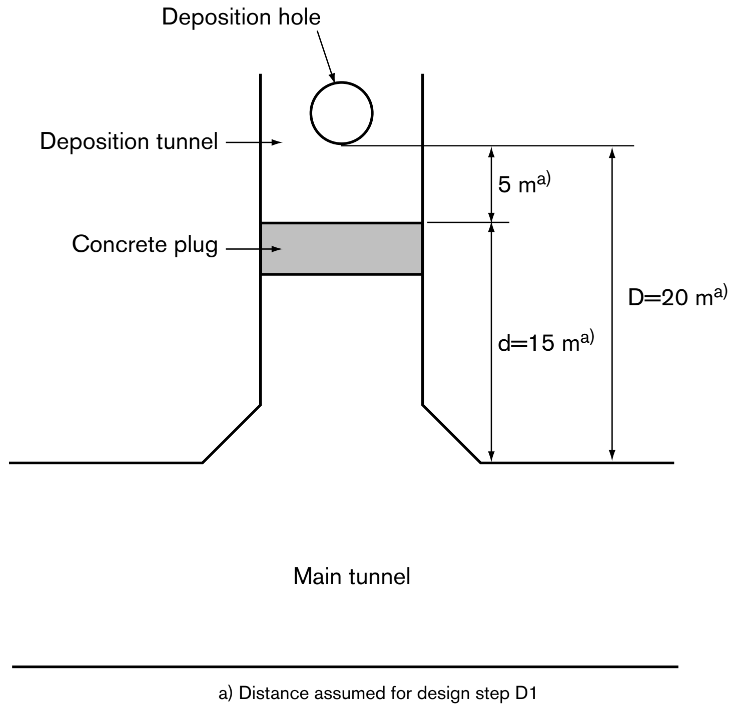


Figure 4-1. *Schematic layout of main tunnel, deposition tunnel and deposition holes. /SKB 2004a/.*

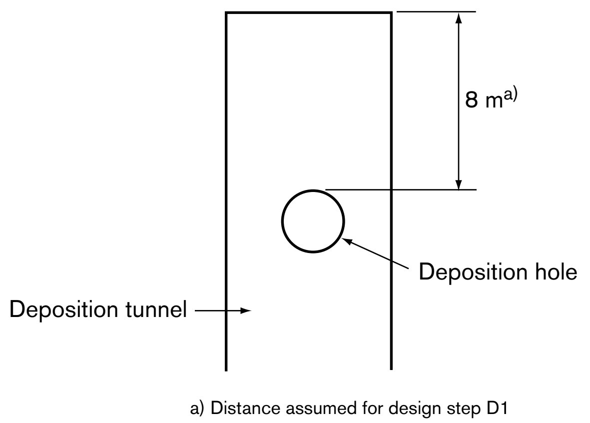


Figure 4-2. *Distance between deposition holes and tunnel end assumed for design step D1. /SKB 2004a/.*

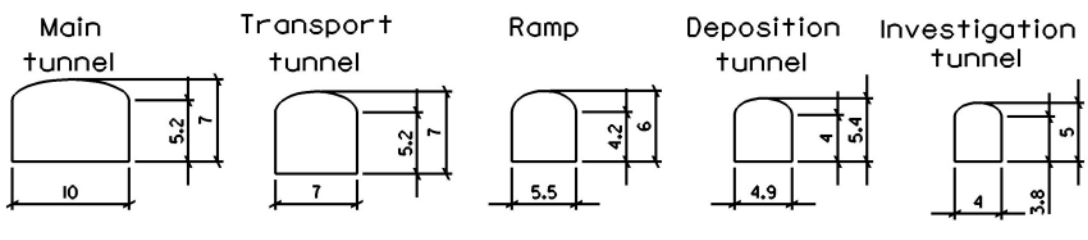


Figure 4-3. *Tunnel sections. Unit in meters. Revised after /SKB 2002a/.*

4.2 Distance between deposition tunnels and deposition holes

4.2.1 Input data and assumptions

The aim of this aspect of the study is to determine the minimum allowable distance between deposition holes and deposition tunnels, based on an assumed maximum allowable canister surface temperature of 100°C. The thermal properties of the rock mass and the thermal gradient at the site have been assessed with reference to the maximum temperature allowed at the canister surface. This has lead to the definition of a minimum deposition hole spacing for each rock domain and potential storage level studied. The storage levels considered in the current study are –400, –500, –600 and –700 m.

The following parameters have been taken into account:

- the rock mass thermal properties,
- initial in situ temperature for each potential storage level,
- the canister heat output,
- the bentonite buffer and its thermal properties.

In accordance with UDP the following assumptions were made:

- distance between the deposition tunnels: 40 m (c/c),
- initial canister heat output: 1,700 W/canister,
- thermal conductivity of the buffer: 1.0 W/mK.

A maximum allowable canister surface temperature of 80°C was assumed, after taking into account the uncertainties in the input data and the air gap between buffer and canister /Hökmark and Fälth 2003/.

Input data for the analysis has been taken from the Simpevarp site descriptive model version 1.2 /SKB 2005a/ and /Sundberg et al. 2005/.

4.2.2 Execution

The work has been carried out in accordance with Section 5.4.2 of SKB's design guidelines-Deep Repository – Underground design premises, Edition D1/1 /SKB 2004a/.

Thermal conductivity and heat capacity of the rock mass

The rock mass thermal properties are largely dependent on the thermal conductivity and specific heat capacity of the rock mass. However, these parameters are investigated by testing a very small volume of rock material in the laboratory, approximately 10 cm^3. In an attempt to overcome discrepancies associated with these scale differences, the site modelling included an assessment of the spatial variation of thermal properties within the rock domains, followed by an analysis of the thermal conductivity at a scale of 0.1–60 m.

In the site descriptive model the final presented results of thermal conductivity were based on a scale of 0.75 m. The reasoning behind the selection of a scale in the lower range of those investigated was due to the judged uncertainties in the analysis and the uncertainty of which scale was most relevant for canister emissions /SKB 2005a/. Results are presented in Section 4.2.3, Table 4-1 and 4-2.

Initial in situ temperature at repository depth

Temperature as a function of depth has been measured in six boreholes: KSH01A, KSH02, KSH03A, KAV01, KLX01 and KLX02. Results are presented in Section 4.2.3, Table 4-3.

Verification of canister spacing

Verification of the canister spacing for each rock domain and depth interval was carried out using the temperature-spacing relationship presented in Figure 4-4.

The assessment assumes an initial rock temperature of 15°C and a rock heat capacity of 2.08 MJ/m^3, K. The measured heat capacity is approximately 10% higher than this assumed value resulting in a more favourable relationship, see Table 4-2. This difference in the rock heat capacity is judged to reduce the canister surface temperature by around 1°C /Hökmark and Fälth 2003/. Results are presented in Section 4.2.3, Table 4-4 and Figure 4-6.

4.2.3 Results

Thermal conductivity and heat capacity of the rock mass

The results from the assessment of the thermal conductivity of the rock domains are presented in Table 4-1. The bedrock division into rock domains is presented in Section 3.2. A normal distribution is assumed.

The mean thermal conductivity of the rock mass varies between 2.74 and 2.80 W/mK for rock domains A to C, which are the focus for the design work. Note that the thermal conductivity values presented in Table 4-1 are applicable at 20°C. At higher temperatures the thermal conductivity is marginally lower, with a fall of approximately 1–3% per 100°C temperature increase. However, no adjustment is made for this or other uncertainties included in the assessment of deposition hole spacing based on Figure 4-4.

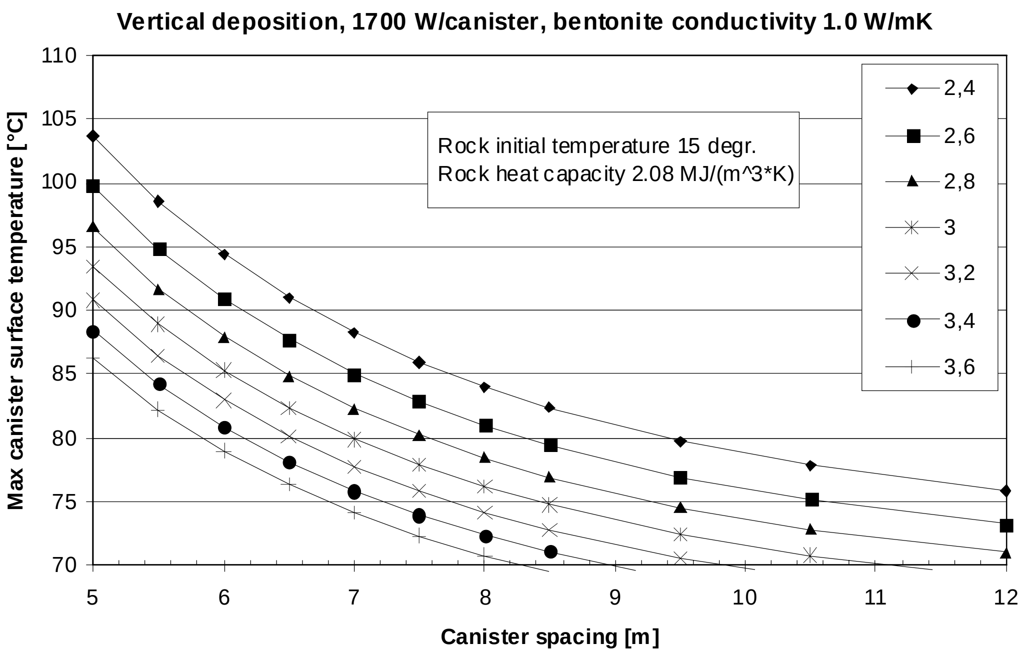


Figure 4-4. *Maximum canister surface temperature vs canister spacing for varying values of rock thermal conductivity /SKB 2004a/.*

Table 4-1. Thermal conductivity of the rock domains assessed at a scale of 0.75 m /SKB 2005a/.

Rock domain	Mean (W/mK)	Std dev (W/mK)	Lower limit at 95% confidence	Upper limit at 95% confidence
Domain A, Ävrö granite	2.80	0.28	2.25	3.35
Domain B, fine grained dioritoid	2.74	0.20	2.35	3.13
Domain C, mixture of quartz-monzodiorite/ Ävrö granite	2.74	0.24	2.27	3.21
Domain D, quartz monzodiorite	2.62	0.28	2.04	3.20

The measured heat capacity for the rock domains within the Simpevarp sub area is presented in Table 4-2. A normal distribution is assumed. The mean heat capacity is around 2.23 MJ/m³K for rock domains A to C, which are the focus for the design work. With consideration to the temperature increase around a canister, it is considered appropriate to assume a 10–15% increase in the rock mass heat capacity. The temperature dependency of the heat capacity is approximately 25–30% increase per 100°C temperature increase for the current rock (SKB, 2005a). When assessing deposition hole spacing, this is carried out according to Figure 4-4 which is based on a fixed heat capacity (2.08 MJ/m³K).

Table 4-2. Heat capacity of the rock domains /SKB 2005a/.

Rock domain	Mean MJ/(m³×K)	Std dev MJ/(m³×K)	Lower limit at 95% confidence	Upper limit at 95% confidence
Domain A, Ävrö granite	2.23	0.12	2.00	2.46
Domain B, fine grained dioritoid	2.23	0.10	2.04	2.42
Domain C, mixture of quartz-monzodiorite/ Ävrö granite	2.24	0.09	2.04	2.42
Domain D, quartz-monzodiorit	2.25	0.06	2.11	2.38

Initial in situ temperature at repository depth

In situ temperature measurements recorded in five of the six site boreholes are presented in Table 4-3. Values are based on background data to Figure 4-5 /Sundberg et al. 2005/. Results from KSH03 have not been included since they have been judged as unreliable.

The results show that temperatures increase from approximately13°C at –400 m to approximately 17.5°C at –700 m. The temperature gradient over the same depth interval within the Simpevarp sub area is approximately 15.0°C/km /SKB 2004b/.

Table 4-3. Measured borehole temperatures between levels –400 to –700 m in the Simpevarp-Laxemar area. Note: KLX01 terminated at –673 m. Background data to Figure 4-1 /Sundberg et al. 2005/.

Borehole	Level –400 m (°C)	–500 m (°C)	–600 m (°C)	–700 m (°C)
KSH01A, 2003	12.97	14.34	15.80	17.30
KSH02, 2003	13.32	14.69	16.12	17.59
KAV01, 2003	12.55	14.62	16.31	17.83
KLX01	13.67	15.35	16.92	–
KLX02, 2003	13.36	14.82	16.32	17.85
Mean	13.2	14.8	16.3	17.6

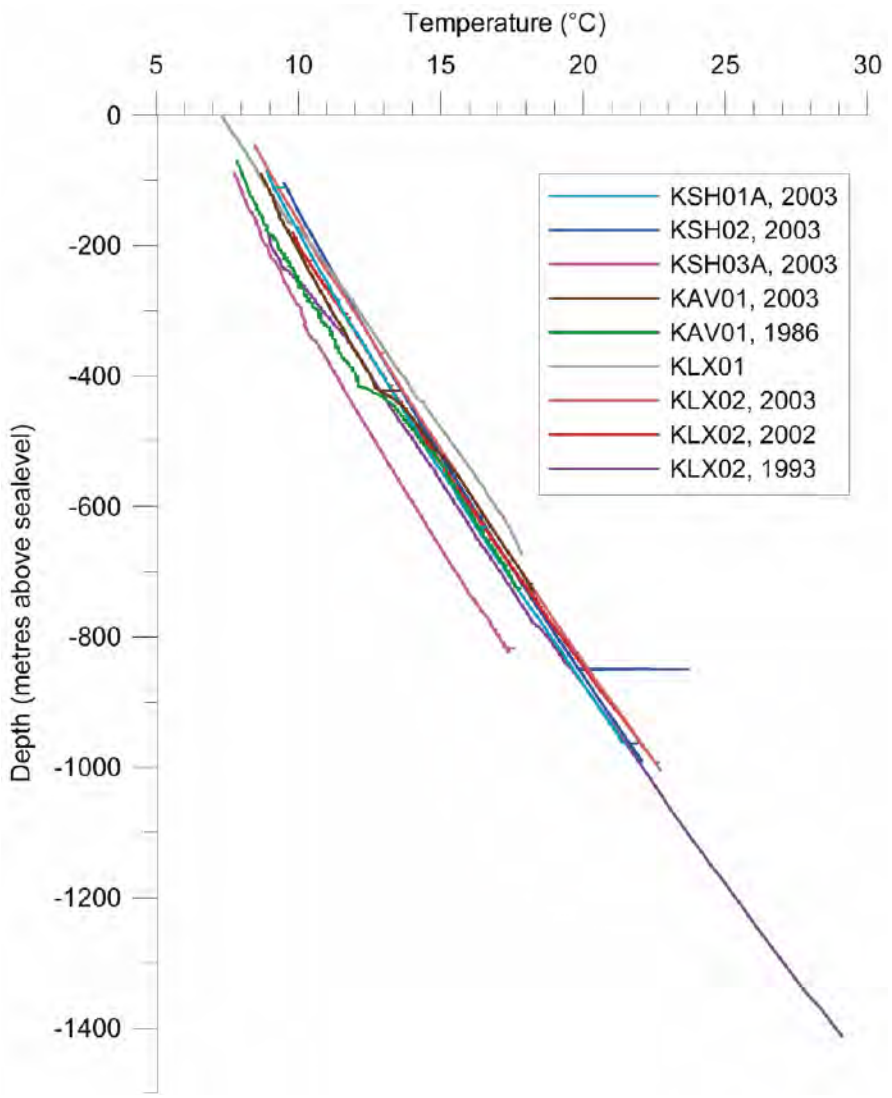


Figure 4-5. *Measured temperature as a function of depth, from boreholes in the Simpevarp-Laxemar local model area /SKB 2005a/.*

Verification of canister spacing

The results from the verification of the canister spacing for each rock domain and depth interval are presented in Table 4-4 and Figure 4-6. For rock domains A to C, which are of interest for the current design, the minimum allowable canister spacing varies from 7.1 to 7.3 m at –400 m depth and between 8.3 to 8.5 m at –700 m depth.

Table 4-4. Minimum allowable canister spacing as a function of storage depth, based on mean values of thermal conductivity for each rock domain.

Depth	Initial temperature	Max canister surface temp	Canister spacing (m)			
			Domain A	Domain B	Domain C	Domain D
(m)	(°C)	(°C)	(λ = 2.80)	(λ = 2.74)	(λ = 2.74)	(λ = 2.62)
400	13.2	81.8	7.1	7.3	7.3	7.7
500	14.8	80.2	7.5	7.7	7.7	8.2
600	16.3	78.7	7.9	8.1	8.1	8.7
700	17.6	77.4	8.3	8.5	8.5	9.3

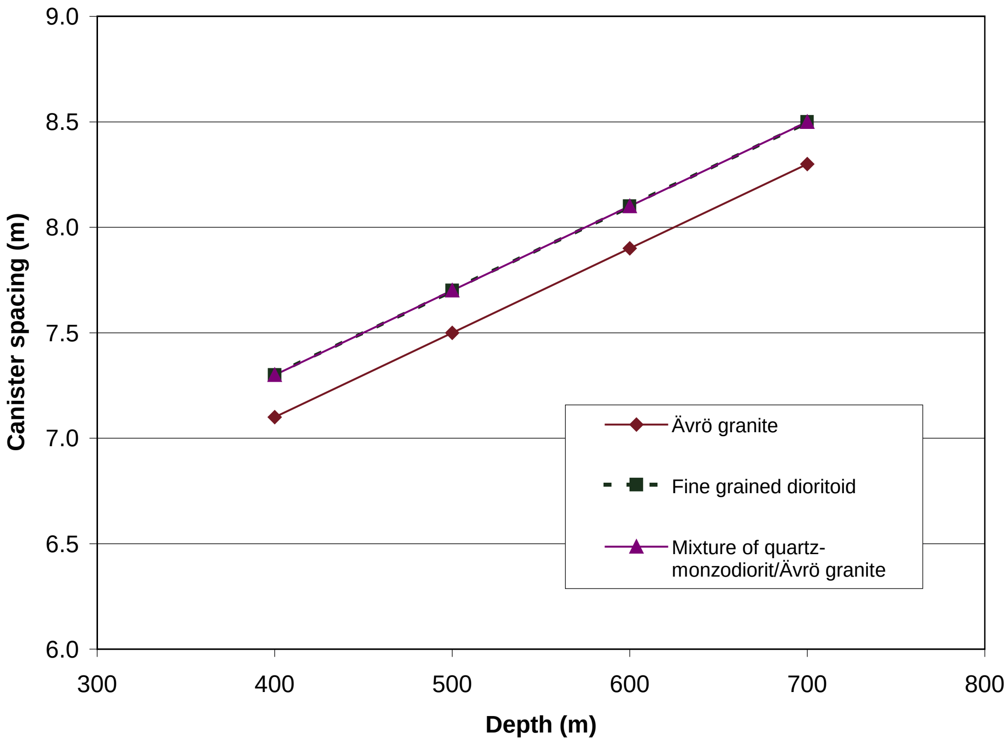


Figure 4-6. *Minimum allowable canister spacing as a function of storage depth, based on mean values of thermal conductivity for each rock domain.*

4.2.4 Sensitivity analysis

A sensitivity analysis, examining variation in the mean value of thermal conductivity for the rock and its effect on the minimum allowable canister spacing, was carried out according to the criteria given in the UDP, with variations applied to input data taken from the site description S1.2 /SKB 2005a/. A 7.5% variation from the mean was applied for rock domain A and ± 5% for domains B to D. A greater variation was considered for domain A, since measured values show a greater spread in values. The bedrock division into rock domains is presented in Section 3.2. The analysis methodology is described in the Section 4.2.2.

The results of a sensitivity analysis, examining variation in the mean value of rock thermal conductivity and its effect on the minimum allowable canister spacing are presented in Table 4-5.

The variation of ± 7.5% in thermal conductivity in rock domain A results in a variation in minimum allowable canister spacing of ± 0.6 m at –400 m depth and ± 0.9 m at –700 m depth. The variation of ± 5% in thermal conductivity in the other domains results in a variation in minimum allowable canister spacing of ± 0.5 m at –400 m depth and ± 0.7 m at –700 m depth. The analysis indicates that variations between different rock domains and storage depths have only a moderate effect on minimum allowable canister spacing. For a repository of 6,000 canisters this variation in thermal conductivity results in a possible variation of 3,000–6,000 m in the required total deposition tunnel length valid for the depth interval studied (–400 to –700 m), which corresponds to 5–10% of the total deposition tunnel excavation volume.

Table 4-5. Sensitivity analysis with variation in mean thermal conductivity and the resulting minimum allowable canister spacing.

Depth (m)	Initial temp (°C)	Max canister surface temp (°C)	Canister spacing (m)											
Conductivity variation (W/mK)			Domain A			Domain B			Domain C			Domain D		
			–7.5%	Mean	+7.5%	–5%	Mean	+5%	–5%	Mean	+5%	–5%	Mean	+5%
			2.59	***2.80***	***3.01***	***2.60***	***2.74***	***2.88***	***2.60***	***2.74***	***2.88***	***2.49***	***2.62***	***2.75***
400	13.2	81.8	7.7	7.1	6.6	7.7	7.3	6.9	7.7	7.3	6.9	8.2	7.7	7.3
500	14.8	80.2	8.2	7.5	7.0	8.2	7.7	7.3	8.2	7.7	7.3	8.8	8.2	7.7
600	16.3	78.7	8.7	7.9	7.3	8.7	8.1	7.7	8.7	8.1	7.7	9.3	8.7	8.1
700	17.6	77.4	9.3	8.3	7.6	9.3	8.5	8.0	9.3	8.5	8.0	9.9	9.3	8.5

4.2.5 Conclusions

The results show that for the rock domains relevant for design, namely domains A to C, the minimum allowable canister spacing varies from 7.1 to 7.3 m at –400 m depth and from 8.3 to 8.5 m at –700 m depth.

For a repository with a 6,000 canister storage capacity, a variation of ± 7.5% in rock thermal conductivity results in a variation of around 3,000–6,000 m in the required total length of deposition tunnels for the studied depth interval. This length corresponds to ca 5–10% of the total excavation volume for the deposition tunnels.

The assessment of canister spacing results in the overall conclusion that –400 m depth is the preferred repository depth.

4.3 Orientation of deposition tunnels

4.3.1 Input data and assumptions

This chapter summarizes the studies carried out to optimize the orientation of the deposition tunnels with a view towards minimizing the quantity of seepage, the risk of spalling and the volume of potentially unstable wedges in the deposition tunnels and deposition holes.

The design task has been carried out in accordance with Section 5.4.3 of UDP /SKB 2004a/ with the following modifications:

- The study of tunnel orientation, with reference to the risk for spalling in the deposition tunnels, has been verified by calculation of the tangential stresses on the deposition tunnel periphery analytically, rather than by numerical modelling. This is further discussed in /Martin 2005/.
- The study of tunnel orientation, with reference to the risk for potential wedge failure in the deposition tunnels, is verified by kinematic block analysis without generating a stochastic fracture network.

4.3.2 Execution

Quantity of seepage into the deposition tunnels and deposition holes for different tunnel orientations

Analytical model

The analysis of tunnel orientation with regard to the quantity of water inflow, was based on a simulation of a deposition tunnel as a circular horizontal drain without deposition holes, see Figure 4-7.

Hydraulic conductivity was assessed with reference to orientation dependence, with input data sourced from the site descriptive model S1.2, as presented in Table 4-6 and by application of the following relationship in accordance with UDP /Harr 1999/:

$$K_\alpha = \frac{K_{\max} K_{\min}}{\left[K_{\max} \sin^2 \alpha + K_{\min} \cos^2 \alpha\right]} \qquad \text{Equation 4-1}$$

and

$$K_b = \left[K_z K_\alpha\right]^{0.5}$$

where,

K_α = representative hydraulic conductivity in an arbitrary direction α

K_b = representative hydraulic conductivity of the rock mass in the x-y plane, for in-plane flow

K_z = representative hydraulic conductivity of the rock mass in the vertical direction.

The hydraulic conductivity in different directions is presented in the site descriptive model S1.2 and is a result of bloc modelling in DarcyTools and Connect Flow. It should however be noted in Table 4-6 that the conductivity in different directions is very similar.

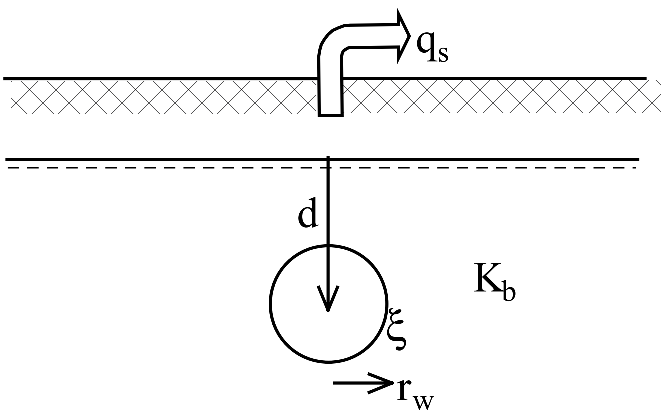


Figure 4-7. *Analytical model for the deposition tunnels UDP, /SKB 2004a/. See explanation of symbols below Equation 4-1.*

Table 4-6. Hydraulic conductivities in the x, y, z-planes for a 100×100×100 m block scale with data taken from borehole KSH01A /SKB 2005a/.

Direction	Mean (m/s)	Std deviation $log_{10}(K)$ (m/s)
x (N)	6.6E–09	0.37
y (E)	6.4E–09	0.37
z	6.4E–09	0.37

The tunnel orientation has been varied in relation to the major principle stress direction (N135°) with the relative angles 0° (parallel with the main principle stress) 15°, 30°, 45°, 60°, 75°, 90°, 105°, 120°, 135° and 150° to study any effect of anisotrophy.

In order to take account of the variation and uncertainty, the representative hydraulic conductivity K_α has been assessed using a MonteCarlo simulation method. Parallel with this simulation K_b has been calculated, followed by steady state seepage q_s, in accordance with Equation 4-2 /Alberts and Gustafson 1983/. The representative hydraulic conductivity is set at right angles to the tunnel direction.

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

where,

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

d = deposition tunnel's centre depth below the groundwater table (m)

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

K_b = representative hydraulic conductivity of the rock mass for analysed tunnel orientations (m/s)

ξ = deposition tunnel's natural skin factor (dimensionless).

DFN-model

Studies to optimize the tunnel orientation of the deposition tunnels with a view towards minimizing the quantity of water inflows to the deposition tunnels and deposition holes were also carried out with the aid of a stochastically generated fracture network via DFN analysis (Discrete Fracture Network). The analysis was carried out using Connectflow© computing tools /Hartley and Holton 2003/ and Napsac© /Hartley et al. 2003/.

The analysis involved 20 simulations of differing fracture networks based on the following framework:

- model size 300×300×500 m (H×B×L)
- rock domain Ävrö granite
- storage depth 500 m
- tunnel dimension length 300 m, width 5.5 m, height 5.5 m
- hole dimensions diameter 1.75 m, hole depth 8 m
- hole depth 7.5 m

The model is divided into an outer and inner box. The inner box has the dimensions 25.5×25.5×320 m (HxWxL) whilst the outer box has the dimensions 300×300×500 m.

DFN-parameters used in the model are based on evaluations carried out by the DarcyTools© group for the Simpevarp site descriptive model S1.2. /SKB 2005a/. Due to a lack of fracture properties for rock domain A, which is dominated by Ävrö granite, the DFN analysis is based on fracture data from rock domain D that is dominated by quartz monzodiorite. However, the fracture data is considered representative for both domains.

The model does not take into account any depth dependency in the fracture properties. This is due to the fact that such information is not available in the Simpevarp site description S1.2.

All fracture sets have been included in the model and generated with lengths between 0.5 m and 1,000 m. For technical reasons each set is subdivided into two length interval groups 0.5–10 m and 10–1,000 m. The fracture group with the shorter length interval is only generated in the inner model box while the longer group is generated throughout the entire model volume.

The chosen approach is based on the precept that the fracture transmissivity is a function of the fracture length /Follin et al. 2005, Harley et al. 2005/. Short fractures are only of significance in close proximity to the tunnel and only where they are in hydraulic contact with longer fractures.

The distance between the deposition tunnel ends and the periphery of the nearest deposition hole is set at 8 m. This is a deviation from UDP /SKB 2004a/, which states that a distance of 20 m should be used between the nearest deposition hole periphery and the junction with the transport tunnel. The deposition holes are spaced at 7.5 m centres, which for a 300 m long deposition tunnel, gives 38 deposition hole positions. These values correspond to those used for the proposed design layout presented in Section 5 of this report.

The orientation of the deposition tunnels with respect to minimizing the water inflow to the deposition tunnels and deposition holes has been analysed for six different tunnel orientations; 0°, 30°, 60°, 90°, 120° and 150°, see Figure 4-8. The deposition tunnel is rotated clockwise in 30° steps with NW-SE, the orientation of the major principal stress, taken as the initial orientation.

The model's inner boundary, consisting of the deposition tunnel and deposition hole peripheries, is at atmospheric pressure and no inflow at the tunnel ends is specified as the boundary condition. The model's outer boundary is set at hydrostatic water pressure, corresponding to the specified storage depth, as the boundary condition. No skin effects have been taken into consideration in the model.

"Cluster data", which indicates if the fractures are distributed evenly throughout the rock mass or not, have not been used since this information is not available in the Simpevarp site description S1.2. Storage coefficients have not been calculated in the model. Inflow in the model is assumed to increase linearly with storage depth and reduce linearly with the difference between the normal and the depressed groundwater surface.

Risk for spalling in the deposition tunnels

Analysis of deposition tunnel orientation with a view to minimizing the risk for spalling has been carried out by /Martin 2005/. The analysis in general is in accordance with UDP guidelines. However, instead of calculating the tangential stresses on the deposition tunnels periphery in a numerical 3D model, the stress level has been calculated analytically with the

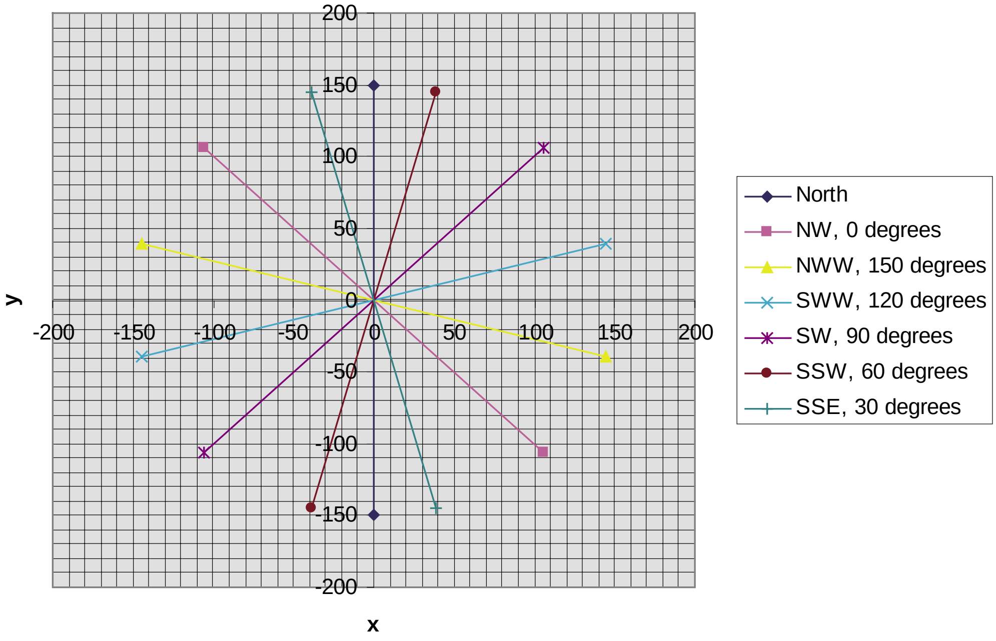


Figure 4-8. *Tunnel orientations studied in the DFN analysis. The tunnel is rotated clockwise from NW-SE, the orientation of the major horizontal principal stress, in 30° intervals.*

aid of the Kirsch equation for a single circular opening /Hoek and Brown 1982/. The assessment presents the stresses in a vertical section at the midpoint of the tunnel. The tunnel is modelled without deposition holes. The analysis method generally results in a somewhat lesser stress concentration around the deposition tunnel compared with the method proposed in UDP.

The risk for spalling has been evaluated by calculating a factor of safety (FOS), dependent on the ratio between the rock mass spalling strength, representing the compressive stress at fracture initiation and the major tangential stress on the deposition tunnel's periphery according to Equation 4-3.

$$FOS = \frac{\sigma_{sm}}{\sigma_{\theta\theta}} \qquad \text{Equation 4-3}$$

where,

σ_{sm} = the rock mass spalling strength (MPa)

$\sigma_{\theta\theta}$ = the maximum tangential stress on the deposition tunnel periphery (MPa).

If the tangential stress exceeds the compressive stress for fracture initiation, it is assumed that spalling will occur in the deposition tunnels. Based on the Äspö Pillar Stability Experiment and AECL's Mine-by Experiment, it is assumed that the compressive stress for fracture initiation is related to the uniaxial compressive strength for intact rock according to the following relationship /Andersson et al. 2004/, /Read et al. 1997/:

$$\sigma_{sm} = 0.57 \pm 0.02\sigma_{ci} \qquad \text{Equation 4-4}$$

where,

σ_{sm} = the rock mass spalling strength (MPa)

σ_{ci} = uniaxial compressive strength for intact rock (MPa).

The analysis has taken into account the variation in the initial in situ stress magnitude and uniaxial compressive strength of the intact rock, by carrying out MonteCarlo simulations, assuming a triangular division of parameters.

For the case where the tangential stress exceeds the compressive stress for fracture initiation (FOS < 1), the depth of spalling has been evaluated based on results from Äspö and AECL's URL. The spalling depth (S_d), measured perpendicularly to the tunnel periphery, has been assessed based on the following empirical relationship /Martin et al. 2001/:

$$S_d = a\left(0.5\frac{\sigma_{\theta\theta}}{\sigma_{sm}} - 0.52\right) \qquad \text{Equation 4-5}$$

where,

a = deposition tunnel radius (m)

σ_{sm} = rock mass spalling strength (MPa)

$\sigma_{\theta\theta}$ = maximum tangential stress on the tunnel periphery (MPa).

In accordance with the Simpevarp site description S1.2, it has been assumed that there are two in situ stress domains within the Simpevarp sub area /SKB 2005a/. Stress domain II is judged to have significantly lower stress levels than domain I while the stress orientations are essentially the same. Stress domain II is demarcated by deformation zones ZSMNE012A in the west and ZSMNE024A in the east.

The initial in situ stress gradient used in the analysis for calculation of $\sigma_{\theta\theta}$, along with the calculated in situ values for storage depths –400 to –700 m are presented in Table 4-7. The gradient used in /Martin 2005/ differs somewhat from that presented by the site descriptive model /SKB 2005a/.

Uncertainty associated with the in situ stress field's horizontal component has been allowed for in the analysis by including a ± 25% margin to the values presented in Table 4-7.

The uniaxial compressive strength of the intact rock is based on results from laboratory tests on samples taken from KSH01A and KSH02A. In the site description model S1.2 mean values are presented for rock domains dominated by quartz monzodiorite and Ävrö granite (165 MPa) and finegrained diorite (210 MPa). Here, a mean value of σ_{ci} = 183 MPa has been taken to represent the sub area as a whole. The application of the relationship $\sigma_{sm} = 0.57\pm0.02\times\sigma_{ci}$ results in a spalling strength in the range of σ_{sm} = 100–108 MPa /Martin 2005/.

Table 4-7. Initial in situ stress gradients used in the analysis for calculation of $\sigma_{\theta\theta}$ along with mean in situ values calculated for storage depths –400 to –700 m /Martin 2005/.

	Stress domain I			**Stress domain II**		
Gradient	**σ_{Hmax} (MPa/m)**	**σ_{hmin} (MPa/m)**	**σ_{vert} (MPa/m)**	**σ_{Hmax} (MPa/m)**	**σ_{hmin} (MPa/m)**	**σ_{vert} (MPa/m)**
	2+0.06z	0.019z	0.0265z	0.0314z	0.011z	0.0265z
Depth, z (m)	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**
400	26.0	7.6	10.6	12.6	4.4	10.6
500	32.0	9.5	13.3	15.7	5.5	13.3
600	38.0	11.4	15.9	18.8	6.6	15.9
700	44.0	13.3	18.6	22.0	7.7	18.6

Volume of unstable wedges in the deposition tunnels and deposition holes

Analysis of deposition tunnel orientation with a view to minimizing the volume of potentially unstable wedges in the deposition tunnels and holes has been carried out by kinematic block analysis and does not include the generation of a stochastic fracture network as prescribed by UDP /SKB 2004a/.

The analysis has been performed with the computing tool *Unwedge*©. The tool assumes a fracture plane can occur anywhere within the rock volume and presents the maximum possible wedge geometry for any specified tunnel orientation and fracture sets. In order for the results to be realistic and not overly conservative, the wedge volumes have been limited by reference to the fracture lengths as observed in the field.

Input data for the analysis consists of the intact rock density and initial in situ stress field, along with the length and shear strength of the different fracture sets. The input data was sourced from the Simpevarp site description S1.2.

The fracture orientations and fracture length distribution are based on the DFN model (Alt 1) in Simpevarp site description S1.2 /SKB 2005a/. The tunnel dimensions and layout used were as presented in Layout E, see Section 4.1 of this report.

The initial stress field has been used in the analysis but not the water pressure in the fractures. The inclusion of the initial stress field can lead to an increase in the safety factor for wedge failure due to the stress generated confining effect.

The analysis is based on the assumption presented in the site description model S1.2 /SKB 2005a/, that there are two in situ stress domains with in the Simpevarp sub area, as outlined in the pervious section concerning the risk for spalling in the deposition tunnels.

The initial in situ stress gradient used in the analysis, along with the calculated values for storage depths –400 to –700 m are presented in Table 4-8. Further analysis focused on storage levels –400 m and –700 m.

All the fracture planes in the model have been taken to be continuous and planar with the same shear strength. Mohr-Coloumb failure criteria have been applied with peak shear strength values for the fracture planes. The fracture plane friction angle was set at $\varphi = 32°$ and cohesion at $c = 0.5$ MPa /SKB 2005a/.

According to the DFN-model (Alt 1) for Simpevarp, the fractures can be divided into six sub vertical and one sub horizontal fracture sets. Two of these fracture sets, EW-WNW and BGNW /SKB 2005a/, have been combined to form a single set. The wedge analysis has been carried out for a total of six fracture sets as presented in Table 4-9.

Table 4-8. Initial in situ stress gradients used in the analysis with mean values calculated for storage depths –400 to –700 m /SKB 2005a/.

	Stress domain I			**Stress domain II**		
Gradient	**σ_{Hmax} (MPa/m)**	**σ_{hmin} (MPa/m)**	**σ_{vert} (MPa/m)**	**σ_{Hmax} (MPa/m)**	**σ_{hmin} (MPa/m)**	**σ_{vert} (MPa/m)**
	3+0.058z	0.019z	0.028z	0.032z	0.011z	0.028z
Depth, z (m)	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**	**(MPa)**
400	26.2	7.6	11.2	12.8	4.4	11.2
500	32.0	9.5	14.0	16.0	5.5	14.0
600	37.8	11.4	16.8	19.2	6.6	16.8
700	43.6	13.3	19.6	22.4	7.7	19.6

Table 4-9. Orientation of fracture sets used for the wedge analysis.

ID no	Orientation	Dip	Dip direction
1	NNE-NE	88	298
2	EW-WNW, BGNW	86	199
3	NW-NNW	85	253
4	BGNE	85	146
5	BGNS	86	279
6	SubHZ	4	213

BG – Background.

The analysis considers all possible tetrahedral wedges that can be formed by any three fracture-plane combinations, from the six fracture sets available. This results in 20 possible combinations.

The results are sorted on maximum wedge volume and the necessary support pressure required to prevent wedge fallout and achieve a safety factor of 1.5. Wedges formed at the tunnel ends are not considered.

Wedge side length has been limited on the basis of the likelihood that a certain length will occur according to the DFN-model (Alt1) for Simpevarp, /SKB 2005a/. For wedge faces formed by sub vertical fractures the side length has been limited to a maximum of 3 m. The side length corresponds to an approximate mean value of the fracture diameter at the 1%-percentile for the sub vertical fracture sets. The maximum side length for a wedge face is formed by the sub horizontal fracture set and has been limited to the deposition tunnel width of 5.5 m.

4.3.3 Results

Quantity of water seepage to the deposition tunnels and deposition holes

Seepage according to analytical model

Figure 4-9 shows how the quantity of water inflow to the deposition tunnel varies depending on the tunnel orientation. The tunnel orientation in the figure is shown as the angle from the major principle stress direction, varying from 0° to 150°.

The results indicate that there are only marginal differences in water inflows between the different tunnel orientations studied. The mean inflow into the 300 m long deposition tunnel at 500 m depth is approximately 60 l/min and is essentially independent of tunnel orientation. At higher inflow levels a variation with tunnel orientation is indicated. The greatest difference being 7 l/min at the 99% fractal.

Seepage according to DFN-model

The combined water inflow to the deposition holes and deposition tunnel for the six different tunnel orientations is presented in Table 4-10. Mean values of water inflows to the deposition holes and deposition tunnel are presented both separately and combined in Figure 4-10.

The simulation results presented in Table 4-10 suggest that there is a small difference in combined water inflow depending on tunnel orientation, if the median values are compared. Comparison of the mean values suggests that there is a larger difference dependent on tunnel orientation. This could be explained by the fairly large spread in the results obtained, which is typical for stochastic simulations, and that the mean value is more sensitive than the median for extreme values in individual realisations. In this study only 20 realisations were performed, and it is likely that if more realisations were performed the variation based on tunnel orientation would be reduced.

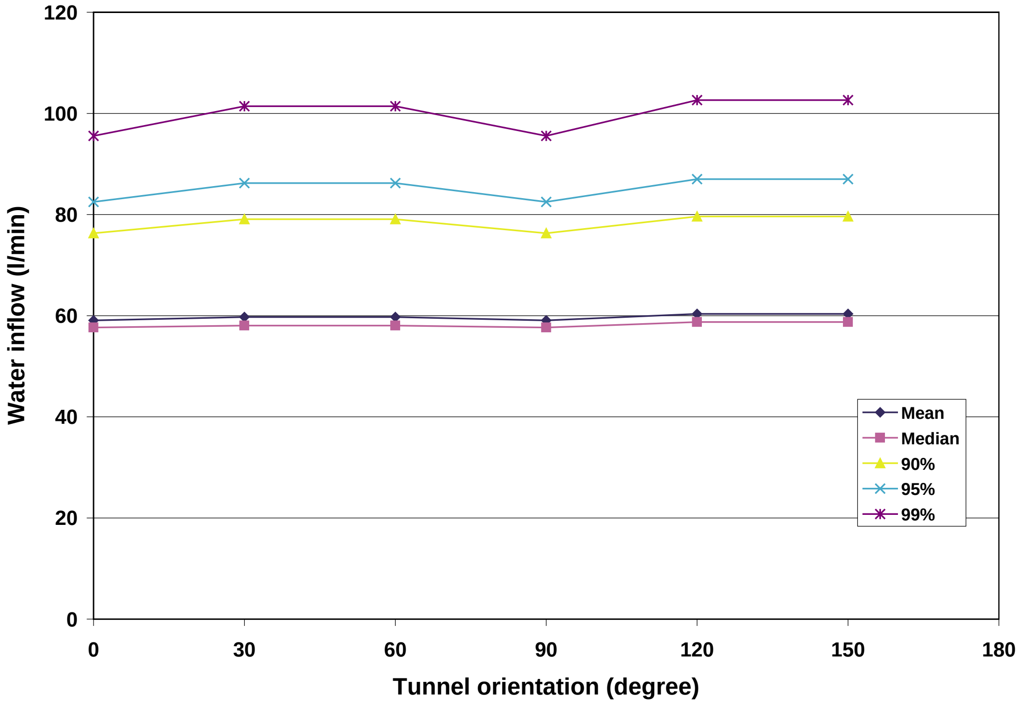


Figure 4-9. *Analytically calculated water inflow to the deposition tunnel for storage depth –500 m for six different tunnel orientations related to the major principal stress.*

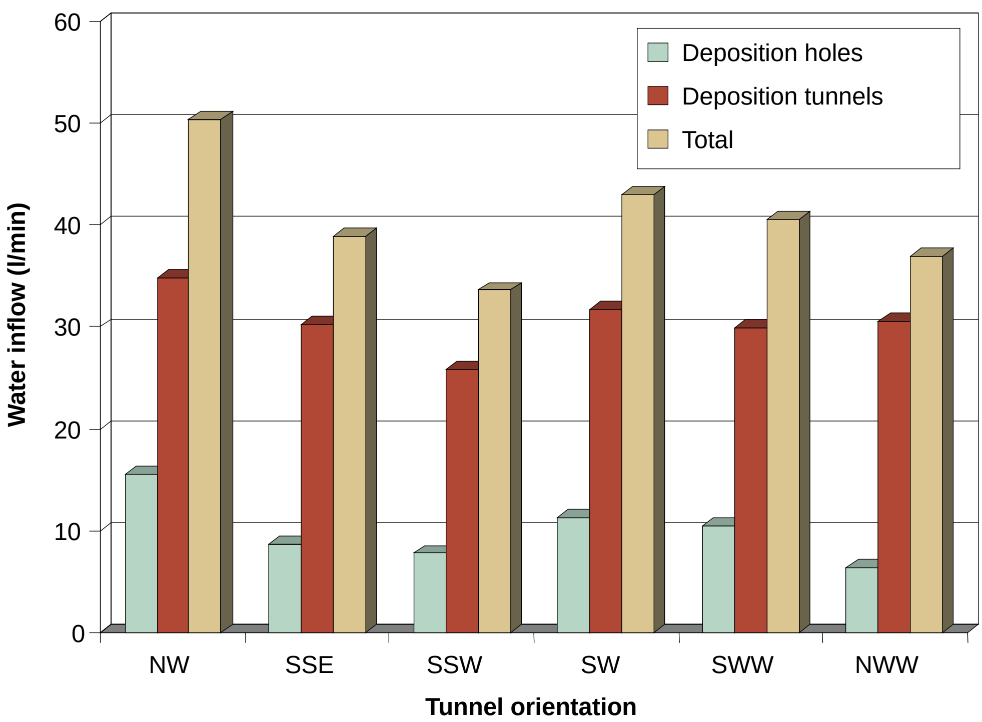


Figure 4-10. *Mean water inflow to deposition holes and deposition tunnel as a function of tunnel orientation.*

Table 4-10. Combined inflow to the deposition holes and deposition tunnel for different tunnel orientations. The analysis is based on 20 simulated stochastic fracture networks for each of the six tunnel orientations. The deposition tunnel is 300 m long with 38 deposition holes at a storage depth of –500 m.

	Tunnel orientation NW 0° (l/min)	SSE 30° (l/min)	SSW 60° (l/min)	SW 90° (l/min)	SWW 120° (l/min)	NWW 150° (l/min)
Mean	50	39	34	43	41	37
Median	31	30	24	32	27	26
Standard dev	60	38	24	31	34	32
90%-percentil	79	79	72	84	84	79
95%-percentil	92	124	74	98	101	98
99%-percentil	241	143	79	112	114	103

The variation in the calculated mean water inflows between the different tunnel orientations studied is no greater than approximately 15 l/min. The analysis indicates that the orientation of the deposition tunnel has only a limited effect on the quantity of water inflow to the tunnel for the rock domain studied.

The mean value of water inflow to the deposition holes alone reaches a minimum for tunnel orientation NWW (N105), while the combined inflow to both the deposition holes and deposition tunnel reaches a minimum for tunnel orientation SSW (N015). Based on the results from the DFN analysis it is recommended that a tunnel orientation of N015 is selected with N105 as the primary alternative.

Risk for spalling in the deposition tunnels

The analysis results for stress domain I are presented in Figure 4-11 and for stress domain II in Figure 4-12. The safety factor (FOS) against spalling, as a function of depth, is presented for the three different tunnel orientations considered. The results for stress domain I are summarised in Table 4-11. The table includes the safety factor against spalling (FOS), the probability for spalling (PofS) and spalling depth (S_d) as a function of storage depth, for a tunnel perpendicular to the maximum horizontal stress direction.

The results for stress domain I indicate that there is only a very marginal risk for spalling at a storage depth of –400 to –500 m. However, at depth –600 to–700 m there is an increased risk for spalling for a tunnel orientation perpendicular to the maximum horizontal stress direction

Table 4-11. Safety factor (FOS), probability of spalling (PofS) and spalling depth as a function of depth, for a deposition tunnel in stress domain I oriented perpendicular to the maximum horizontal stress /Martin 2005/.

Depth	FOS	PofS (%)	S_d (m)
400	1.6	0	0.00
500	1.3	6	0.01
600	1.1	32	0.03
700	~ 1.0	~ 76	~ 0.06

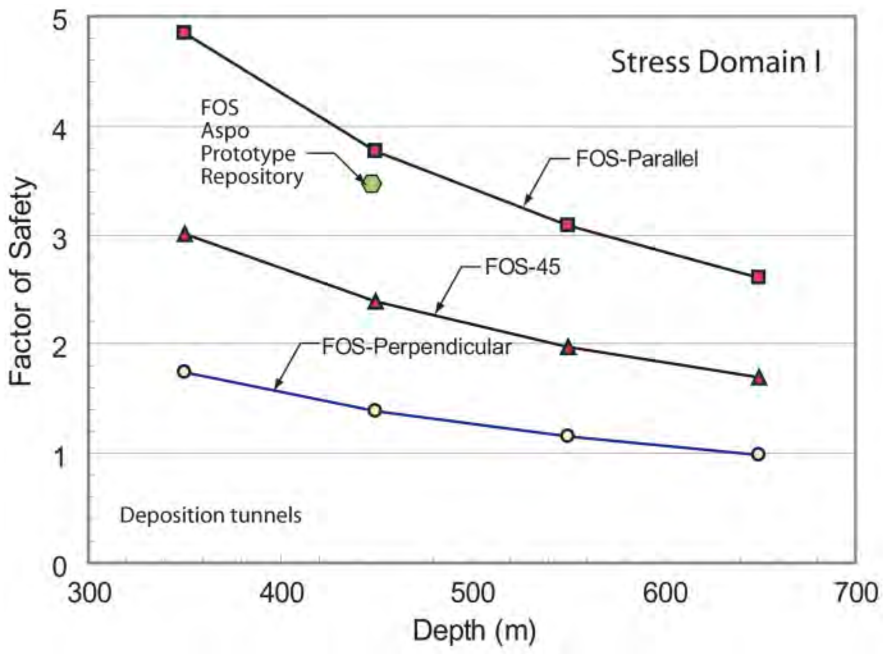


Figure 4-11. *Factor of safety against spalling for a deposition tunnel in stress domain I as a function of depth, for the three different tunnel orientations (0°, 45° and 90° to the maximum horizontal stress direction). Also shown, for reference purposes, is the the TBM tunnel at Äspö at a depth of 450 m. This tunnel is oriented approximately 30° from the maximum horizontal stress. No spalling was observed in the TBM tunnel. /Martin 2005/.*

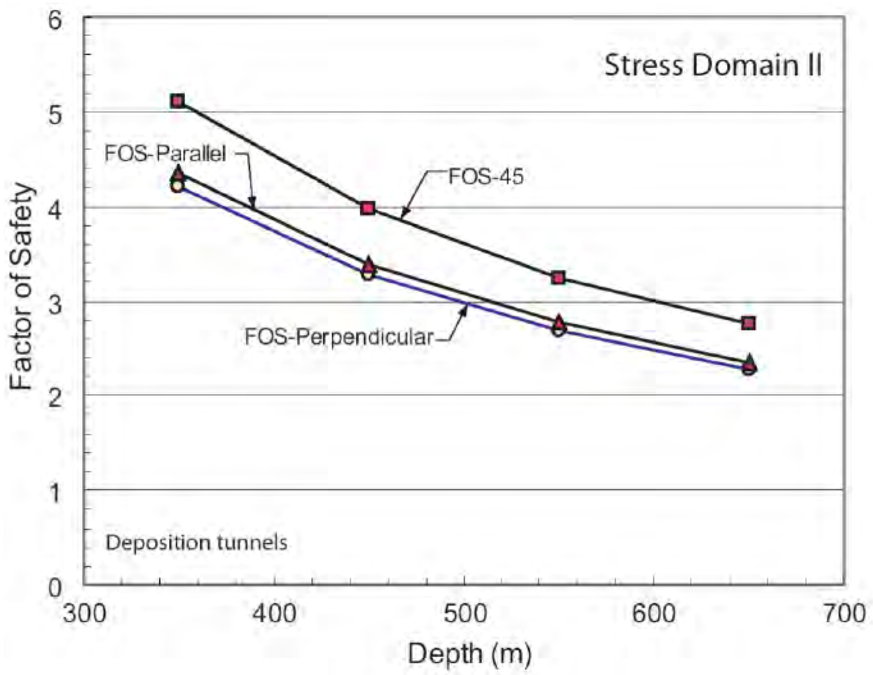


Figure 4-12. *Factor of safety against spalling for a deposition tunnel in stress domain II as a function of depth, for the three different tunnel orientations (0°,45° and 90° to the maximum horizontal stress direction) /Martin 2005/.*

The highest factor of safety against spalling in stress domain I is obtained by a tunnel oriented parallel to the maximum horizontal stress direction (N135). For this orientation there is no risk for spalling at any of the depths considered.

The results for stress domain II indicate there is no risk for spalling for the investigated depths and orientations, with the factor of safety never falling below 2.

Volume of potentially unstable wedges in the deposition tunnels

The results indicate that the factor of safety against wedge failure is only very marginally affected by variation in storage depth and the differences in stress magnitudes between the different stress domains. Consequently, only the results for level –400 m and stress domain I are presented here as a representative example.

Of the 20 fracture combinations analysed, only seven require a support pressure in order to achieve a factor of safety of 1.5 against wedge failure. For these seven fracture combinations the effect of tunnel orientation on wedge volume has been assessed. However, even for these seven fracture combinations it is possible to orientate the tunnel such that no unstable wedges are formed. Additionally, even in the worst case, the required support pressure never rises above 0.01 MPa, which implies that rock support consisting of by very light rock bolting would be sufficient /Hoek et al. 1995/.

The tunnel orientation, which results in the minimum wedge volume, varies depending on the selected fracture combination studied. The maximum wedge volume versus tunnel orientation for fracture combination 4, 5, 6, which can lead to unstable wedges, is presented in Figure 4-13. This fracture combination shows that a minimum wedge volume is achieved with a tunnel orientation of N110 (N290).

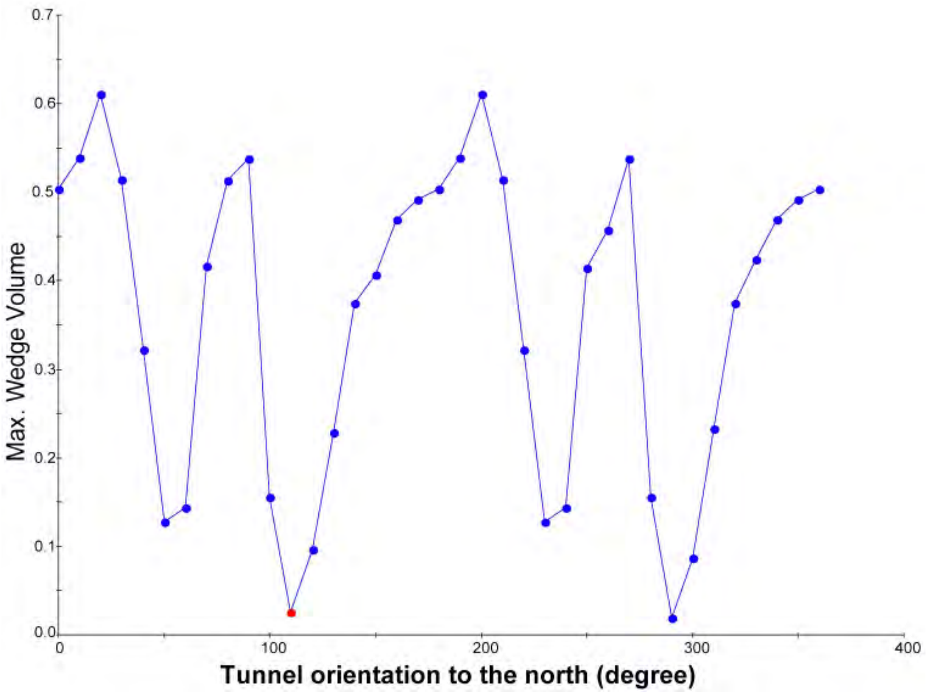


Figure 4-13. *Maximum wedge volume versus tunnel orientation for fracture combination 4, 5, 6. The wedge volume is minimized with a tunnel orientation of N110.*

The tunnel orientation for minimizing wedge volumes, involving the other fracture combinations resulting in a factor of safety of less than 1.5, has been analysed in the same manner. The results are compiled in a bar chart, Figure 4-14, presenting frequency as a function of tunnel orientation, for those orientations where minimal wedge volumes are found.

Figure 4-14 shows that a tunnel orientation of N110 is optimal in that it results in the minimum wedge volume being achieved on the highest number of occasions for the fracture combinations studied.

4.3.4 Discussion and conclusion

The following section presents a compilation of the key variables necessary for the selection of an optimal tunnel orientation.

The analytical analysis shows that water inflow to the deposition tunnel is in principle independent of tunnel orientation, while an orientation that minimizes the water inflow is possible to identify by DFN simulation. In the assessment below only the results from the DFN analysis have been considered, since it is only this model that enables evaluation of an optimal tunnel orientation.

Tunnel orientation, with reference to combined water inflow to the deposition tunnels and holes, along with the risk for spalling, is presented for 500 m depth of the repository in Figure 4-15 for stress domain I and in Figure 4-16 for stress domain II. The inflow levels are represented by normalized mean values of total water inflow to the tunnel and deposition holes in the DFN model i.e. the mean value for the maximum combined inflow to both the deposition tunnel and deposition holes (NW = 50 l/min) has been set as 1.0. The risk for spalling is presented as a factor of safety against spalling.

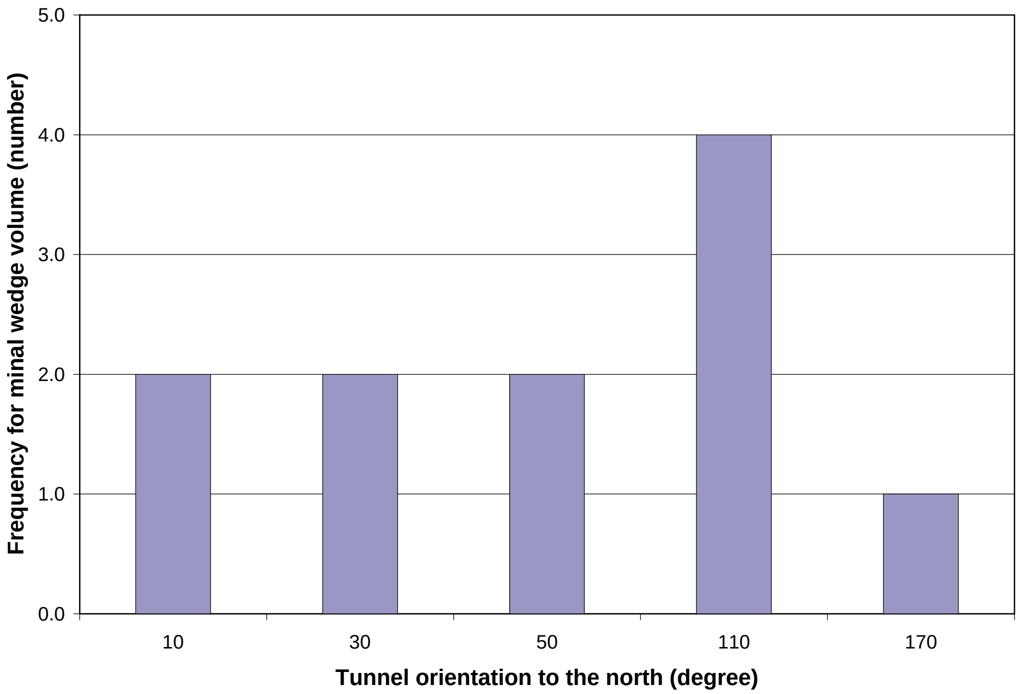


Figure 4-14. *Frequency chart for minimum wedge volume as a function of tunnel orientation, for those fracture combinations that require a support pressure to achieve a factor of safety FOS > 1.5.*

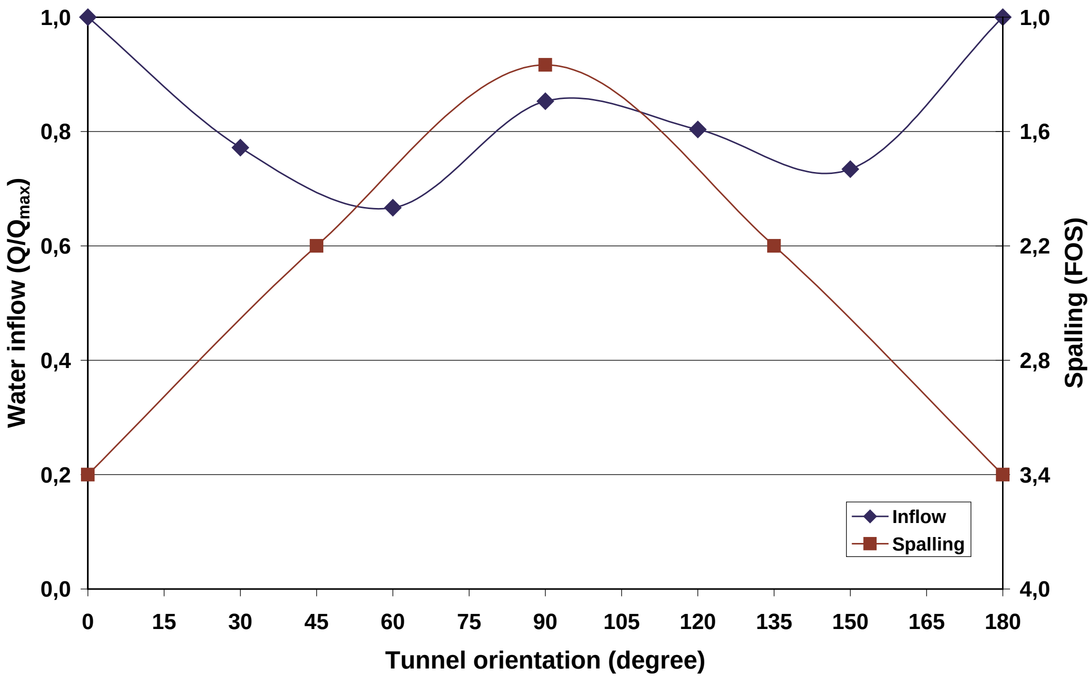


Figure 4-15. *Compilation of water inflow and spalling risk (FOS = Factor of Safety) for stress domain I at a storage depth –500 m. The tunnel orientation is shown relative to the maximum horizontal stress.*

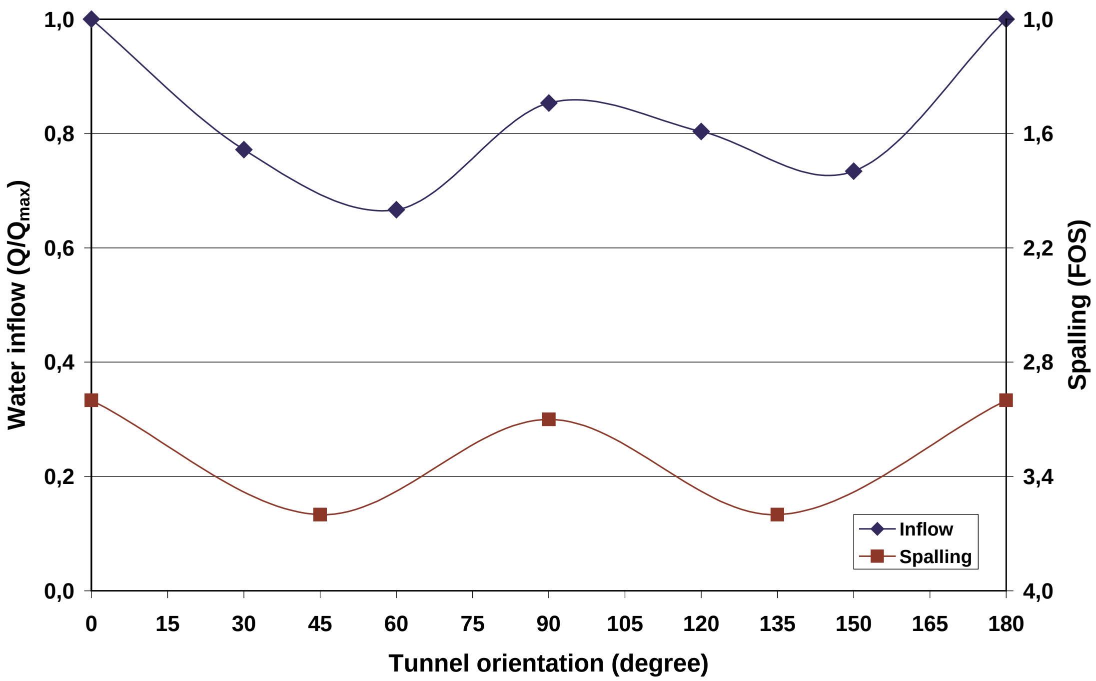


Figure 4-16. *Compilation of water inflow and spalling risk (FOS = Factor of Safety) for stress domain II at a storage depth –500 m. The tunnel orientation is shown relative to the maximum horizontal stress.*

In stress domain I water inflows reach a minimum with a tunnel orientation of approximately 60° to the maximum horizontal stress (N015). This orientation is not optimal for minimizing the risk for spalling, however, this orientation is unlikely to result in spalling at a storage depth of –400 m to –500 m. The risk for spalling is minimized by a tunnel orientation parallel to the maximum horizontal stress direction.

In stress domain I a tunnel orientation of 150° from the maximum horizontal stress direction (N105) is a possible alternative orientation from both the point of view of water inflow and spalling.

Within stress domain II it is considered that there is no risk for spalling and the selection of the most suitable tunnel orientation can be made without reference to this parameter, see Figure 4-16. This leads to the selection of the same tunnel orientations for stress domain I as for Stress domain II, with reference to the quantity of water inflows to the deposition tunnels.

With consideration to minimizing the volume of potentially unstable wedges, a tunnel orientation of N110 is most advantageous. However, the required support pressure to achieve a universal factor of safety against wedge failure of FOS > 1.5 is very limited. The analysis indicates that a support pressure of only 0.01 MPa is required to stabilize the largest individual wedge that can realistically be considered (< 1 m^3). Allowing for the implementation of only minor and straightforward rock support it is no longer necessary to consider minimizing potentially unstable wedge as a key variable in optimising the overall tunnel orientation.

The proposed orientation based on the analyses, is for a repository depth of –400 m to –500 m an optimal orientation for the deposition tunnel of N015, irrespective of stress domain, while N105 constitutes a reasonable alternative.

It should be noted that transmissivity measurements in the spiral of Äspö HRL are in sharp contrast to the results presented here. These measurements show a clear anisotropy with a factor of hundred between the direction of minimal and maximal transmissivity. The highest transmissivity was recorded in fractures in a NW-SE direction /Rhén et al. 1997/.

4.4 Loss of deposition holes

4.4.1 Input data and assumptions

The section covers the loss of deposition holes during the construction phase due to the existence of the following conditions at proposed deposition hole locations:

- unacceptably long fractures,
- water inflows,
- wedge failures,
- spalling.

The work was carried out according to SKB guidelines as presented in UDP /SKB 2004a/, Section 5.4.4, subject to the following modifications:

- Loss of deposition holes due to failure to fulfil a minimum distance criterion between the deposition hole and a stochastic fracture with R > 100 m is only studied with taking 2 m as the minimum distance. When this criterion has not been fulfilled such holes have not been moved as described in UDP.

- Loss of deposition holes due to potential wedge failures is based on kinematic block analysis that does not include the generation of a stochastic fracture network.
- Loss of deposition holes due to spalling was based on an analysis of tangential stresses on the hole periphery without 3D numerical modelling.

The analyses involving the generation of a stochastic fracture network (DFN) have been carried out in cooperation with Kemakta Konsult AB. The assessment of spalling potential has been carried out by Prof. C. Derek Martin, University of Alberta /Martin 2005/.

4.4.2 Execution

Minimum allowable distance between a deposition hole boundary and a stochastic fracture with a radius R > 100 m

Minimum allowable distance has been studied by an analytical method /Hedin 2005/ and a Discrete Fracture Network analysis (DFN).

Method with analytical analysis

The analytical method is based on the simple notion that if the distribution of the fracture size and orientation in a host rock are known, and if a canister is emplaced randomly in that host rock, than it is possible to calculate the probability that the canister is intersected by a fracture that exceeds a certain size.

Input data to the analysis are the statistical descriptions of fracture sizes and orientations that emerge from the site investigation, i.e. the same input data was used in the two alternative analyses. The analysis including the full fracture population, i.e. both “open” and “sealed” fractures. All fractures are assumed to be infinitesimally thin, circular discs. The distance criteria used for the analytical approach is: with a 100 m respect distance, deposition holes must not be placed in the central parts of fractures with radius R > 50 m.

Method with numerical (DFN) analysis

The DFN-analysis was carried out with the same DFN-model that was used for studies to optimize the tunnel orientation, using *Connectflow*© and *Napsac*© software. The analysis is similarly based on the generation of 20 different fracture networks and the same general approach as outlined in Section 4.3.2.

The loss of deposition holes due to the allowable minimum distance between a deposition hole and a stochastic fracture with a radius of R > 100 m is analysed for a 2 m minimum distance. In the model this was represented by defining a larger deposition hole, with a radius of 2.875 m and a depth of 10 m, instead of dimensions in accordance with layout E of 0.875 m and 8 m. The lengths of the fractures crossing the volume were then analysed.

The other loss of deposition hole criterion included in UDP, where the minimum allowable distance between a deposition hole periphery and a stochastic fracture shall be 0.01R for fractures with R > 200 m, has not been analysed in the current study. Additionally, holes that fail to fulfil such a criterion have not been moved as required by UDP specifications.

Fractures are generated in the size distribution with lengths from 0.5 to 1,000 m, i.e. radii from 0.3 to 564 m. Due to the power-law distribution within the used DFN model, and using the assigned total fracture area, many small fractures and very few large fractures are created. Hence, no large impact on the results is expected, by not using the second criterion.

Water inflows to deposition holes

The loss of deposition holes due to water inflows has been analysed with the aid of Discrete Fracture Network analysis (DFN). The analysis was carried out with the same DFN-model that was used for studies to optimize the tunnel orientation, using *Connectflow*© and *Napsac*© software. The analysis is similarly based on the generation of 20 different fracture networks and the same general approach as outlined in Section 4.3.2

The calculated inflow per deposition hole has been assessed for all deposition holes in all versions of the generated fracture network. Those holes that have inflows of $q > 10$ l/min are assumed to be lost according to UDP /SKB 2004a/. A sensitivity analysis for inflow levels has also been carried out with deposition hole losses being analysed for $q > 1$ l/min.

Potential wedge failure in deposition holes

Loss of deposition holes due to potential wedge failures has been analysed with the aid of kinematic block analysis. However, the generation of a stochastic fracture network was not included as specified by UDP /SKB 2004a/. The analysis was carried out with *Unwedge*©, the same software used for studies to optimize the tunnel orientation. The analysis is based on the same general approach as outlined in Section 4.3.2.

The results are sorted on minimum wedge volume and factor of safety against wedge failure. Wedges formed at the bottom of the deposition holes are not considered. The maximum side length, for a wedge face formed by the sub horizontal fracture set, has been limited to the deposition hole diameter 1.75 m.

According to UDP, the total volume of potentially unstable wedges in a deposition hole should be compared with the criterion for loss of deposition holes $V_{tot} \geq 0.15$ m^3/deposition hole. Additionally, a sensitivity analysis is required by UDP, involving other volume criteria for loss of deposition holes, namely $V_{tot} > 0.1$ m^3 and $V_{tot} > 0.2$ m^3. However, this step proved to be unnecessary since the fracture plane shear strength showed itself to be sufficient to stabilize all the potentially unstable wedges in the model.

Risk of spalling in deposition holes

Analysis of deposition holes with a view to the risk for spalling has been carried out in general accordance with UDP guidelines. However, instead of calculating the tangential stresses on the deposition holes' periphery in a numerical 3D model, the stress level has been calculated analytically with the aid of the Kirsch equation for a single circular opening /Hoek and Brown 1982/. The assessment presents the stresses in a horizontal section at the midpoint of the deposition hole. The effect of the proximity to the deposition tunnel is not taken into account. The analysis is based on the same general approach as outlined in Section 4.3.2, details are presented in /Martin 2005/.

The minimum allowable centre to centre spacing for the deposition holes, based on the rock mass thermal properties, is calculated to be 7 m in Ävrö granite /SKB 2005a/.This spacing leads to the deposition holes having a minor effect on each others stress field but this effect is considered sufficiently small that it may be discounted. This assumption is based on the fact that the radius of influence around a circular excavation is not normally greater than 5 radii /Brady and Brown 1985/.

The length of the spalling failure is taken to be approximately 75% of deposition hole length and its lateral extension along the hole periphery to be 0.8 m. Both these assumptions are based on the results of from the Äspö Pillar Stability Experiment /Andersson et al. 2004/. The spalling volume could then be estimated by combining these values with the assessed spalling depth.

4.4.3 Results

Minimum allowable distance between a deposition hole periphery and a stochastic fracture with a radius R > 100 m

Method with analytical analysis

Applying the analytical method to the same input data as the DFN simulation presented below results in approximately 13% rejected positions /SKB 2005b/. The result is independent of the tunnel orientation.

Method with numerical (DFN) analysis

The results from the DFN-analysis indicates that a 300 m long deposition tunnel will intersect 3 to 6 fractures with radii R > 100 m that can result in the loss of holes. Assuming that the deposition hole positions are not modified, then these structures result in a loss of 7–15% of the deposition holes depending on the tunnel orientation. For the tunnel orientations considered for the proposed repository layout, SSW (N015) and NWW (N105), deposition hole losses are approximately 7% of the theoretical total.

The estimated canister losses are a few percent lower than the results presented above, yet still in the same order of magnitude. The difference in losses is mainly due to the smaller model volume of the DFN simulation /SKB 2005b/. The dependence on tunnel orientation in the loss of holes in the DFN-model is probably due to a limited number of fracture network simulations.

Quantity of water inflow to the deposition holes.

The results from the DFN analysis are summarised in Table 4-12 to Table 4-14. Mean, min and max values, along with 75% and 95%-percentiles of the number of deposition holes lost, are presented for the inflow criteria 10 l/min and 1 l/min.

Application of the inflow criterion of 10 l/min per deposition hole, for a 300 m long tunnel, at –500 m depth, results in generally only a single deposition hole being lost, out of the possible total 38 holes. The number of lost holes rises to 1–2 holes for the 95%-percentile, see Table 4-12. This suggests that if adjustments are made to the deposition hole positions then it may be possible to avoid any deposition hole losses due to inflow of water.

Table 4-12. Mean, min, max, 75% and 95%-percentiles of the number of lost deposition holes for an inflow criterion of 10 l/min. The analysis assumes a 300 m long deposition tunnel, at a depth of –500 m, with a total of 38 deposition holes and considers 20 different fracture network simulations for six different tunnel orientations.

	Tunnel orientation NW 0° (no)	SSE 30° (no)	SSW 60° (no)	SW 90° (no)	SWW 120° (no)	NWW 150° (no)
Mean	0.5	0.1	0.3	0.2	0.2	0.3
Min	0.0	0.0	0.0	0.0	0.0	0.0
Max	4.0	1.0	2.0	2.0	1.0	2.0
75%-percentile	0.3	0.0	0.0	0.0	0.3	0.0
95%-percentile	2.1	1.0	1.1	1.1	1.0	1.1

Table 4-13. Mean, min, max, 75% and 95%-percentiles of the number of lost deposition holes for an inflow criterion of 1 l/min. The analysis assumes a 300 m long deposition tunnel, at a depth of –500 m, with a total of 38 deposition holes and considers 20 different fracture network simulations for six different tunnel orientations.

	Tunnel orientation NW 0° (no)	SSE 30° (no)	SSW 60° (no)	SW 90° (no)	SWW 120° (no)	NWW 150° (no)
Mean	1.9	1.6	1.1	1.8	1.3	1.0
Min	0.0	0.0	0.0	0.0	0.0	0.0
Max	5.0	5.0	5.0	6.0	5.0	4.0
75%-percentile	3.3	2.5	2.0	2.5	1.0	2.0
95%-percentile	4.1	4.1	5.0	6.0	2.2	2.1

Table 4-14. Mean values of the number of lost deposition holes expressed as a percentage of the theoretical total number of holes for inflow criteria of 10 l/min and 1 l/min. The analysis assumes a 300 m long deposition tunnel, at a depth of –500 m, with a total of 38 deposition holes and considers 20 different fracture network simulations for six different tunnel orientations.

Criteria	Tunnel orientation NW 0° (%)	SSE 30° (%)	SSW 60° (%)	SW 90° (%)	SWW 120° (%)	NWW 150° (%)
10 (l/min)	1.2	0.3	0.5	0.5	0.7	0.7
1 (l/min)	4.9	4.1	3.2	4.5	2.6	2.6

Application of the inflow criteria of 1 l/min per deposition hole results in 1–2 lost holes and 4–5 lost holes for the 95%-percentile of the allowable inflow, see Table 4-13.

If no adjustment is made to the hole positions then the mean number of holes lost for an inflow criterion of 10 l/min is < 1% for the tunnel orientations considered, N015 and N105, see Table 4-14. For an inflow criterion of 1 l/min the mean number of holes lost rises to approximately 3% for the considered tunnel orientations.

The analysis indicates that any change to the repository depth between –500 m and –400 m levels will not result in any significant change in the percentage loss.

The current model assumption, that the transmissivity is a function of the fracture length, leads to the conclusion that it is the larger structures, with radii R > 100 m, that are also responsible for the loss of deposition holes due to water inflows. Consequently losses due to the occurrence of fractures with a radius R > 100 m should not be added to the losses due to water inflow.

Potential wedge failure in deposition holes

The results from the wedge analysis show that the factor of safety against wedge failure is only marginally affected by the variation in stress magnitudes between the different domains and depths considered. Consequently, it has not been necessary to differentiate between the different stress domains and depths for the current presentation of results.

The output results for where the wedge volume is maximized and the factor of safety against wedge failure is minimized, for the 20 fracture combinations analysed, are presented in Figure 4-17. This case involves fracture combination (3, 4, 6) at –400 m in stress domain II. The results indicate that the factor of safety is sufficiently high (FOS > 200) for all of the potential wedges around the deposition hole to be considered stable.

The results clearly indicate that the loss of deposition holes due to wedge failure can be discounted for the Simpevarp sub area. The fracture shear strength is sufficiently high to ensure the stability of all potential wedges around the deposition holes.

Risk of spalling in deposition holes

The results from the spalling analysis are presented in Figures 4-18 and Figure 4-19. The calculated factor of safety (FOS) and probability of spalling (%PofS) is presented as a function of depth. In Figure 4-18 is also shown the factor of safety for the deposition holes in the prototype repository located at a depth of 450 m in the nearby Äspö HRL.

The results for stress domain I are summarised in Table 4-15 including the factor of safety (FOS) against spalling, probability of spalling (PofS), spalling depth and total spalling volume (V_{tot}) as a function of depth.

Overall the results show that there is essentially no risk of spalling in stress domain I at depths of –400 m to –500 m. However, at greater depths the risk increases and at a depth of –600 m the factor of safety against spalling drops below 1.0 and the probability for spalling is in the order of 50%. Results for a depth of –700 m indicate there is a clear risk that the majority of deposition holes located in stress domain I would be lost due to spalling. Where the volume of spalling in a deposition hole, in stress domain I at –700 m depth, is estimated to exceed the volume criterion stipulated for wedge breakout in UDP /SKB 2004a/ ($V_{tot} \geq 0.15$ m³/deposion hole) the hole is assumed to be lost.

The analysis indicates that for deposition tunnels placed in stress domain II there is no significant risk of spalling at any of the repository depths considered.

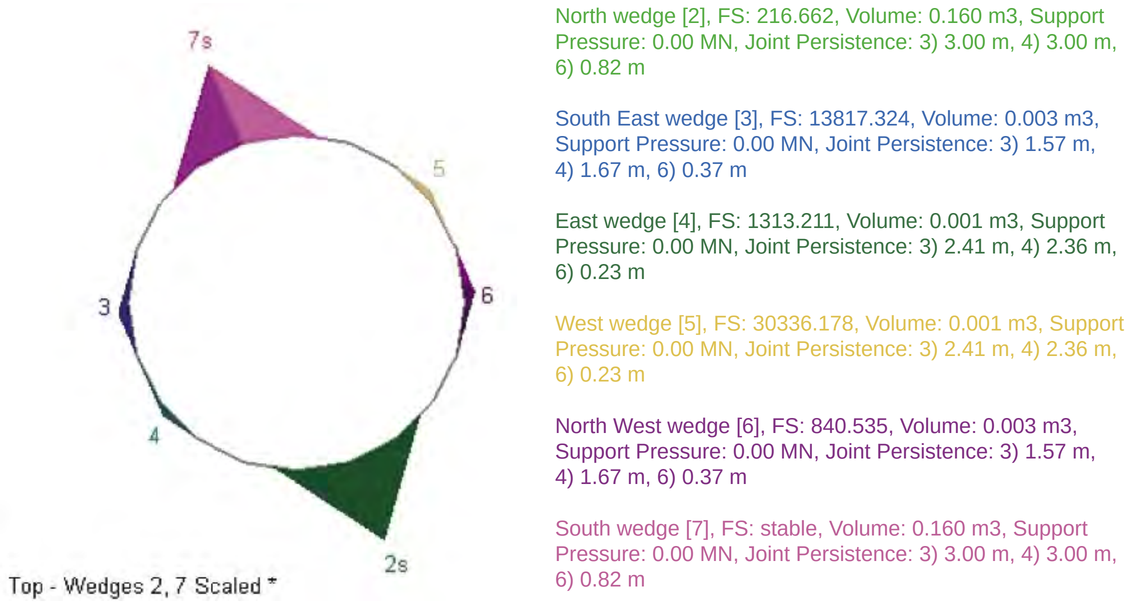


Figure 4-17. *Result output for fracture combinations 3, 4, 6, judged to be most critical of the different combinations analysed*

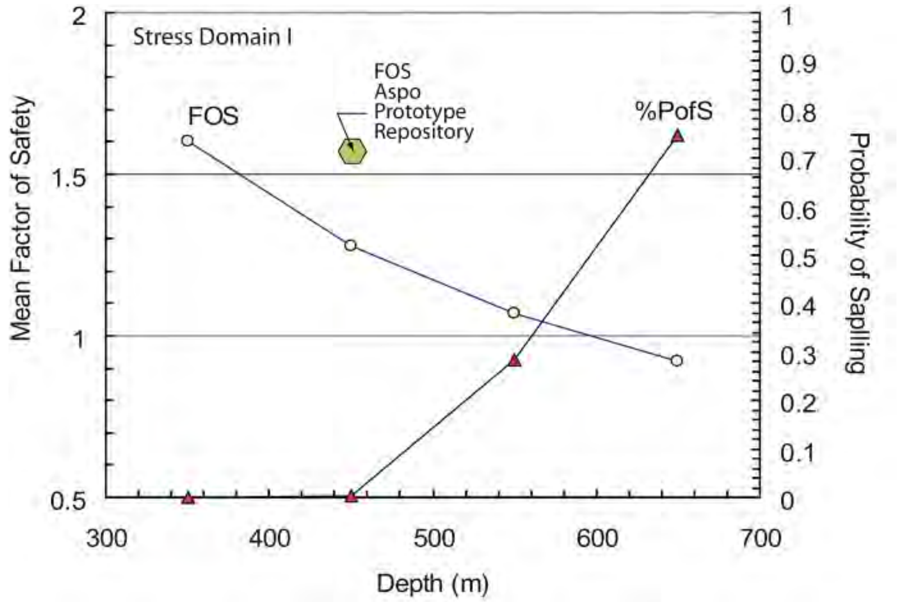


Figure 4-18. *Factor of safety and probability of spalling in deposition holes in stress domain I. The factor of safety for the deposition hole in the prototype repository in Äspö HRL is shown as a reference. When drilled this hole did not show any signs of spalling /Martin 2005/.*

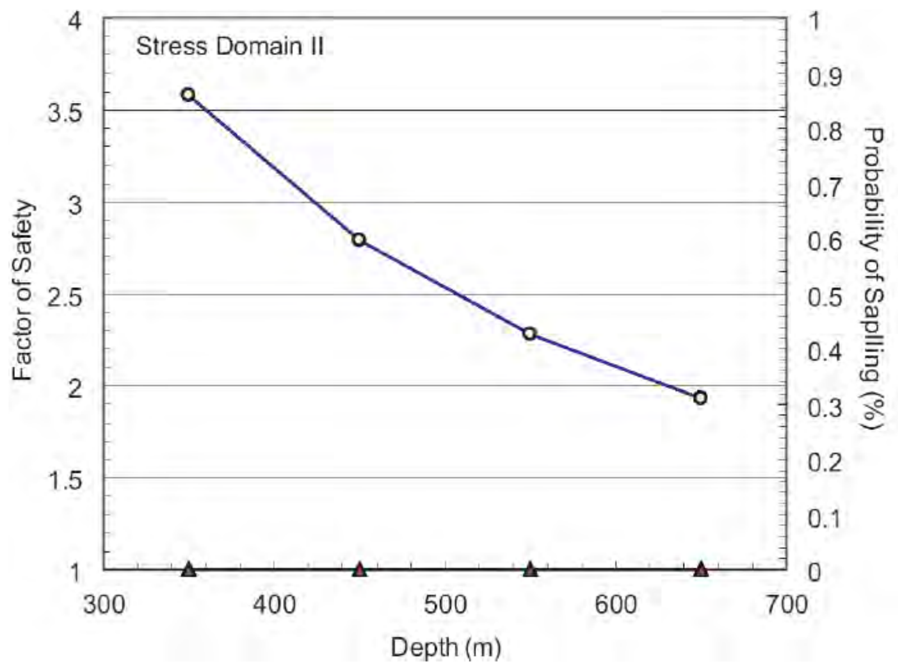


Figure 4-19. *Factor of safety and probability of spalling in deposition holes in stress domain II /Martin 2005/.*

It should be noted that there was no evidence of spalling observed in the deposition holes drilled in the prototype repository in the nearby Äspö HRL at a depth of 450 m. However, compared to the parameters assumed for Simpevarp the factor of safety is increased for the deposition holes in the prototype repository due to higher UCS (211 MPa) and higher minor horizontal stress /Martin 2005/.

Table 4-15. Factor of safety against spalling (FOS), probability of spalling (PofS), depth of spalling (Sd) and total spalling volume per deposition hole (Vtot) as a function of depth in stress domain I.

Depth	FOS	PofS (%)	S_d (m)	V_{tot} (m^3)
400	1.4	0	0.00	0.0
500	1.2	15	0.00	0.0
600	1.0	52	0.02	0.1
700	~ 0.8	~ 100	~ 0.06	~ 0.3

4.4.4 Conclusions

Intersection by large fractures is analysed by two different models, analytical and numerical as described in the sections above. The loss of deposition holes due to large fractures is according to the analytical model 13% and according to the numerical model approximately 10% of the deposition holes, both model results showed independence of the tunnel orientation.

Deposition hole losses due to unacceptably high water inflows are expected to be in the order of 1% for an inflow criterion of 10 l/min per deposition hole and 2–5% for an inflow criteria of 1 l/min per deposition hole depending on the tunnel orientation. Modification of the hole positions will enable this figure to be reduced somewhat.

Based on the assumption that large fractures carry more water than small fractures, the analysis indicates that it is these same larger structures with radii R > 100 m that result in the loss of deposition holes due to the fracture size that are also responsible for the larger water inflows. Consequently, these resulting two measures of hole loss should not be added.

A multitude of hydraulic size distribution models are possible to use in NAPSAC, for example un-correlated or semi-correlated relationships between transmissivity and fracture size. This could change the conclusions to some extent. However, the assumption used is considered realistic for fractured rock.

The results clearly indicate that the loss of deposition holes due to wedge failure can be discounted for the Simpevarp sub area. The fracture shear strength is sufficiently high to ensure the stability of all potential wedges around the deposition holes.

Overall the results show that there is essentially no risk of spalling in stress domain I at depths of –400 m to –500 m. However, at greater depths the risk increases significantly and at a depth of –600 m the factor of safety against spalling drops below unity and the probability for spalling is in the order of 50%. Results for a depth of –700 m indicate there is a clear risk that the majority of deposition holes located in stress domain I would be lost due to spalling. The analysis indicates that for deposition tunnels placed in stress domain II there is no significant risk of spalling at any of the repository depths considered.

In summary, it is concluded that the total combined deposition hole losses are 13% according to the analytical method or 10% according to the numerical method for the theoretical total. Losses are minimized by placing the repository at a depth of –400 m to –500 m.

4.5 Repository depth

4.5.1 Input data and assumptions

The assessment has been carried out in accordance with UDP guidelines. Input data for the analysis is based on the results from design tasks B and C1 to C4 as presented in Sections 3.2, 4.1, 4.2, 4.3 and 4.4 of this report.

4.5.2 Execution

In the earlier sections of the report, covering design tasks B to C4, the potential of the Simpevarp sub area to accommodate the repository has been assessed by examination of the following issues:

- Preliminary assessment of the potential of the site to accommodate the repository (B).
- Design layout of deposition tunnels, deposition holes and main tunnels (C1).
- Distance between deposition tunnels and between deposition holes (C2).
- Orientation of deposition tunnels (C3).
- Loss of deposition holes (C4).

The results from these earlier performed design tasks are synthesized in this section and a recommendation is made concerning the most advantageous depth for the repository.

4.5.3 Results and experience from design tasks B and C

Design Task B – The site's potential

Design task B concerned itself with a preliminary assessment of the available rock volume to accommodate a storage facility within the Simpevarp sub area.

A preliminary assessment of the site's potential to accommodate 6,000 canisters at a particular depth is presented in Section 3.7 and is based on the calculation of a *P* value – an SKB defined measure of the site potential, see Equation 3-3. The result clearly indicated that the site has clear potential to accommodate a storage facility. Whilst a –400 m repository depth has a higher P value than the –500 m level and therefore has a better potential, the difference is marginal.

Design task C1 – Tunnel design

Since the tunnel design does not vary with depth the results from design task C1 have no direct effect on repository depth. However, an increased depth leads to an increase in the excavation volume for the ramps and shafts along with associated increased costs.

Design task C2 – Deposition hole spacing

The aim of this aspect of the study was to determine the minimum allowable distance between deposition holes and deposition tunnels. The thermal properties of the rock mass and the thermal gradient at the site were assessed with reference to the maximum temperature allowed at the canister surface. This leads to the definition of minimum deposition hole spacing for each rock domain and potential storage level.

The assessment of canister spacing results in the overall conclusion that –400 m is the preferred repository depth, see Figure 4-6.

Design task C3 – Orientation of deposition tunnels

Design task C3 summarises the studies carried out to optimize the tunnel orientation of the deposition tunnels with a view towards minimizing the quantity of water inflows, the risk of spalling and the volume of potentially unstable wedges. Since the proposed orientation of the tunnels is equal over the potential depth range considered, the results from this design task have no direct influence on the selection of repository depth.

Design task C4 – Loss of deposition holes

For design purposes the repository needs to have a storage capacity for 6,000 canisters. In order to achieve this level a certain margin needs to be included to allow for the loss of deposition holes due to unfavourable rock conditions. Design task C4 covered the loss of deposition holes during the construction phase due to the existence of unacceptably long fractures, water inflows, wedge failures and spalling.

Of these issues only the risk of spalling was shown to be depth sensitive over the depth range considered and the results indicated that a shallower depth of –400 m to –500 m was favourable.

4.5.4 Discussion and conclusions

SKB have previously carried out studies to identify a general suitable storage depth for the repository KBS-3-system /SKB 2002c/. These studies have concluded that the depth of interest lies within the –400 m to –700 m depth range and that –500 m was determined as being the standard reference level. It is this depth range that has been considered in the current study with particular focus on –400 m to –500 m.

Generally the shallower the storage level the more favourable it appears to be from the point of view of rock properties generally and in particular beneficial for thermal, hydrogeological and in situ stress conditions. The results of the current study from the various design tasks all indicate that for the depth range considered, a storage depth of –400 m is most advantageous with the following benefits:

- Highest potential capacity, P (design task B).
- Lowest excavation volume for the ramp and shafts (design task C1).
- Minimum deposition area required (design task C2).
- Minimum total water inflow (design task C3).
- Minimum loss of deposition holes due to water inflows and spalling (design task C4).

There are however factors of importance for the long-term safety that are not considered in the UDP, and 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 as 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 Simpevarp 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 caverns, since these are available in Layout E /SKB 2002a/ and /SKB 2002b/.

Design of other underground openings (caverns in central area, shaft, ramp and transport tunnels) shall be carried out in design step D1, taking into account:

1. the required space for the activities to be pursued,
2. stability.

In design step D1, requirements according to space for the activities to be pursued shall be considered to be met if the design of caverns in the central area, shafts, ramp and transport tunnels takes place in accordance with facility description Layout E /SKB 2002a/ and /SKB 2002b/ with respect to:

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

Requirements according to stability shall in design step D1 is assumed to be met if the form and cross-sectional dimensions of other rock caverns according to facility description Layout E /SKB 2002a/ and /SKB 2002b/ are applied and rock support according to UDP Section 5.10 is installed.

4.6.2 Theoretical rock caverns and other excavations

The dimensions utilized in the design work are from Layout E /SKB 2002a/ and /SKB 2002b/.

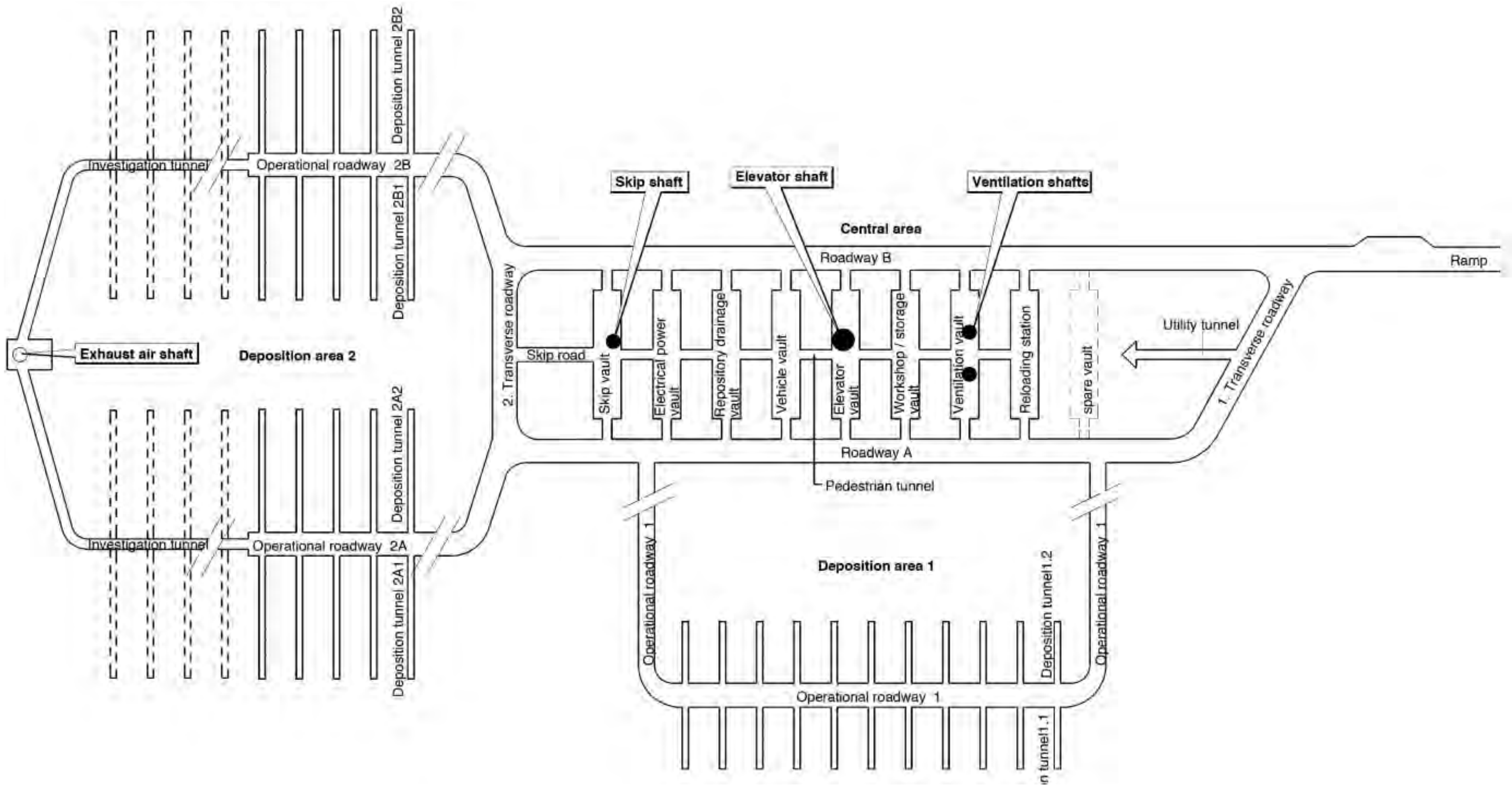


Figure 4-20. *Schematic plan showing underground openings.*

5 Layout studies

5.1 General

A central question for design step D1 is whether or not there is sufficient available rock volume, of sufficient quality, to accommodate the repository. Design task E concerns itself with the development of a potential layout for the repository based on the results and conclusions drawn from preceding design tasks A to C5. The work has been carried out in accordance with UDP guidelines. For the purposes of the layout work the number of deposition holes to identify is dependent on the expected loss according to Section 4.4, and here two different methods where utilised, an analytical and a numerical one. The result from the analytical method, a loss of 13%, was presented late in the design work why the layouts were designed only for the result of the numerical method, a loss of 10%. The repository is assumed to require a storage capacity for 6,600 deposition hole positions including a base requirement of 4,500 canisters, allowance for a potential increase by a further 1,500 due to an extended production period and an additional 10% margin to allow for the loss of deposition holes due to unfavourable local rock conditions.

In accordance with SKB requirements the proposed layout assumes the repository surface *Operational area* is located in close proximity to Clab, on the Simpevarp peninsula. However, an alternative location on Hålö has also been considered.

The proposed layout is presented for the –500 m reference depth. The layout has been developed based on the results and conclusions from the preceding design tasks, which themselves took into consideration the variability in the rock mass properties as stipulated by UDP.

A sensitivity analysis has been carried out investigating how changes in the design criteria and rock mass properties, such as the dip of the various deformation zones and variation in thermal conductivity, would affect the excavation and available storage volumes.

For the purposes of the study the location for the repository was defined by SKB to lie within the *Simpevarp interest area* /SKB 2003/. The study area is shown in grey in Figure 3-1. Two types of safe operational and working distances were defined for design task B, *Respect Distance* (RD) and *Margin for Excavation* (MFE); see Section 3.2 of this report. These safety margins were applied to both the high and low deformation zones to define the rock block volumes, termed *Deposition Blocks* (DB), available for deposition. The results are presented in Figures 5-2 and 5-4 for the reference level, –500 m.

The layout excavation geometry is based on use of the tunnel cross-sections presented in Section 4.1, Figure 4-3. These section geometries are themselves based on those presented in SKB's Layout E, and a revised generic model /SKB 2004d/. Deposition hole design geometry remains unchanged with a diameter of 1.75 m and a length of 8 m.

The minimum allowable deposition hole spacing for the various rock domains and depths considered was investigated under design task C2, see Section 4.2 of this report. For the purposes of the layout work, the relevant spacing distances have been rounded off to the nearest 0.5 m, resulting in a uniform hole spacing of 7.5 m for all deposition blocks and rock domains involved at –500 m depth.

The optimal orientation for the deposition tunnels was considered under design task C3, see Section 4.3 of this report. The study identified an orientation of N015°E as being optimal, based mainly on a reduction achieved in potential water inflow levels. An orientation of N105° was suggested as a slightly less favourable alternative, however, the variations between different orientations are marginal and it can be said that tunnel orientation is not a particularly sensitive variable for the rock properties considered, based on the data currently available.

It should be noted that the geometry of the deposition blocks ought to be taken into account when evaluation optimal tunnel orientation. By selecting a tunnel orientation strictly according to the requirements in UDP, the possible layout alternatives naturally become restricted, which may lead to the potential of the site not being fully utilized.

In accordance with Layout E /SKB 2002a/ it is assumed that an initial deposition area in close proximity to the central area, deposition area 1, will be completed and utilized prior to the excavation and utilization of the much larger *deposition area 2*. UDP states this initial phase requires storage for 200 to 400 canisters, while the main phase involves 5,600 to 5,800 canisters.

5.2 Execution

The proposed layout presents a general arrangement to test whether or not the study area has sufficient storage volume to accommodate the repository. It has been carried out at a feasibility study level and therefore does not constitute a detailed design.

The proposed layout is based on the principles presented in Layout E /SKB 2002a/ and /SKB 2002b/ with minor revisions having been made to the tunnel cross sections. The central area, shaft and ramp design geometry have been taken directly from the generic design without modification.

The layout is to a large extent controlled by the distribution of the deformation zones that criss-cross the study area. The selected orientation of the deposition tunnels in combination with the geometry of the resulting deposition blocks controls the possible deposition tunnel layouts with limited opportunity for flexibility. However, evacuation and safety aspects during construction and operational phases are allowed for, including continuous transport loops in the tunnels with the avoidance of dead-ends.

The layout allows for 6,600 canisters taking into account margins for losses due to locally unsuitable rock conditions and a possible extended operational time for the nuclear power stations.

Layout reference level –500 m

The proposed layout consists of 9 different deposition units lying within 7 deposition blocks. A deposition hole spacing of 7.5 m has been used throughout the layout.

The deposition unit quantities and respective tunnel lengths are summarised in Tables 5-1 and 5-2. The layout is presented as a top and a 3D view in Figure 5-1 and 5-2. The deposition tunnels have a minimum tunnel crown depth of –498 m and slope towards the central area with a general gradient of 1:100 to guarantee the drainage of the repository. The tunnels follow two alignments, N015°E and N105°E.

Table 5-1. Summary of deposition unit quantities, level –500 m.

	Deposition units									
	A	B	C	D	E	F	G	H	I	Tot
Deposition tunnels (no)	20	12	46	16	9	12	41	30	27	213
Max length dep. tunnel (m)[1]	142	292	292	292	292	292	292	292	292	
Min length dep. tunnel (m)[1]	112	120	150	292	105	142	105	202	127	
Total length dep. tunnel (m)[1]	2,740	2,675	13,046	4,676	2,188	3,080	10,700	8,513	6,646	54,262
Deposition holes (no)[2]	306	321	1,603	576	265	375	1,305	1,046	806	6,603
Canisters (no)	278	291	1,457	523	240	340	1,186	950	732	5,997

1) From main tunnel wall.

2) Including loss of 10%.

Table 5-2. Summary of tunnel lengths in the layout, level –500 m.

	Total length
Deposition tunnel length[1] (m)	54,262
Transport tunnel (m)	5,321
Main tunnel (m)	5,220

1) From main tunnel wall to deposition tunnel end.

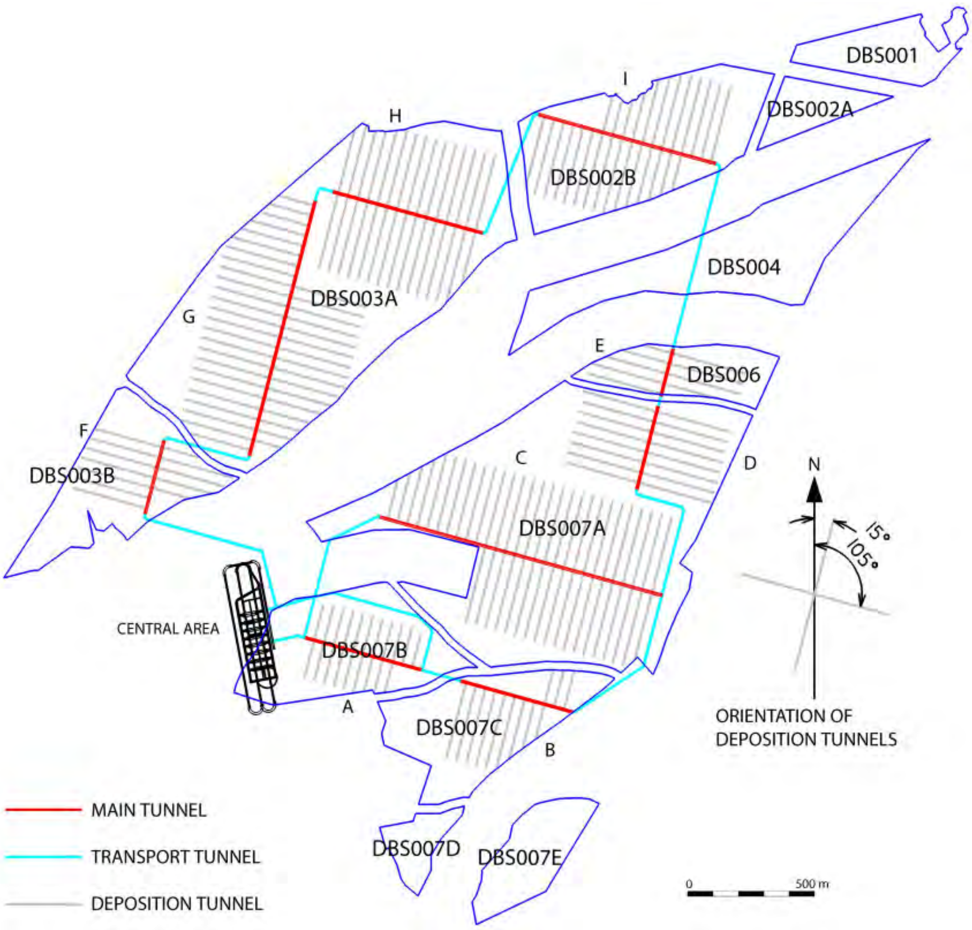


Figure 5-1. *Layout. Level –500 m.*

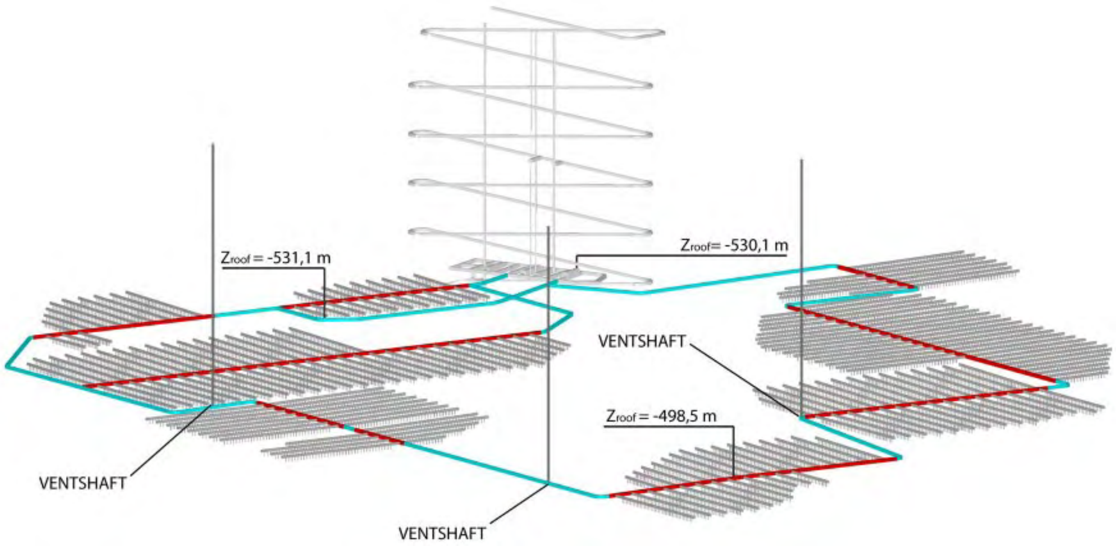


Figure 5-2. *Layout with depth level.*

The layout presents both deposition areas 1 and 2. Deposition area 1, for the initial phase includes the necessary initial transport loop connection to the ramp and shaft. Deposition area 2 consists of deposition units B to I. The overall layout allows for a continuous transport loop in the deposition area. However, to create a flexible transport system with reasonable transport times it will be necessary to include some additional local transport loops in the detailed design. These additional transport loops may require one or two further tunnel passages and reduce the available space in the deposition blocks to some extent.

The deposition tunnel lengths have been minimized in accordance with Layout E /SKB 2002a/.

Central area

The overall design of the central area remains unchanged from the generic design described in Layout E /SKB 2002a/. The proposed caverns have been located outside the MFE boundaries to simplify excavation and reduce costs.

Ramp and Shaft

The design of the ramp and shaft remain unchanged from the generic design described in Layout E /SKB 2002a/. For the detailed design stage, when more detailed information will be available concerning the variability of the rock mass, the ramp design can be optimized and avoid problematic deformation zones or minimize passage lengths.

Rock excavation volume

Estimates of excavation volumes are presented in Tables 5-4 and 5-7. The values represent undisturbed theoretical rock volumes based on the 3D proposed design layouts. The cross-sectional areas of the various excavations used in the calculations are presented in Table 5-3.

Table 5-3. Cross-sectional areas of the different excavations.

Facility part	Cross-sectional area (m^2)
Main tunnel	66.1
Transport tunnel	46.3
Deposition tunnel	25.0
Deposition hole	2.4
Ramp	31.0
Elevator shaft	23.8
Skip shaft	23.8
Ventilation shaft	4.6 and 9.6

Table 5-4. Theoretical rock excavation volumes per facility part, excluding central area, ramp and shafts.

Facility part	Excavation volume (m^3)
Transport tunnel	246,600
Main tunnel	345,000
Deposition tunnel	1,355,000
Total tunnel volume	**1,946,600**
Total deposition hole volume	**127,300**
Total theoretical rock excavation volume	**2,073,900**

Development and excavation sequence of the underground facility

The repository excavation sequence is described in Layout E /SKB 2002a/. Excavation begins with the Ramp and continues with the central area and then the main, transport and deposition tunnels along with the deposition holes that constitute deposition area *A*.

The excavation sequence for deposition area B begins with investigation tunnels covering the extent of the entire planned repository. These will subsequently be enlarged to form the main tunnels. An exhaust ventilation shaft, located remotely from the central area, is drilled and then raise-bored followed by excavation of a series of deposition tunnels. The current layout allows for deposition in a stepped sequence prior to complete excavation of the entire repository.

Maximum potential storage capacity

In order to assess the full potential storage capacity of the site a modified design layout was developed that utilized the entire available rock volume. The results are presented in Table 5-5.

For the presented layout the additional 1,107 canisters represent an 18% increase in capacity including allowance for loss of deposition holes.

The layout design presented is based on a 10% loss of deposition holes, but utilising the analytical method according to Section 4.4 the loss should be slightly higher resulting in a 13% loss. If the additional loss should be taken into account this would result in 5 extra deposition tunnels which could be fit into the presented layout after minor adjustments.

Table 5-5. Additional canister capacity based on maximum exploitation.

Depth (m)	Total surface used for deposition (m²)	Remaining surface, A_R (m²)	Equivalent no of dep.holes for A_R excluding losses (no)	Equivalent no of canisters for A_R including losses (no)
–500	4,120,000	674,824	1,230	1,107

5.3 Alternative possibilities for site adaptation

5.3.1 Sensitivity analysis

As the project proceeds and more information becomes available, the underlying geoscientific and hydrogeological models that define the underlying geometrical framework for the design will be modified. These changes will require sequential modifications to be made to the design. In order to assess the effect of such possible changes in the various parameters on the design, a simplistic sensitivity analysis has been carried out in accordance with Section 5.6.2 of the UDP guidelines /SKB 2004a/. These issues have been further developed by risk analysis with results presented in Section 10 of this report.

The following parameter changes were taken into account:

- Changed premises with respect to the occurrence of deformation zones (confidence level that they exist).
- Changed premises with respect to the orientation of deformation zones (strike and dip).
- Changed premises with respect to the RD of the deformation zones.
- Changed premises with the respect to the mean value of thermal conductivity of the rock in conjunction with the choice of distance between deposition holes.
- Changed premises with the respect to the criteria for calculation of loss of deposition holes.
- Changed premise with the respect to the maximum length of deposition tunnels, assuming an increase to 600 m.

The sensitivity analysis was carried out based on application of the changes listed in Table 5-6 to the proposed layout. This resulted in six modified layout arrangements with variation in theoretical excavation rock volume and enclosed utilized deposition area. All layout alternatives have a storage capacity of 6,600 deposition holes. The results are presented in Tables 5-7 to 5-8.

Table 5-6. Activities carried out as part of the sensitivity analysis.

ID no	Variable	Variation
1	Zone geometry	Extension of ZSMNE018A and ZSMNE024A
2	Zone orientation	Dip adjusted ±15° to give the least favourable orientation
3	Zone respect distance	No variation applied
4	Deposition hole spacing (based on thermal conductivity)	Thermal conductivity ±7.5% for DB001–006 and. +5% for DB007.
5	Criteria for loss of deposition holes	Increased loss by 2%-per deposition block due to raised water inflow levels.
6	Maximum length of deposition tunnel	Increased from 300 m to 600 m

Table 5-7. Changes in rock excavation volume resulting from application of variations defined in Table 5-6.

ID no	Layout alternative	Excavation volume, V[1] (m³)	ΔV (m³)	Relative change (%)
0	Base	2,073,942		
1a	Zone geometry	2,070,457	–3,484	–0.2
2	Zone orientation	2,240,482	166,540	8.0
4a	Dep. hole spacing decrease.	1,973,995	–99,947	–4.8
4b	Dep. hole spacing increase.	2,203,986	130,044	6.3
5	Loss increase	2,105,927	31,985	1.5
6	Max length of dep. tunnel increase	1,971,869	–102,073	–4.9

1) Does not include the central area, ramp or shafts.

Table 5-8. Changes in enclosed utilized deposition area Au, resulting from application of variations defined in Table 5-6.

ID no	Layout alternative	Exploited deposition Area, A_u (m²)	Δ A_u (m²)	Relative change (%)
0	Base	4,461,280		
1a	Zone geometry	4,529,101	67,821	1.5
2	Zone orientation	4,224,218	–237,062	–5.3
4a	Dep. hole spacing decrease.	4,348,375	–112,905	–3.2
4b	Dep. hole spacing increase.	4,675,751	214,471	6.0
5	Loss increase	4,499,065	37,785	0.9
6	Max length of dep. tunnel increase	4,165,911	–295,369	–6.6

The two variations representing natural changes in the rock mass conditions that had the greatest effect on the excavation volume and the utilized deposition area were dip of the deformation zones and the thermal conductivity of the rock. These resulted in variations in the order of ±5–8%.

5.3.2 Localization of the above ground facilities

The localization process needs to consider both existing land use and longer term environmental issues. For example, a position within or near an existing industrial area would give access to an already established transport and utilities system, avoiding the exploitation of a completely new undisturbed site. A possible alternative to an industrial area would be a site located within an area currently used for commercial forestry, with little other land use interests.

Following the generic design layout E /SKB 2002a/ and /SKB 2002b/ it is the positioning of the underground central area that to a large extent controls the location of the above ground facilities since they are connected by vertical shafts.

Currently SKB has targeted a location in close proximity to Clab, see Figure 5-3. Hålö was considered as an alternative location during the earlier preliminary design work, see Figure 5-5.

Ventilation building

To ensure the underground facility is sufficiently well ventilated requires a separate ventilation centre outside of the main operational area. A number of variables must be taken into account when considering the placement of the required shaft and surface buildings, however, the most critical is that the shaft must be placed remotely from the central area in order to ensure functionality.

Alternative locations for the ventilation building and shaft based on operational areas located at Clab and Hålö are presented in Figures 5-4 and 5-5 of this report.

Operation Area in close proximity to Clab

The main advantages with this location are that the facility will fall within an area already designated for industrial use, the area can be accessed by existing infrastructure and canister transport can be confined within the operational area, see Figure 5-3.

Ventilation building

Potential locations for the ventilation shaft and building are presented in Figure 5-4. A location on the Simpevarp peninsula has the advantage that the area is already designated for industrial use, though positioning must be agreed with OKG, the owner-operator of Oskarshamn's nuclear facilities.

A location on the Hålö peninsula has the advantage that there is very little residential property though the proposed location would fall within the 300 m shoreline protection zone. Additionally, the area has been designated as being of interest from the point of view of nature conservation and recreation.

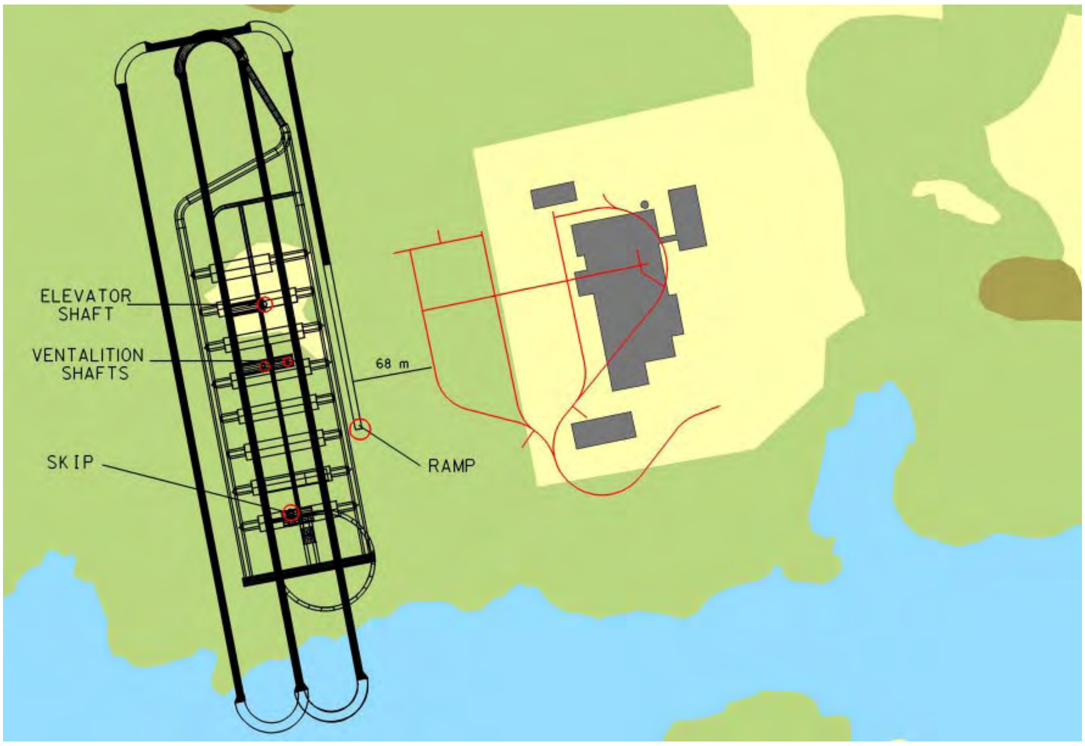


Figure 5-3. *Layout showing the central area, shafts and ramp along with CLAB's existing surface buildings (grey) and tunnel centre lines (red).*

Figure 5-4. *Layout with existing surface infrastructure. Alternative locations for the ventilation building are marked 1 to 3.*

A location on Ävrö is presented in Figure 5-4. However, the area is designated as being of interest for energy production, nature conservation and recreation. If the Ävrö area ever took part in energy production then this area would be of increased interest.

Operation Area on Hålö

From a functional point of view Hålö would be a possible location for the Operational area, see Figure 5-5. However, the location is in close proximity to the village of Lilla Laxemar and the area has been designated as being of interest for conservation and recreation.

Ventilation building

Potential locations for the ventilation shaft and building are presented in Figure 5-5.

Discussion

Placing the central and operational areas in close proximity to Clab has clear advantages related to existing land use, infrastructure and canister transport.

Hålö has the advantages of greater surface area, functionality and sparse population. Disadvantages centre on the fact that the area has been designated as being of interest for conservation and recreation.

The alternative locations for the ventilation shaft and building have been ranked with position 1 being judged most advantageous for both Clab and Hålö. However, the future detailed design work may identify the need for multiple ventilation shafts.

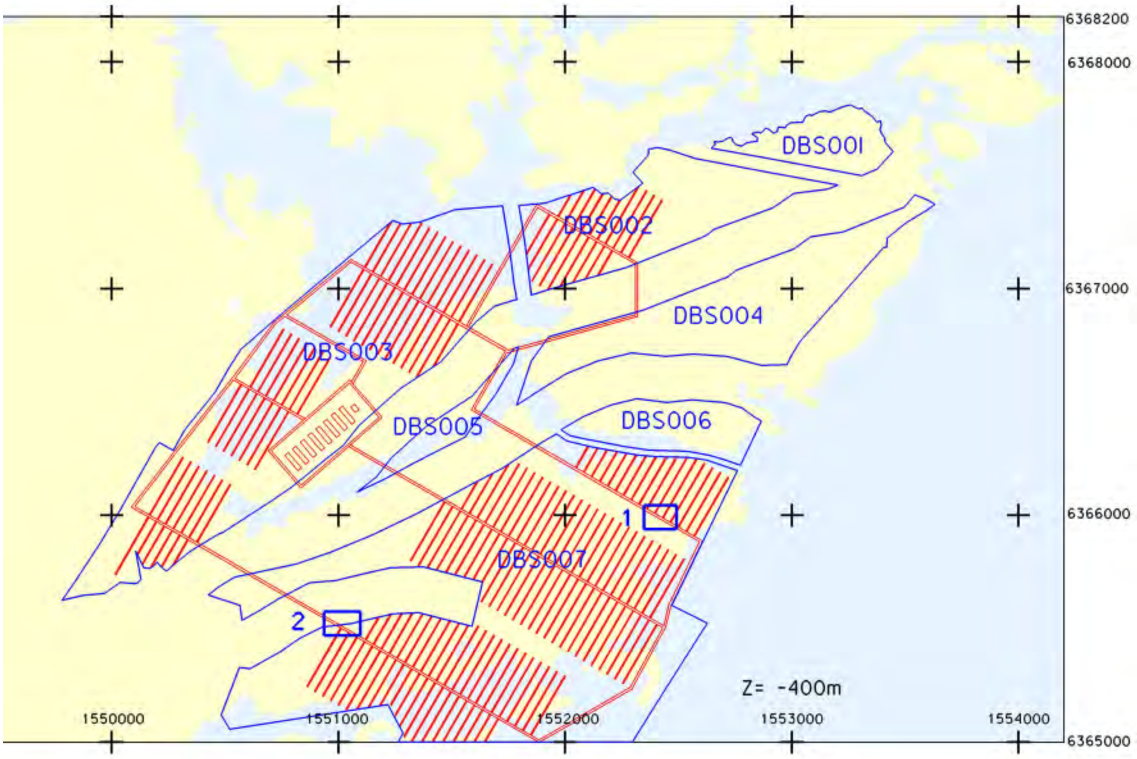


Figure 5-5. Layout with the central and operational areas located on Hålö. It should be noted that the layout shown was developed during an earlier preliminary design stage. Potential locations for the ventilation shaft and building are marked 1 and 2.

5.4 Conclusions

The main goal of the D1 design is "to confirm that site dimensions are large enough to accommodate the required size for a deep repository" according to UDP /SKB 2004a/.

The D1 design layout at level –500 m shows sufficient space and volume are available at the site for the anticipated number of 6,000 canisters. The anticipated volume for the Deposition area is approximately 2 million m³ including 65 km of tunnels and deposition holes.

To allow for uncertainties that are inherent in such an early stage of design, a sensitivity analysis has been performed. Of the various parameter changes considered by the analysis, the variation in the dip of the deterministic deformation zones and change in thermal conductivity of the rock, proved to have the greatest impact on the proposed storage layout. These parameter changes resulted in a ±5–8% variation in excavation volume and utilisation of the deposition areas. An increase in the maximum allowable deposition tunnel length from 300 to 600 m leads to a significant improvement in the efficiency of the layout with an improved excavation/deposition volume ratio.

6 Identification of passages through deformation zones

6.1 Input data and assumptions

This design task deals with the identification and assessment of tunnel passages through deformation zones. The aim being to create a basis for determining measures with respect to excavation, grouting and rock support in these tunnel sections, and to permit comparisons between different layouts by relating the number of passages and the total length of passages to the rock quality. The assessment assumes tunnel excavation is carried out using conventional drill and blast techniques; grouting is generally cement based and conventional rock support is used, consisting predominantly of rockbolts and shotcrete.

The initial basis for the work is the geometrical framework defined by the deformation zones. The deformation zone geometries have been taken from the Simpevarp site descriptive model version 1.2 /SKB 2005a/. The transport tunnels between the deposition units will have to pass through these deformation zones and the likely rock conditions and excavation problems need to be assessed as well as potential for seepage and long term stability. The passage locations are based on the proposed layout presented in design task E, Section 5 of this report.

The study only includes passages in the deposition area. Zone interceptions along the ramp to the central area and along the transport tunnels in the central area are not considered. However, the latter passages, which due to repeatedly intercepted deformation zones result in a total length of approximately 560 m only in the ramp, are included in the estimation of rock grouting and rock support requirements presented in Chapters 8 and 9.

The proposed layout at a depth of –500 m takes into account the existence of both high and “possible” deformation zones. The design layout resulted in a total of 11 passage locations, see Figure 6-1. The affected tunnel sections are assumed to be of standard transport tunnel dimensions with 7 m width, 7 m height and a cross-sectional area of 46.3 m².

For any particular deformation zone the thickness and engineering geological properties have been assumed to be uniform and neither vary laterally nor with depth. This is clearly a gross simplification of the likely spatial distribution of rock conditions. In reality both geometry and properties will vary significantly for any deformation zone. However, the properties presented are considered conservative and make allowance for this variation.

Certain properties, such as degree of weathering and transmissivity, are likely to be depth dependent to some degree. Since the majority of the borehole-deformation zone intersections are at relatively shallow depths, many of the deformation zones are likely to show improved characteristics from an engineering viewpoint at the expected repository depth.

Input data concerning the extent and character of the deformation zones have been taken principally from the site descriptive model S1.2 /SKB 2005a/. Supplementary information has been taken from local excavation experience as documented during the excavation of the Äspö Hard Rock Laboratory (HRL), the Central Interim Storage Facility for spent nuclear fuel (Clab 1 and 2) and the Oskarshamn nuclear power stations (O1, O2, O3), along with information in background reports to the Laxemar site descriptive model 1.2 /Curtis et al. 2003ab/; /Hultgren et al. 2004/; /Lindqvist 2004/; /Markström and Erlström 1996/; /Mattsson et al. 2004/; /SKB 2005a/; /Rhén et al. 1997/; /Stanfors et al. 1997/;

/Chang et al. 2005/. Rock quality assigned to the deformation zone passages has been interpreted by the design team from information gathered in the references quoted above. In some cases engineering geological parameters are directly detailed and for other projects the information available is more limited. In such cases the assessment is based on the information available, along with a simple comparison with better documented deformation zones judged to be of similar dimensions and character.

6.2 Execution

This design task has been carried out in accordance with SKB guidelines as presented in Section 5.7 of UDP, /SKB 2004a/, and was based on the following steps:

1. Identification of passages through deformation zones to provide access routes between different parts of the repository.
2. Classification of each passage with respect to rock quality.
3. Estimation of the length, L (m), of each passage.
4. Estimation of anticipated difficulties associated with each passage with regards to excavation, rock support and grouting based on empirical knowledge, and how these difficulties can be overcome.

The identification of passages is based on the layout set at –500 m depth as presented in Figure 6-1.

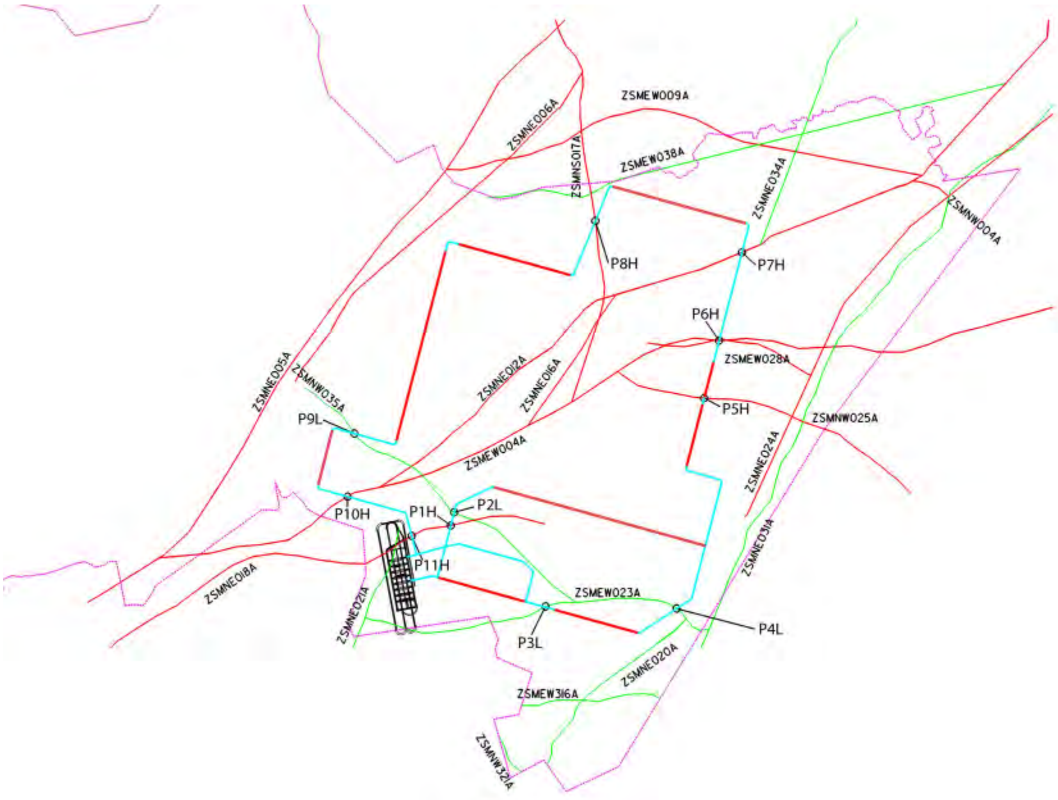


__Figure 6-1.__ Proposed layout for –500 m level. High confidence zones are shown in red. "possible" zones are shown in green. Passage locations are labelled P with a sequential number and H or L indicating a High or "possible" classification for the zone.

6.3 Results

Identification of passages

The proposed layout at –500 m depth including the deformation zones, with the passage locations identified, is presented in Figure 6-1. High confidence deformation zones are shown in red and "possible" zones in green. A summary of the deformation zone passages and supporting data are presented in Table 6-1.

The definitions of the terms; zone width, interception angle and passage length, used in Table 6-1, are presented in Figure 6-2.

Table 6-1. Summary of the deformation zone passages and supporting data.

Passage ID no	Def. zone ID no	Basis for interpret-ation[1]	Zone thickness		Dip[1]	Interception angle	Passage length[4]
			t[1] (m)	t[2] (m)	(degree)	(degree)	(m)
P1H	ZSMNE018A	B, G	30	15–30[3]	90	63	38
P2L	ZSMNW035A	G, Tg	20	no value	90	79	22
P3L	ZSMEW023A	G, Tg	20	no value	90	33	47
P4L	ZSMEW023A	G, Tg	20	no value	90	73	23
P5H	ZSMNW025A	B, G	5	10	88	80	7
P6H	ZSMEW004A	B, G, T	30 ± 20	30	70 ±15	80	34
	ZSMEW028A	B, G	10		83		
P7H	ZSMNE012A	B, G, T	41	50	50 ±15	53	72
	Del A			10			18
	Del B			40			55
P8H	ZSMNS017A	B, G, T	20 (0.5–10)	20	90	31	51
P9L	ZSMNW035A	G, Tg	20	no value	90	39	40
P10H	ZSMEW004A	G, T	30 ± 20	30	70 ± 15	46	52
P11H	ZSMNE018A	B, G	30	15–30[3]	90	78	32

Abbreviations: B – Bore hole, G – Geophysics, T – Tunnel, Tg – Topography

1) /SKB 2005a/.

2) /Markström and Erlström 1996/.

3) /Curtis et al. 2003ab/.

4) Value based on zone thickness as presented in /SKB 2005a/.

Geological description of deformation zone crossings

Geological information concerning the deformation zones is available in the site description S1.2 and also from other reports covering the Simpevarp area /Curtis et al. 2003ab/; /Hultgren et al. 2004/; /Lindqvist 2004/; /Markström and Erlström 1996/; /Mattson et al. 2004/; /SKB 2005a/; /Rhén et al. 1997/; /Stanfors et al. 1997/. An assessment of the available data allows the following generalizations to be made as regards typical deformation zone characteristics relevant to the current study:

- there are three or more fracture sets,
- fracture frequency is 10–20/m,
- fracture surfaces are planar and smooth to undulating and rough,

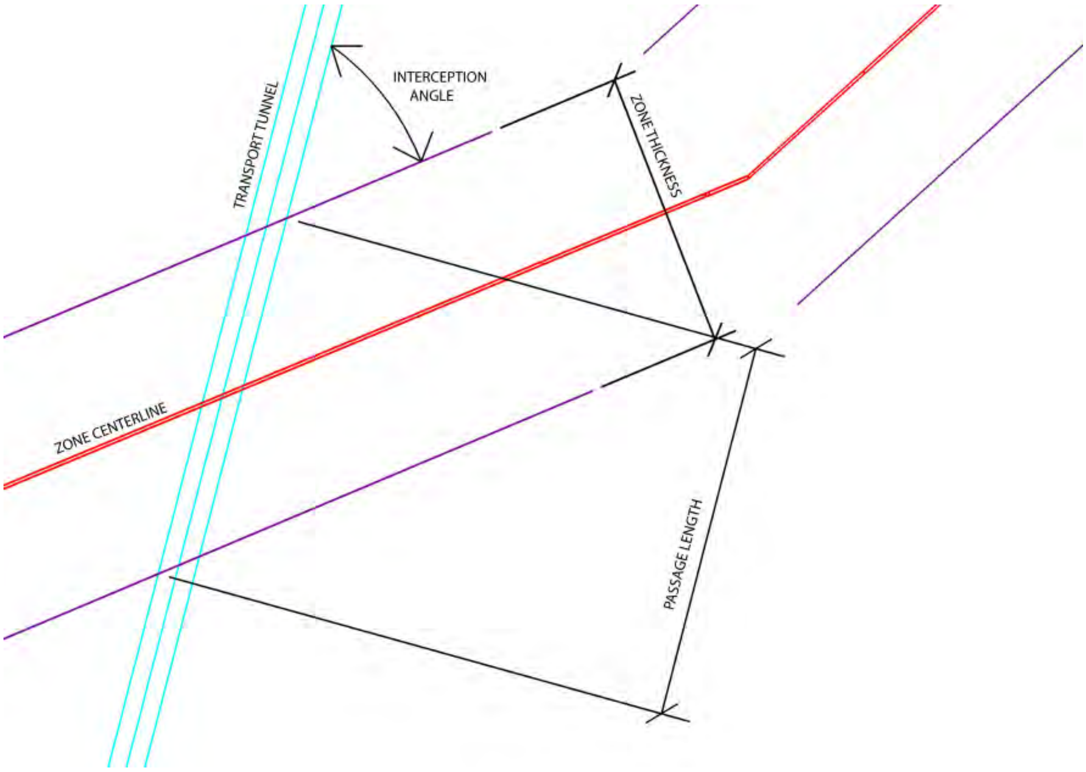


Figure 6-2. *Definition of zone thickness, interception angle and passage length used in Table 6-1.*

- degree of weathering slight to high,
- fracture fillings consist of chlorite, calcite, epidote and clay.
- seismic velocities variy between 2,400–4,200 m/s
- transmissivities vary from 10^{-4} to10^{-7} m^2/s.

Based on the geological information available an attempt has been made to subdivide the zones into three general classes with similar characteristics and geometry. Simplistic sketches representing the three different types A, B and C are presented in Figures 6-3 to 6-5.

Type A- consists of a 5–10 m wide core of highly fractured granite with mylonite and often clay alteration. In certain cases a distinct clay core of up to 1–2 m is present. The core is bounded on either side by a 10–20 m wide transition zone of fractured quartz- monzo-diorite, fine grained granite or Ävrö granite. The sketch shows the transition zones to be symmetrical around the core, however, this is unlikely to be the case in reality.

A Type B deformation zone is presented in Figure 6-4. Type B consists of a 5–30 m wide zone of highly fractured rock. The zone is commonly associated with an igneous intrusion of fine-grained granite (aplite) or fine grained diorite-gabbro (greenstone). A 0.1 to 0.5 m contact boundary zone containing chlorite or clay commonly occurs along the junction with the competent host rock.

A Type C zone is presented in Figure 6-5. This type consists of a 10–30 m wide zone of fractured quartz monzodiorite or Ävrö granite. It includes a series of parallel minor shear zones or fracture groups that may be strongly water bearing, however, the overall geometry of the zone may be less well defined.

Type A and B zones could lead to both stability and water inflow problems, whilst Type C zones are more likely to be associated with the risk of major inflows into any tunnel. Potential problems associated with the passage of deformation zones are discussed and proposed rock support are presented in subsequent sections.

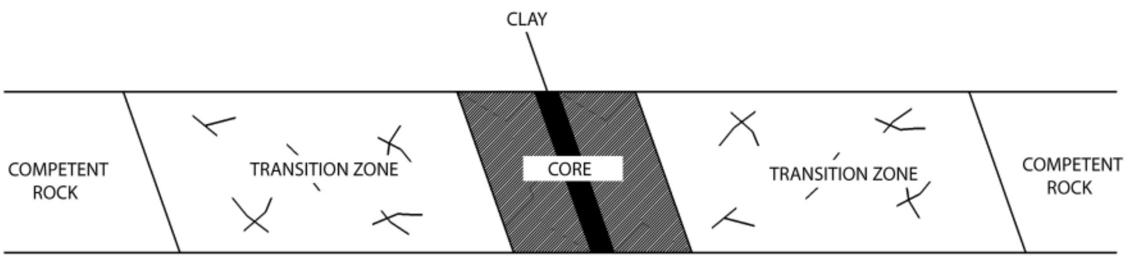


Figure 6-3. *Schematic figure representing a Type A deformation zone geometry. Modified after /Chang et al. 2005/.*

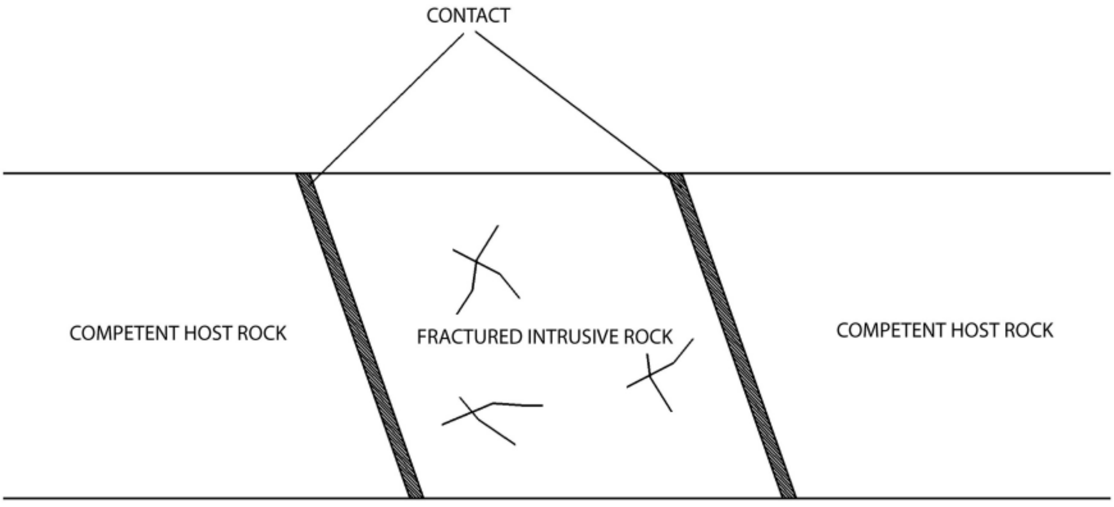


Figure 6-4. *Schematic figure representing a Type B deformation zone geometry.*

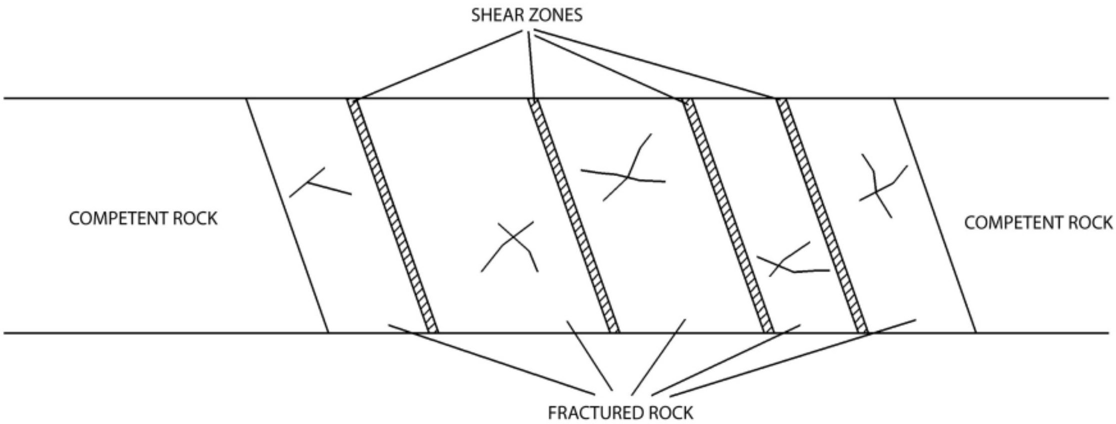


Figure 6-5. *Schematic figure representing a Type C deformation zone geometry.*

Classification of passages due to rock mass quality

The rock mass quality for each passage has been assessed in accordance with the Q system /Barton 2002/, based on the geological information compiled for the various deformation zones. The amount and character of the information available varies considerably and this variability affects the amount and degree of certainty in relation to the input data for the rock mass classification. The rock mass classification of Passages P2L and P9L, involving deformation zone ZSMNW035A, has a particularly weak basis.

For a number of deformation zones seismic refraction velocity data is available /Linqvist 2004/. In such cases rock mass quality has also been assessed based on the established empirical relationship between seismic velocity and the Q system /Barton 2002/.

$$Q \approx 10^{(Vp-3500)} \qquad \text{Equation 6-1}$$

where,

V_p = Seismic velocity (m/s)

Seismic velocities and the estimated rock mass quality based on Equation 6-1 are presented in Table 6-2. It should be noted that as with all seismic refraction surveys, the measured velocities are measured at relatively shallow depths and it is expected that velocities and rock mass quality would be higher at repository depth.

The subdivision of the deformation zones according to Type and the assessed rock mass quality are presented in Table 6-3.

The number of passages as a function of rock quality and confidence class is presented in Figure 6-6. The combined passage tunnel lengths as a function of rock mass quality are presented in Figure 6-7.

For passage P7H the relevant deformation zone is subdivided into A and B based on rock mass quality. For this case the poorer quality has been applied to the passage for Figure 6-6 whilst the zone subdivision is included directly in Figure 6-7.

Table 6-2. Rock mass quality estimates based on measured seismic velocity.

Passage ID no	Deformation zone ID no	Seismic velocity	Rock mass quality	
		V_p[1] (m/s)	Q-index[2]	Class (Q)
P1H	ZSMNE018A	no data		
P2L	ZSMNW035A	no data		
P3L	ZSMEW023A	2,800–3,700	0.1–1	Very poor
P4L	ZSMEW023A	2,800–3,700	0.1–1	Very poor
P5H	ZSMNW025A	2,400–3,700	0.1–1	Very poor
P6H	ZSMEW004A	3,900–4,200	1–4	Poor
	ZSMEW028A	no data		
P7H	ZSMNE012A	2,500–3,600	0.1–1	Very poor
P8H	ZSMNS017A	no data		Very poor
P9L	ZSMNW035A	no data		
P10H	ZSMEW004A	3,900–4,200	1–4	Poor
P11H	ZSMNE018A	no data		

1) /Lindqvist 2004/.

2) /Barton 2002/.

Table 6-3. Classification of deformation zones according to Type and rock mass classification based on mapping of existing tunnel intersections.

Passage ID no	Deformation zone ID no	Zone Type	Rock mass quality	
			Q-index	Class (Q)
P1H	ZSMNE018A	A	0.1–1[1, 2]	*Very poor*
P2L	ZSMNW035A	C	1–4[1]	*Poor*
P3L	ZSMEW023A	C	0.1–1[1, 2]	*Very poor*
P4L	ZSMEW023A	C	0.1–1[1, 2]	*Very poor*
P5H	ZSMNW025A	C	0.1–1[1, 3, 5]	*Very poor*
P6H	ZSMEW004A	B	0.1–1[1, 4]	*Very poor*
	ZSMEW028A	C	0.1–1[1, 5]	
P7H	ZSMNE012A	A		
	Del A		< 0.1[1, 4]	*Extremely poor*
	Del B		0.1–1[1, 4]	*Very poor*
P8H	ZSMNS017A	C	0.1–1[1, 4]	*Very poor*
P9L	ZSMNW035A	C	1–4[1]	*Poor*
P10H	ZSMEW004A	B	0.1–1[1, 4]	*Very poor*
P11H	ZSMNE018A	A	0.1–1[1, 2]	*Very poor*

1) /SKB 2005a/.

2) /Curtis et al. 2003ab/.

3) /Hultgren et al. 2004/.

4) /Markström and Erlström 1996/.

5) /Mattsson et al. 2004/.

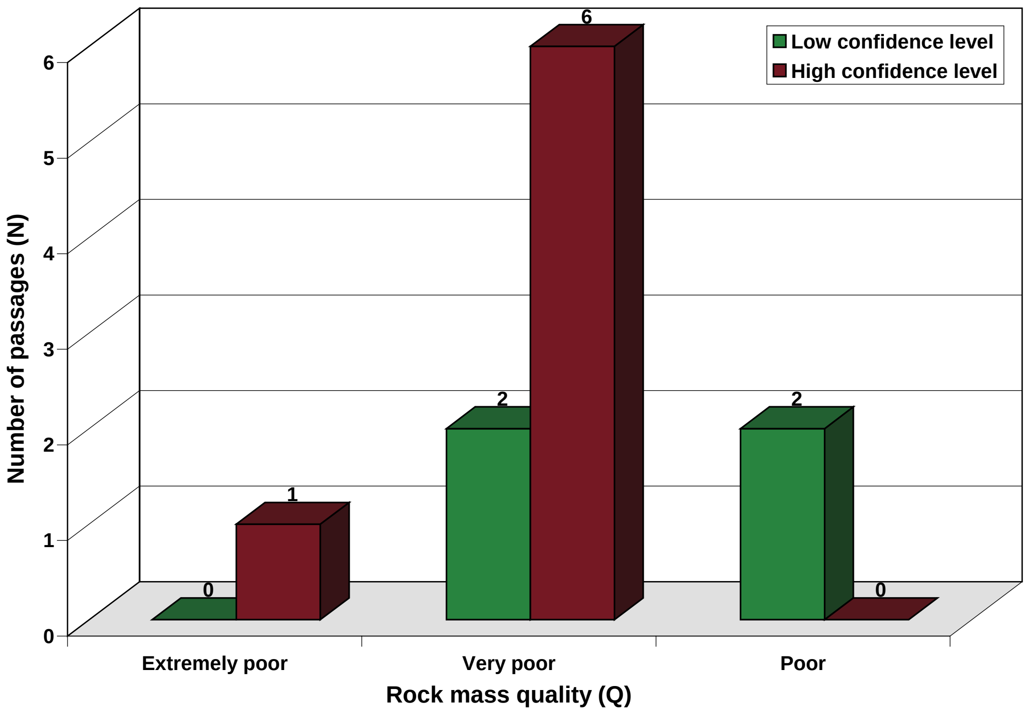


Figure 6-6. *Number of passages as a function of rock mass quality and deformation zone confidence level.*

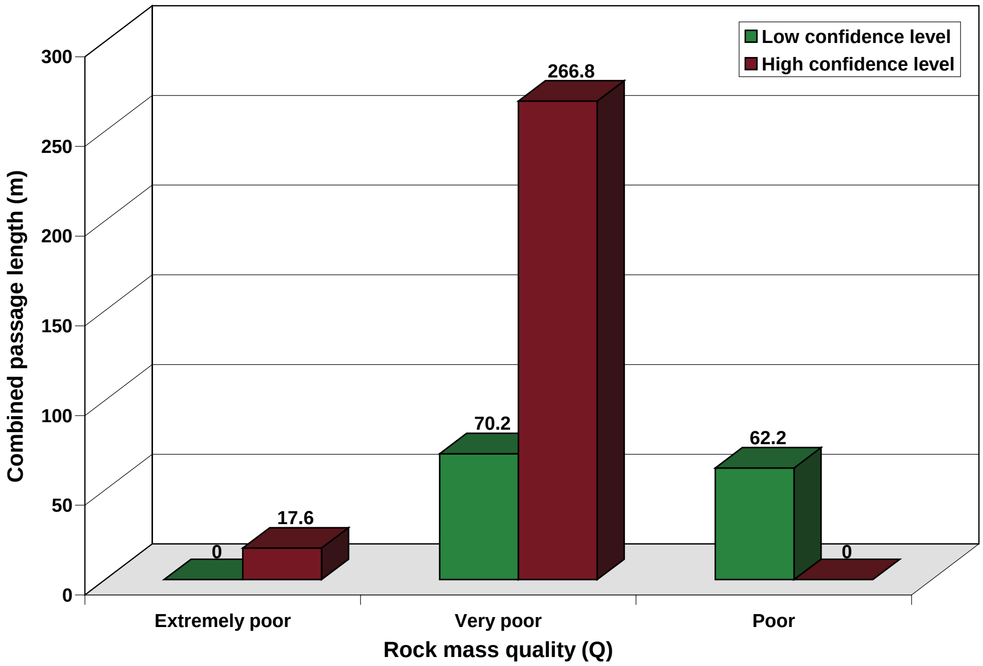


Figure 6-7. *Combined passage tunnel lengths as a function of rock mass quality and deformation zone confidence level.*

The assessment gives the following overall results:

Of the eleven passages, the rock mass quality is *extremely poor* (Q < 0.1) in one passage, *very poor* (Q = 0.1–1) in eight passages and *poor* (Q = 1–4) in two passages.

Of the combined passage tunnel length of 417 m, 132 m pass through "possible" zones and 285 m through high confidence zones.

Of the combined passage tunnel length of 417 m, 18 m are judged to in *extremely poor* rock, 337 m in *very poor* rock and 62 m in *poor* rock.

As previously mentioned, some of these lengths can be significantly reduced by modifying the interception angles of the tunnels to the deformation zones.

Much of the supporting data to the rock mass classifications is taken from the Äspö HRL access tunnel. During excavation the tunnel was mapped and classified according to Bieniawski's *Rock Mass Rating system*, RMR_{76} /Bieniawski 1976/. A compilation of these mapping results is presented in Figure 6-8 /Stanfors et al. 1997/.

The results show that the RMR values vary considerably within any particular deformation zone. Additionally, generally the worst rock mass quality mapped in most zones lies in the range of RMR_{76} = 20–40. It is possible to convert these RMR values to the Q index system, for the purpose of comparison, by applying the following relationship /Barton 2002/:

$$Q \approx e^{\frac{(RMR_{76}-44)}{9}} \qquad \text{Equation 6-2}$$

Application of this relationship converts the RMR_{76} = 20–40 grouping to a Q index in the range of Q = 0.1–1. Since the original mapped RMR values took into consideration the fracture orientation in relation to the access tunnel orientation, a general compensation has been made by increasing the RMR values by 5 prior to conversion.

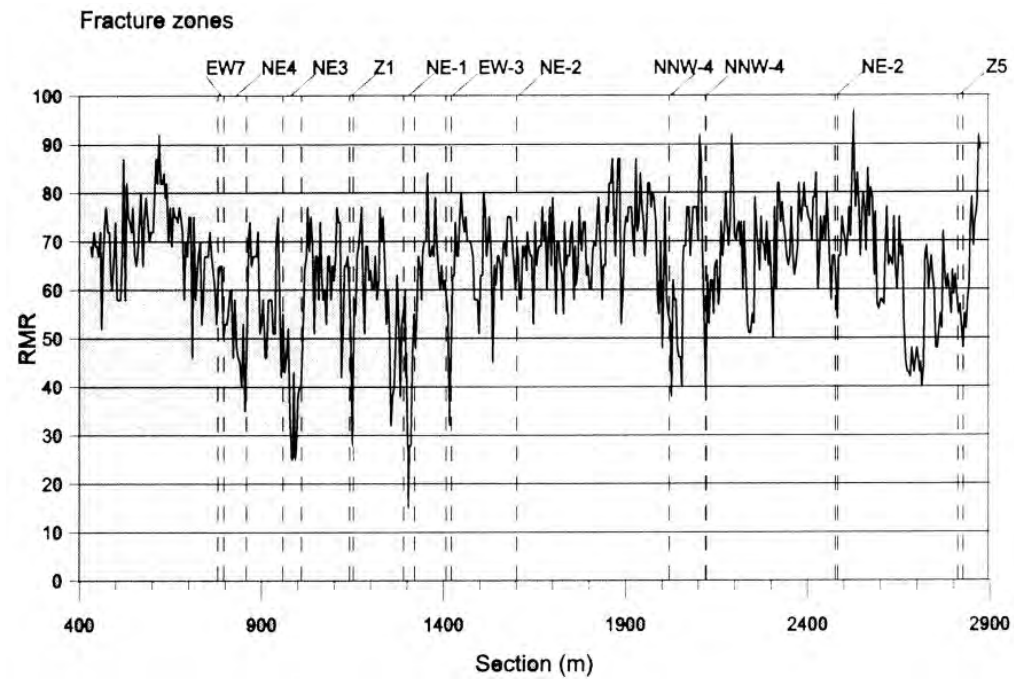


Figure 6-8. *Compilation of values for RMR_{76} mapped from the Äspö tunnel chanage 450–2,875 /Stanfors et al. 1997/.*

There is relatively good agreement between the assessed rock mass quality for the passages in the layout and the mapped rock quality in the access tunnel.

Classification of passages due to water inflows to the passages

The passages have been classified on the basis of the level of likely water inflows. The level of likely inflows has been assessed using Equation 6-3 /Alberts and Gustafson 1983/. The different classes are presented in Table 6-4.

$$q_s = \frac{2\pi K_b d}{\ln\left[\frac{2d}{r_w}\right] + \xi} \qquad \text{Equation 6-3}$$

where,

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

K_b = representative hydraulic conductivity of rock mass for analysed tunnel orientations (T/b), (m/s)

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

b = deformation zone hydraulic thickness (m)

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

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

ξ = deposition tunnel's natural skin factor (dimensionless)

Table 6-4. Classification of water inflow.

Inflow class	Potential inflow (l/min/10 m)
Low inflow	0–25
Medium inflow	25–250
High inflow	250–2,500
Very high inflow	> 2,500

The hydraulic conductivity of a deformation zone was based on the zone transmissivity and hydraulic thickness according to the site description S1.2. /SKB 2005a/. The tunnel depth below the groundwater table was set at –500 m. The tunnel equivalent radius was set at r_w = 3.84 m and the tunnel natural skin factor at $\xi = 5$ based on experience from Äspö HRL /Dalmalm 2001/.

A compilation of the hydraulic properties for the tunnel passages and water inflow potential, assuming no grouting measures are taken, is presented in Table 6-5. The results, taking into account zone confidence level, are presented graphically in Figure 6-9 and Figure 6-10.

The transmissivities presented in Table 6-5 are taken from the site description S1.2 with supplementary data taken from probe hole measurements, performed where certain zones were penetrated during the excavation of the Äspö HRL access tunnel /Markström and Erlström 1996/. The deformation zone hydraulic thickness is based on the estimated structural geological thickness as presented in the site description S1.2.

Table 6-5. Compilation of assessed hydraulic properties for the tunnel passages and potential water inflow, assuming no grouting.

Passage ID no	Deformation zone ID no	Transmissivity		Hydraulic thickness[1]	Potential inflow	
		T_A[1] (m²/s)	T_B[2]	b (m)	q_s[3] (l/min/10 m)	Class
P1H	ZSMNE018A	$2.9{\cdot}10^{-6}$		30	17.5	Low
P2L	ZSMNW035A	$1.3{\cdot}10^{-5}$		20	112.4	Medium
P3L	ZSMEW023A	$1.3{\cdot}10^{-5}$		20	112.4	Medium
P4L	ZSMEW023A	$1.3{\cdot}10^{-5}$		20	112.4	Medium
P5H	ZSMNW025A	$2.6{\cdot}10^{-7}$		5	9.3	Low
P6H	ZSMEW004A	$1.3{\cdot}10^{-5}$	$2.8{\cdot}10^{-6}$–$2.0{\cdot}10^{-7}$	30	75.0	Medium
	ZSMEW028A	$8.5{\cdot}10^{-8}$		10		
P7H	ZSMNE012A	$1.1{\cdot}10^{-4}$	$3.4{\cdot}10^{-4}$–$2.1{\cdot}10^{-5}$	41	461.4	High
P8H	ZSMNS017A	$6.5{\cdot}10^{-5}$	$1.5{\cdot}10^{-3}$–$2.8{\cdot}10^{-5}$	20	580.0	High
P9L	ZSMNW035A	$1.3{\cdot}10^{-5}$		20	112.4	Medium
P10H	ZSMEW004A	$1.3{\cdot}10^{-5}$	$2.8{\cdot}10^{-6}$–$2.0{\cdot}10^{-7}$	30	75.0	Medium
P11H	ZSMNE018A	$2.9{\cdot}10^{-6}$		30	17.5	Low

1) /SKB 2005a/.

2) /Markström and Erlström 1996/.

3) Value based on transmissivity in site description S1.2 /SKB 2005a/.

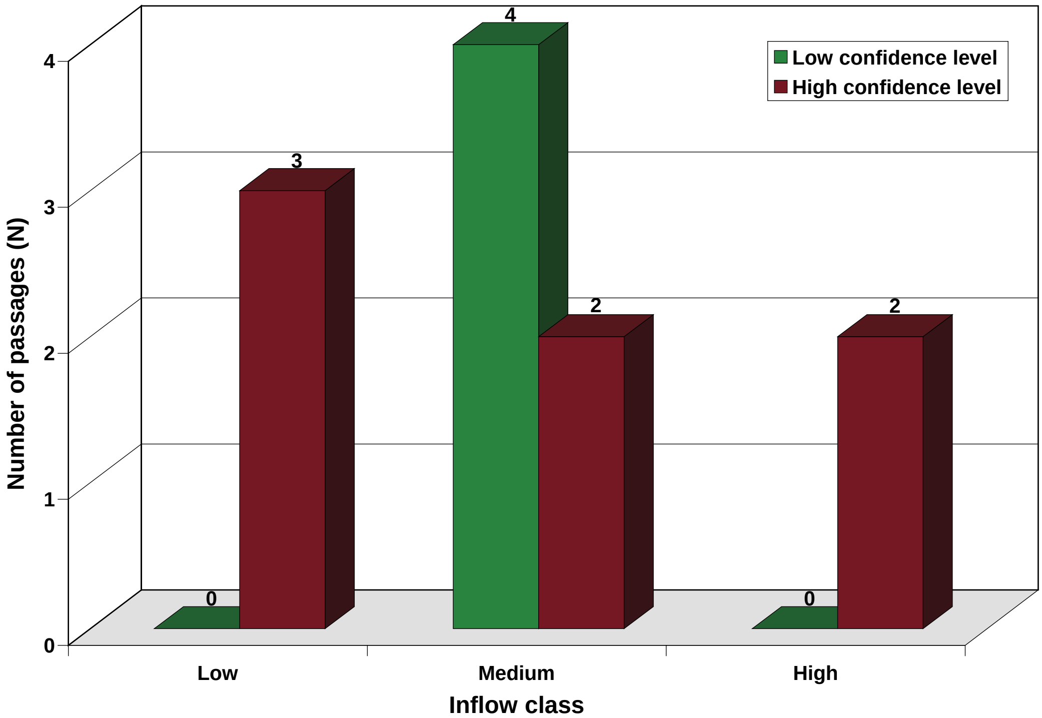


Figure 6-9. *The number of passages as a function of water inflow potential and zone confidence, assuming no grouting.*

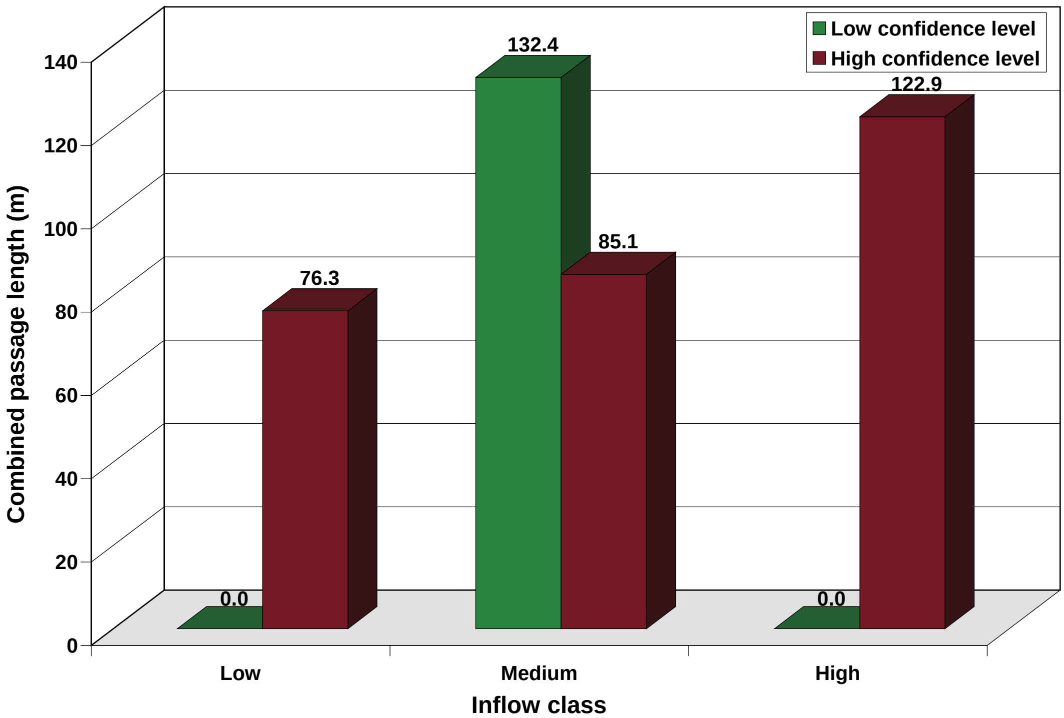


Figure 6-10. *Combined passage length as a function of potential water inflow class and zone confidence assuming no grouting.*

The results show that there are significant differences in potential water inflows between the various deformation zones. Assuming no grouting is carried out, two passages have potential for high inflows (~ 460–580 l/min/10 m), six passages have potential for medium inflows (~ 75–115 l/min/10 m) and three passages have potential for low inflows (~ 10–20 l/min/10 m).

Of the total passage tunnel length of 417 m, 132 m correspond to “possible” zones and 285 m to high confidence zones. 125 m of the total tunnel length are judged to have potential for high water inflows, 215 m potential for medium water inflows and 75 m potential for low water inflows. Optimisation of tunnel orientations to reduce passage length, in particular for passages P7H and P8H with potentially high water inflows, would clearly be advantageous.

A compilation of transmissivity values assessed for the various deformation zones encountered during the excavation of the Äspö HRL access tunnel are presented in Figure 6-11 /Rhén et al. 1997/. As the figure illustrates, the potential inflows vary with 2 or 3 orders of magnitude. Additionally, there are a number of zones with transmissivities in the order of $T = 10^{-3}$ m^2/s. This suggests that for isolated sections within deformation zones, there is clearly a risk of much higher water inflows than suggested by the calculated values based on the transmissivities presented in the site description S.1.2.

Potential problems and risks associated with the passages

Based on the information available for the differing deformation zones, potential problems and risks are associated with both stability and water inflow issues. It is considered that these types of problems will largely be confined to the excavation phase, however, some of them may be time dependent and be of longer-term significance.

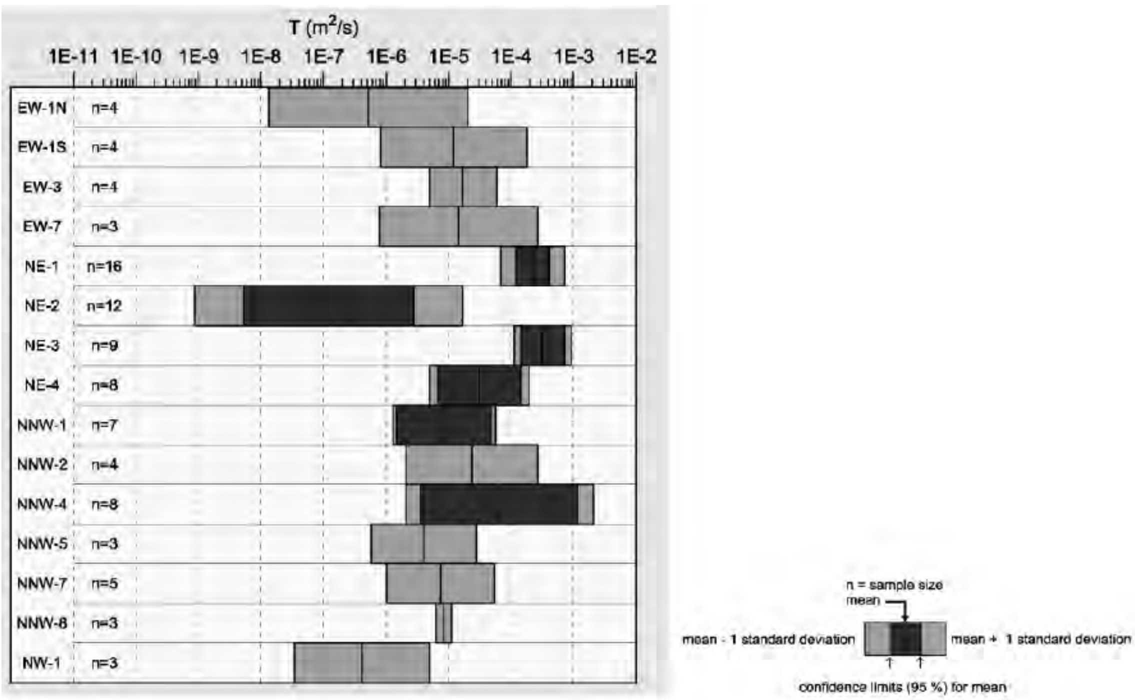


Figure 6-11. *Compilation of measured transmissivity values for deformation zones encountered during the excavation of Äspö HRL access tunnel /Rhén et al. 1997/.*

Deformation of the tunnel

For zones of highly fractured rock with clay dominated cores there is the risk for significant deformation during excavation. There is also the risk of time dependent plastic deformation since both in situ rock stress and surrounding water pressures are high. In such sections it is recommended that excavation is the carried out with short advance distances and consideration is given to subdividing excavation of the tunnel face. Forepoling techniques may be needed along with stabilization of the face with shotcrete between rounds.

Stability of the tunnel face

Results indicate that for certain zones both transmissivity and water pressure are high and there is clearly a risk for local instability of the tunnel face. Wherever there is a major difference in hydraulic conductivity between the deformation zone and the surrounding rock there is the risk that the water pressure gradient is high enough at the face to trigger a sudden collapse. Instability at the face may even take the form of erosion of the rock mass if the water, under high pressure, is allowed to flow into the tunnel. Both types of instability can be avoided if probing and effective grouting are carried out sufficiently far ahead of the advancing tunnel face.

Rock wedge failure

There is a high risk for rock wedge failure in the deformation zone passages. This is due not only to the presence of multiple fracture sets but also the existence of planar fracture surfaces, some with chlorite or clay fillings, along with the potential for high water pressure in the surrounding rock. Such wedge failures can be minimized by the use of forepoling and the prompt installation of temporary support.

Swelling clays

A number of deformation zones contain clay filled fractures, commonly in a local core area or associated with the interface with the bounding competent rock. The clay in such fractures can be expansive and give rise to time dependent deformations. The presence of the expansive clay mineral smectite is repeatedly documented /Curtis et al. 2003a/ as filling some of the fractures intercepted by excavations in the Simpevarp area. However, no significant failures or problems were linked to its presence. Consequently, it is assumed that the swelling pressures generated are only moderate or the smectite is present in insufficient quantities to generate noteworthy problems.

Water problems

Since water pressures at repository depth are high and certain deformation zones have high transmissivities there is a clear risk for large water inflows into the tunnel. In order to maintain a safe working environment and a stable tunnel geometry, it will be important to include sufficiently long probing and grouting holes ahead of the advancing tunnel face. Several alternative programs of supplementary grouting need to be prepared in advance, and when long probing holes indicate the severity of the zone, a decision for suitable program will be taken and activated to reduce inflows to stipulated levels. As recommended in /Chang et al. 2005/ probe drilling in niches 100–200 from the water-bearing zone is carried out to define the zone properties and location. Thereafter a second niche is excavated at 30–50 m from the zone for detailed probing and grouting. A third niche may be needed depending on the results of the grouting from the previous niche.

In cases such as P7H and P8H, where potential inflows are judged to be high to very high, the need for multiple grouting cycles is inevitable. Due to the high water pressure the grout design must also target even the finer fractures. If problems continue then the use of ground-freezing and a concrete lining should be considered.

In order to obtain a grasp of the degree of reduction in hydraulic conductivity that needs to be achieved, the post grouting inflows have been analysed with the help of Equation 6-4. /Alberts and Gustafson 1983/.

$$q_s = \frac{2\pi K_b d}{\ln\left[\frac{2d}{r_w}\right] + \left[\frac{K_b}{K_t} - 1\right] \cdot \ln\left[1 + \frac{m}{r_w}\right] + \sigma} \qquad \text{Equation 6-4}$$

where

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

K_b = representative hydraulic conductivity for the deformation zone (T/b), (m/s)

K_t = representative hydraulic conductivity for the grouted deformation zone (m/s)

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

b = deformation zone hydraulic thickness (m)

d = the tunnel depth below the groundwater table (m)

m = the grouted zone thickness (m)

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

σ = skin factor for the grouted zone (dimensionless).

The deformation zone hydraulic conductivity is calculated from the transmissivity and hydraulic thickness as presented in the site description S1.2 /SKB 2005a/. The tunnel depth below the groundwater table is set at d = 500 m. The tunnel radius is set at r_w = 3.84 m. The skin factor is set at σ = 5 based on experience from Äspö HRL /Dalmalm 2001/. The grouted part of the zone is assumed to extend for 10 m beyond the tunnel boundary and have a hydraulic conductivity in the range of 10^{-7}–10^{-8} m/s. The results are presented in Table 6-6.

The analysis shows that a zone requires grouting until a hydraulic conductivity of approximately 10^{-8} m/s is obtained, in order to lower inflows to the tunnel to acceptable levels. It is judged that such a reduction in hydraulic conductivity using cement-based grouts is achievable but will require the inclusion of systematic grouting ahead of the tunnel face and a high standard of professionalism in the performance of the work.

As well as problems associated with water inflows during the excavation phase, there is also the possibility of time dependency issues. For example, inflows can increase with time due to erosion of natural and partially grouted joint fillings.

Table 6-6. Calculated water inflows in the passages after grouting.

Inflow class	Passage ID no	Inflow after grouting (l/min/10 m)		
		K_t 10^{-7} (m/s)	K_t 5×10^{-8} (m/s)	K_t 10^{-8} (m/s)
Low	P1H, P5H, P11H	10–20	10–15	5–10
Medium	P2L, P3L, P4L, P6H, P9L, P10H	55 –70	40 –50	10 –15
High	P7H, P8H	115–120	65	15

Rock support and grouting actions in the passages

The following section gives proposals for rock support and grouting activities with reference to the passage of the various deformation zones. The deformation zones are grouped according to the rock mass classification used in Section 5. The rock support proposals at the excavation front and behind the face are presented in Table 6-7 and Table 6-8. The grouting proposals are presented in Table 6-9.

The rock support proposals are largely based on recommendations associated with the Q-system /Grimstad and Barton 1993/. The solutions follow a standard methodology of reducing the tunnel advance distance along with the installation of rock bolts and shotcrete.

Table 6-7. Proposed excavation cycle and rock support at the tunnel face during passage of a deformation zone.

Rock mass class (Q-index)	Passage ID no	Proposed excavation cycle and rock support
Poor 1–4	P2L, P9L	Reduce tunnel advance distance to 3 m rounds. Rock support to be installed after alternate rounds.
Very poor 0.1–1	P1H, P3L, P4L, P6H, P7HB, P8H, P10H, P11H	Reduce tunnel advance distance to 2–3 m rounds. Rock support to be installed after each round.
Extremely poor dålig < 0.1	P7HA	Forepoling prior to tunnel advance. Reduce tunnel advance distance to 2 m. Immediate temporary shotcrete support of the tunnel roof prior to completion of mucking out.

Table 6-8. Proposed rock support behind the face during passage of a deformation zone.

Rock mass class (Q-index)	Passage ID no	Proposed rock support
Poor 1–4	P2L, P9L	From: Fibre-reinforced shotcrete ~ 50 mm, bolt c1.5 m L 3.0 m To: Shotcrete ~ 30 mm, bolt c2.0 m L 3.0 m
Very poor 0.1–1	P1H, P3L, P4L, P6H, P7HB, P8H, P10H, P11H	From: Fibre-reinforced shotcrete ~ 150 mm, bolt c1.0 L 3.0 m To: Fibre-reinforced shotcrete ~ 50 mm, bolt c1.5 L 3.0 m
Extremely poor < 0.1	P7HA	From: Fibre-reinforced shotcrete ~ 250 mm, bolt c1.0 L 3.0 m To: Fibre-reinforced shotcrete ~ 150 mm, bolt c1.0 L 3.0 m

Table 6-9. Proposed grouting action during passage of a deformation zone.

Water inflow class	Passage ID no	Proposed grouting action
Low	P1H, P5H, P11H	Cement based pre-grouting ahead of the face with control holes and a secondary grout-hole sequence if needed. End-of- hole spacing 3 m, hole length 20 m, over-lap 5–10 m.
Medium	P2L, P3L, P4L, P6H, P9L, P10H	Cement based pre-grouting with control holes with possibly secondary and tertiary grout-hole sequences if needed. End-of-hole spacing 3 m, hole length 20 m, overlap 5–10 m.
High	P7H, P8H	Cement based pre-grouting in the tunnel face and additional niches with long holes that penetrate the entire zone. Definition of a grout design with inner and outer curtain. Secondary and tertiary grout-hole sequences if needed. Access to special safety drilling equipment required. Access to freezing equipment required and equipment for follow-up placement of a concrete lining.

The proposed grouting activities are focused on a strict program of probing, grouting and control holes of sufficient length, using cement-based grouts. Freezing and the installation of a local concrete lining is proposed as an alternative method, for passages with potentially high water inflows and very to extremely poor rock conditions.

6.4 Discussion and conclusions

For this design task information has been taken from the site description S1.2 and from documentation covering excavation of Äspö HRL and OKG tunnels. However, if the Simpevarp subarea continues to be of interest for the repository site it is considered that more useful information can be extracted from Äspö and OKG records.

The proposed repository layout involves eleven passages through deformation zones. Seven of these deformation zones are classed as high confidence and four as "possible". The total tunnel length of passages is approximately 415 m. Individual passage lengths vary from 10 to 70 m. In certain cases there is a clear opportunity to reduce these passage lengths by optimising the tunnel orientation as they approach a deformation zone.

The overall layout allows for a continuous transport loop in the deposition area. However, to create flexible transport system with reasonable transport times it will be necessary to include some additional local transport loops in the detailed design. These additional transport loops may require one or two further tunnel passages.

The rock mass quality according to the Q system /Barton 2002/ is judged *extremely poor* for one passage, *very poor* for eight passages and *poor* in two passages. Of the total passage tunnel length approximately 20 m are judged to pass extremely poor rock, approximately 335 m very poor rock and approximately 60 m poor rock.

There is a risk of potentially *high* water inflows for two of the passages, *medium* water inflows in six passages and *low* water inflows in three passages. Of the total passage tunnel length, approximately 125 m have a risk for *high* water inflows, 215 m risk for *medium* water inflows and approximately 75 m risk for *low* water inflows.

It should be noted that since the geological properties of any particular deformation zone are assumed to be uniform and not vary with depth, the length of the passages most probably are overestimated. Similarly the properties presented for the identified passages are clearly considered to be somewhat conservative concerning both rock mass quality and hydraulic properties.

The problems and risk associated with the passage of the deformation zones are related to both stability and water inflow problems. The potential stability problems involve the risk of large deformations in certain tunnel sections due to the presence of clay and highly fractured rock; risk for instability at the tunnel face due to very poor rock conditions with high water pressure gradients and the risk for wedge failures in the roof and walls. The potential water inflows to the tunnel need to be investigated by probe holes of sufficient number and length, along with a systematic but flexible grouting program.

The proposed rock support is to a large degree based on recommendations from the Q system. The solutions follow a standard methodology of reducing the tunnel advance distance along with the installation of rock bolts and shotcrete. The proposed grouting activities are focused on a strict program of probing, grouting and control holes of sufficient length using cement-based grouts. Freezing and the installation of a local concrete lining is proposed as an alternative method for passages with potentially high water inflows and very to extremely poor rock conditions /Chang et al. 2005/.

7 Seepage and hydrogeological situation around the repository

7.1 Input data and assumptions

The work involved has been carried out in general accordance with Design Task G, Section 5.8 of UDP /SKB 2004a/. Deviations from UDP guidelines are presented in Section 7.2 below. The study has involved both analytical and numerical approaches. SKB has performed numerical analyses using the approved computing tool, DarcyTools v 3.0, and the presentation in this report is a summary of the work /Svensson 2005/. Hydrogeological input data has been sourced from the Simpevarp site description v 1.2 /SKB 2005a/.

The geometrical framework for the analysis is provided by the distribution of the deformation zones, shown in Figure 6-1, in combination with the proposed repository layout, shown in Figure 5-1, set at SKB's reference depth of –500 m.

Seepage into the repository, the surrounding hydrogeological situation with respect to salinity (TDS) and lowering of the groundwater table, was assessed on the basis of assumed grouting levels. The purpose of the assessments is to assist with the preliminary safety evaluation, environmental impact assessment and help assess pumping and water treatment requirements.

Seepage into the repository is dependent on how much effective rock grouting is performed. Since no specific requirements are formulated for design step D1 /SKB 2004a/, the grouting need cannot be assessed on the basis of acceptable seepage level requirements. Instead the strategy stipulated is to assess the hydrogeological situation on the basis of various assumptions of achieved grouting results (resulting conductivity) at different points in time (phases of excavation).

7.2 Execution

The study of seepage and the hydrogeological situation around the repository involves three main areas of concern:

- quantity of seepage into the repository,
- depression of the groundwater table,
- variation of salt content in the rock volume in close proximity to the canisters.

The assessment of the above issues was considered for the different phases of construction and for different grouting levels:

- level 0: no grouting,
- level 1: resulting conductivity of $K = 10^{-7}$ m/s in the defined grouting zone,
- level 2: resulting conductivity $K = 10^{-9}$ m/s in the defined grouting zone.

The number and geometry of the deformation zones, coupled to the interaction with the proposed design layout, results in a complex hydrogeological situation. It can be expected that a number of deformation zones have hydraulic contact with the sea. The deformation zones create a well developed hydraulic anisotropy in the rock volume and will dominate possible water transport through the rock mass.

The hydrogeological model from the Simpevarp site description v 1.2 /SKB 2005a/ formed the basis of the numerical analysis and thus the included anisotropy and inhomogeneity were taken into account. It consists of deformation zones, termed Hydraulic conductor domains (HCD) and the rock mass outside these zones, termed Hydraulic rock domains (HRD). The overlying soil layer constitutes the Hydraulic soil domain (HSD).

To describe the hydrogeological situation for the entire repository and its surroundings with analytical analysis necessitate a high degree of simplification of the hydrogeological situation and the repository geometry. However, analytical calculations are suitable for gross estimates of the total seepage, the study of certain parts of the repository and the effectiveness of different grouting scenarios.

It is primarily numerical analysis that has been used to assess the seepage, groundwater depression and variation of salt content in the rock volume for different grouting levels, while analytical assessments have been used for the passage of deformation zones and study of grouting effectiveness. The numerical code has been verified by a number of idealized hydrogeological situations for which analytical solutions are available.

The approach outlined above involves a development of the methodology presented in UDP and is considered more applicable for the current early design step.

7.2.1 Analytical calculations

As previously stated, the assessment of the hydrogeological situation around the repository is largely described by the numerical analysis. However, complimentary analytical calculations and Monte Carlo simulations have also been performed. These were primarily focused on the grouting levels for the tunnels, which could not be satisfactorily handled by the selected numerical modelling tool.

Seepage to the tunnels

Seepage to the tunnels, q_s, was calculated according to Equation 7-1 /SKB 2004a/.

$$q_s = \frac{2 \cdot \pi \cdot K_b \cdot d}{\ln\left[\frac{2 \cdot d}{r_w}\right] + \left[\frac{K_b}{K_t} - 1\right] \ln\left[1 + \frac{m}{r_w}\right] + \sigma} \qquad (m^3/s,m) \qquad \text{Equation 7-1}$$

where,

K_b = representative hydraulic conductivity of the rock mass (m/s). Log normal distribution, mean $\log^{10}(K_b) = -8.8$ and standard deviation $\log^{10}(K_b) = 1$ (m/s)

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

d = deposition tunnel's centre depth below the groundwater table (m)

r_w = deep repository's representative radius, tunnel radius = $[A_{tunnel}/(\pi)]^{0.5}$ (m)

m = thickness of grouting (m)

σ = skin factor inside grouting, tunnel skin factor. Rectangular distribution (min/max = 3/7) (dimensionless).

With reference to the length of each respective tunnel type, the total seepage is calculated according to Equation 7-2.

$Q_S = q_S \times L$ (m^3/s) Equation 7-2

where,

L = tunnel length for each respective tunnel type (m)

Equation 7-1 does not take into account any reduction in the groundwater pressure and therefore is judged to result in an overestimation of seepage. However, since the lateral extent of the depression is normally limited, the changes in groundwater level are expected to have only a minor effect on the calculated total seepage to the tunnels.

The hydraulic conductivity is different for the three different grouting levels set in accordance with UDP:

- Grouting level 0: No grouting with $K_b = K_{t0} = 1.6 \times 10^{-9}$ m/s /SKB 2005a/ for rock mass (HRD). K_b values for the deformation zones are presented in Table 7-1.
- Grouting level 1: rock mass is sealed to a resulting conductivity of $K_{t1} = 10^{-7}$ m/s.
- Grouting level 2: rock mass is sealed to a resulting conductivity of $K_{t2} = 10^{-9}$ m/s.

In the Simpevarp area, the hydraulic conductivity of the rock mass outside the deformation zones is lower than that prescribed by UDP's Grouting level 1 and therefore seepage into the HRD tunnels was not analysed. The calculated seepage levels for the deposition, transport and main tunnels, at a point in time when the highest number of tunnels are judged to be open, are presented in Table 7-1. Results are presented in Section 7.3.1 of this report.

7.2.2 Numerical simulations

The hydrogeological model utilized in the analysis is identical to that presented in the Simpevarp site description v 1.2. It consists of deformation zones (HCD), the rock mass outside these zones (HRD) and the overlying soil (HSD).

The numerical analysis carried out by /Svensson 2005/ using DarcyTools is based on a hydrologeogical model of HRD consisting of a single DFN.

The shortest fracture generated in the DFN was set at 100 m. Additional fracture networks, with a shorter cut-off length have been generated and are presented in Section 7.3.2. as complementary analyses.

The tunnel layout used for the analyses is the –500 m reference level layout presented in Section 5 of this report.

The initial analysis involved a very conservative approach with all 213 deposition tunnels and 6,600 deposition holes open at the same time. A complementary study with the exclusion of all the deposition holes was also performed.

For a general description of the basic assumptions and mathematical formulation of DarcyTools, the reader is referred to /Svensson et al. 2005/. In this section some problem specific settings and assumptions will be discussed.

DarcyTools version 3.0 is used for the study. This is the first major project using v 3.0 and the project thus also serves as an evaluation of v 3.0.

7.3 Results

7.3.1 Analytical calculations

The calculated seepage levels for the deposition, transport and main tunnels, at a point in time when the highest number of tunnels are judged to be open, are presented in Table 7-1.

The relatively low seepage levels in the rock mass outside the deformation zones are notably different from those associated with the passage of the deformation zones for grouting levels 0 and 1.

Table 7-1. Predicted seepage into the deposition, transport and main tunnels.

Facility part	Total length (m)	Seepage[2] (l/s)		
Grouting level		**0**	**1[3]**	**2**
Deposition tunnel, 20[1] nos.	5,100	2.7	–	2.5
Main tunnel	5,220	2.9	–	2.8
Transport tunnel excluding zone passages	4,900	2.7	–	2.5
Transport tunnel zone passages	417	142	49	1.0

1) Number of deposition tunnels judged to be open. Deposition holes not included.

2) Seepage into zone passages has been calculated analytically without Monte Carlo simulations. For all other cases the 50%-percentile has been specified.

3) K_b < grouting level 1 for the rock mass.

An analytical assessment of the seepage into the repository was performed for a case where all of the deposition tunnels and holes were considered open, as assumed in the numerical analysis. The resulting seepage level is judged to be an overestimate since it assumes that seepage to the deposition holes does not affect seepage to the deposition tunnels. For a skin factor of $\xi = 0$, the resulting seepage estimates for grouting level 0 are as follows:

- tunnels excluding deformation zone passages approximately 55 l/s,
- tunnel passages through deformation zones approximately 270 l/s,
- deposition holes (6,600) approximately 40 l/s,
- total seepage approximately 365 l/s .

Transport tunnel passage of deformation zones

It can be expected that such passages will be subject to much higher seepage levels than those experienced in the intervening rock mass outside the deformation zones. The deformation zones are intercepted by the transport tunnels at 11 locations, according to the proposed layout, see Figure 6-1. The 11 passages involve a total tunnel length of 417 m, see Table 7-2. A skin factor of 5 has been used in the calculations in accordance with experience from Äspö HRL /Dalmalm 2001/.

Table 7-2. Prediction of seepage into the transport tunnels at deformation zone intersections.

Passage no H-high conf. L - conf. "possible"	T (m^2/s)	Width (m)	K_b (m/s)	Passage length (m)	Grouting level 0 Q_s (l/min)	 Q_s (l/s)	Grouting level 1 Q_s (l/min)	 Q_s (l/s)	Grouting level 2 Q_s (l/min)	 Q_s (l/s)
P1H	2.9E–06	30	9.8E–08	37	65	1.1	65	1.1	5	0.1
P2L	1.3E–05	20	6.3E–07	22	245	4.1	149	2.5	3	0.1
P3L	1.3E–05	20	6.3E–07	47	528	8.8	321	5.4	7	0.1
P4L	1.3E–05	20	6.3E–07	23	260	4.3	158	2.6	3	0.1
P5H	2.6E–07	5	5.2E–08	7	6	0.1	6	0.1	1	0.0
P6H	1.3E–05	30	4.2E–07	34	252	4.2	182	3.0	5	0.1
P7H	1.1E–04	41	2.6E–06	72	3,340	55.7	831	13.8	11	0.2
P8H	6.5E–05	20	3.3E–06	51	2,929	48.8	606	10.1	7	0.1
P9L	1.3E–05	20	6.3E–07	40	454	7.6	276	4.6	6	0.1
P10H	1.3E–05	30	4.2E–07	51	386	6.4	278	4.6	7	0.1
P11H	2.9E–06	30	9.8E–08	32	56	0.9	56	0.9	4	0.1
TOTAL				417	8,523	142.0	2,929	48.8	60	1.0

7.3.2 Numerical simulations

The following result is a summary of the study presented in reference /Svensson 2005/.

The proposed repository layout, along with the intercepted fractures from a single DFN simulation is presented Figure 7-1.

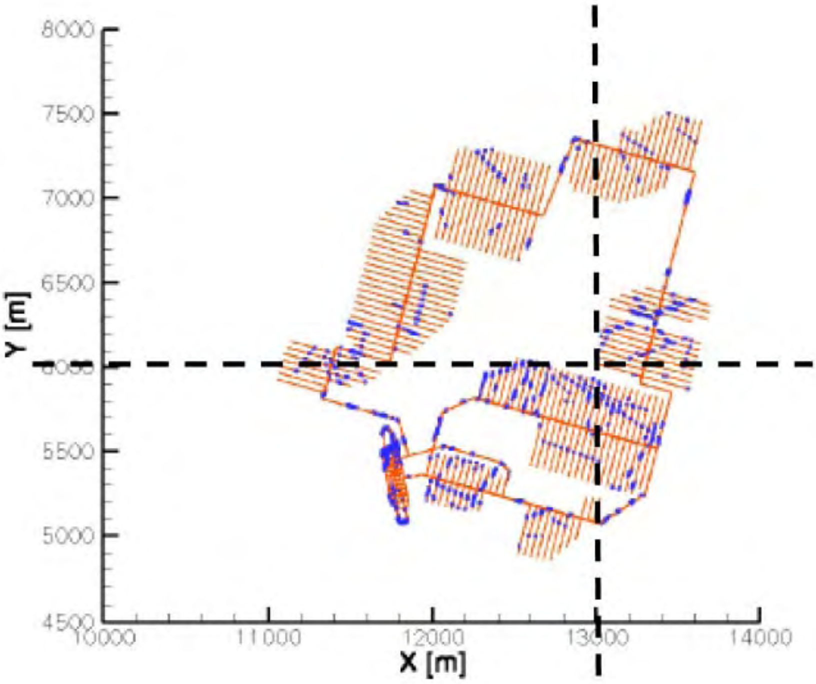


Figure 7-1. Results from a single simulation of the proposed repository layout, –500 m level. Blue represents intercepts between the layout geometry and the generated fractures. Dashed lines show vertical section positions.

The simulation is based on the repository having been in operation for 40 years. It is assumed that the entire repository is open, including the central area, 65 km of tunnels and 6,600 deposition holes. A complementary analysis was run with the deposition holes being omitted.

DarcyTools takes into account the existence of the individual fractures with hydraulic conductivity being dependent on fracture length. In order to simulate different grouting levels DarcyTools allows the application of different grouting factors ("*skin factor*"- in DarcyTools terminology) for where the fractures intercept the tunnel geometry. The distribution of hydraulic conductivity for the calculation cells involving intercepts with the tunnel geometry is presented in Figure 7-2.

Seepage

The total seepage levels presented in Table 7-3 assume steady-state flow conditions and that the entire repository is completely open and has been in operation for 40 years.

The quantity of predicted seepage changes significantly with the application of different grouting factors. A reduction in the applied grouting factor from 1.0 to 0.001 leads to a decrease in total seepage from 10,400,000 to 220,750 m³/year.

Table 7-3. Predicted seepage levels for the repository.

Grouting factor	Seepage (l/s)
1.0[1]	330
0.1	180
0.01	50
0.001	7

1) No grouting.

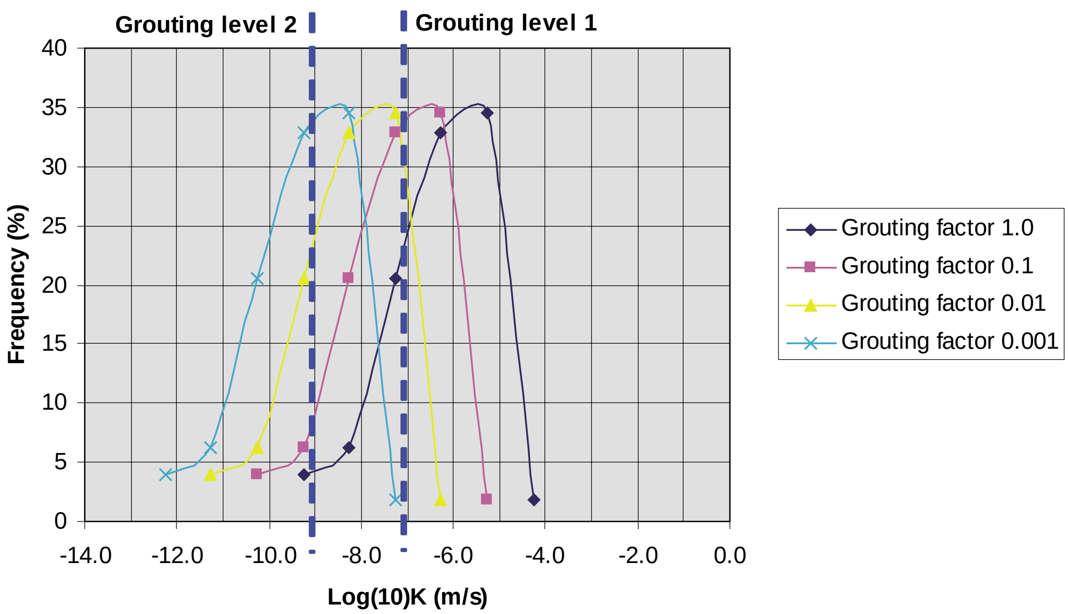


Figure 7-2. *Distribution of hydraulic conductivity for different grouting factors involving calculation cells that intercept the tunnel geometry.*

The seepage into the repository has an uneven spatial distribution and is largely concentrated at the deterministically determined deformation zones and associated discrete fractures. An example of a single simulation is presented in Figure 7-3, where the major flow is into transport tunnels at passages of deterministically determined deformation zones. The figure is representative only of the northern part of the repository and illustrates a horizontal section through the deposition areas. The flow is represented by vectors.

It should be noted that the location of the discrete fractures in Figure 7-3 is only an example from a single simulation and fracture position change with each simulation.

Groundwater table drawdown

The area subject to drawdown with an applied grouting factor of 0.1 is presented in Figure 7-4. The figure illustrates a horizontal section taken at level –1 m, with red indicating a groundwater level that is at or above this elevation and blue represents the drawdown. Figure 7-5 illustrates a similar horizontal section taken at level –100 m and shows where the groundwater table falls below level –100 m in two areas, over the central area and to the northeast of the central area.

The relatively limited lateral extent of the depression is illustrated by the vertical sections presented in Figures 7-6 and 7-7. The maximum depth of the depressions shown is approximately level –200 m.

It should be noted that the relatively high inflow to the transport tunnels associated with the intersection of deformation zone ZSMNS017A, indicated in Figure 7-3, does not generate a significant depression according to Figure 7-7 above. The results indicate that drawdowns associated with tunnels intercepting zones with higher hydraulic conductivities are of limited lateral extent.

A grouting factor of 0.01 results in no significant drawdown, see Figure 7-8. As shown in Figure 7-9 a grouting factor of 0.1 results in a limited spread of atmospheric pressure beyond the individual deposition holes. A grouting factor of 0.001 results in atmospheric pressure being confined to the individual deposition holes.

Total salinity (TDS)

The distribution of salts will be affected by the seepage pattern into the repository. The undisturbed salinity distribution, consisting of an increase of salinity with depth, is shown in Figure 7-11. Seepage into the repository will be from all directions and include drawing in salt water from deeper levels. The salinity distributions resulting from grouting factors of 0.1 to 0.001 are presented in Figures 7-12 to 7-14.

Complementary analyses

Exclusion of deposition holes

A complementary analysis with the exclusion of the deposition hole geometries has been performed for a grouting factor of 0.1.

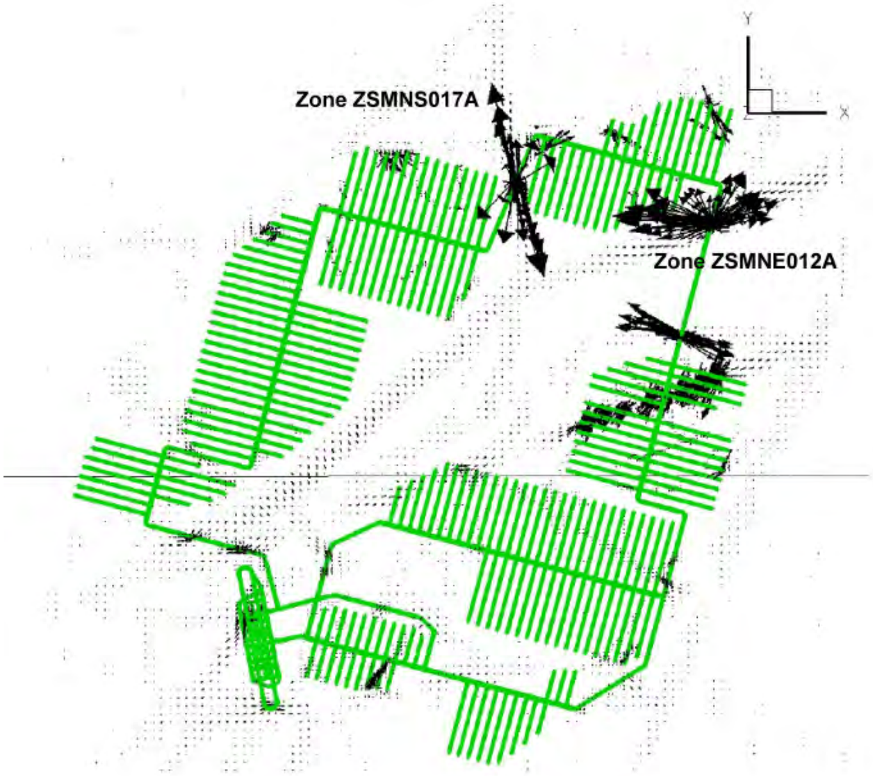


__Figure 7-3.__ Seepage into the repository at –515 m depth (representative depth for the repository's northern part). Seepage into the tunnels is concentrated at the deformation zone intercepts.

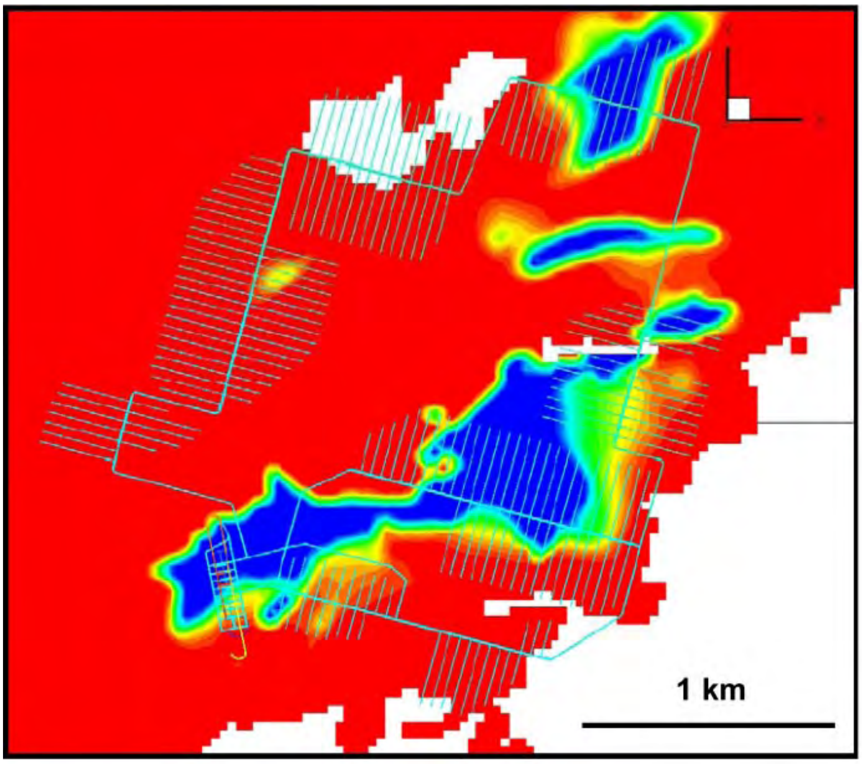


__Figure 7-4.__ Groundwater table at a grouting factor of 0.1. Horizontal section at level –1 m. Red represents where the groundwater table is at or above level –1 m. Blue represents the drawdown.

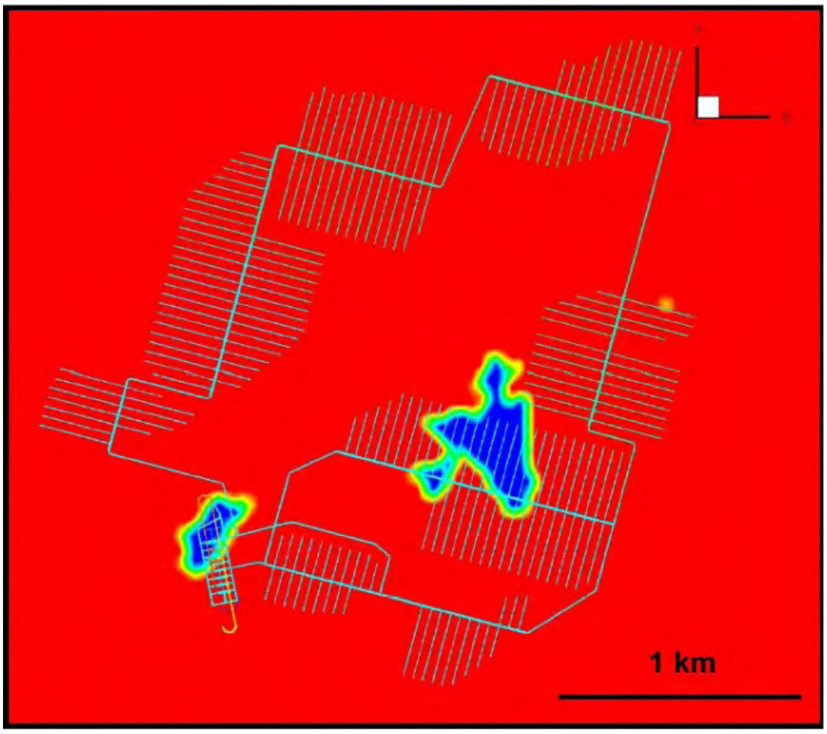


Figure 7-5. *Groundwater level for grouting factor of 0.1. Horizontal section at level –100 m. Red represents where the groundwater table is at or above level –100 m. Blue represents the drawdown.*

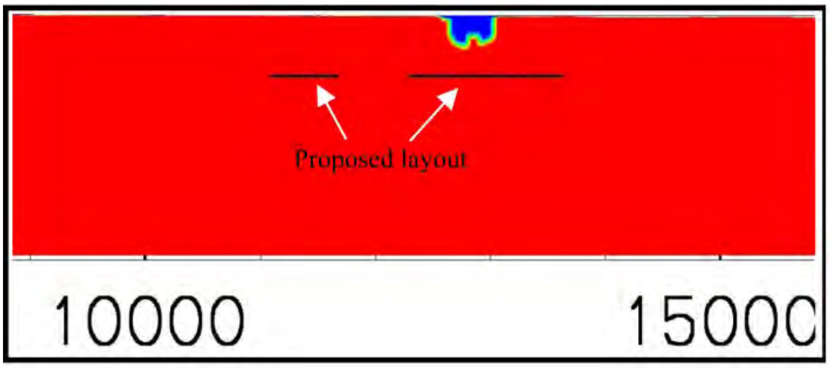


Figure 7-6. *Vertical section W-E showing the groundwater table drawdown for a grouting factor of 0.1. Maximum drawdown approximately level –250 m.*

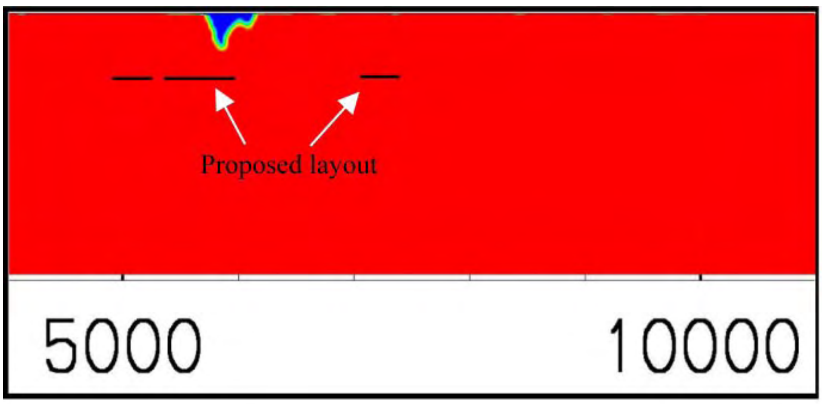


Figure 7-7. *Vertical section S-N showing the groundwater table drawdown for a grouting factor of 0.1. Maximum drawdown approximately level –250 m*

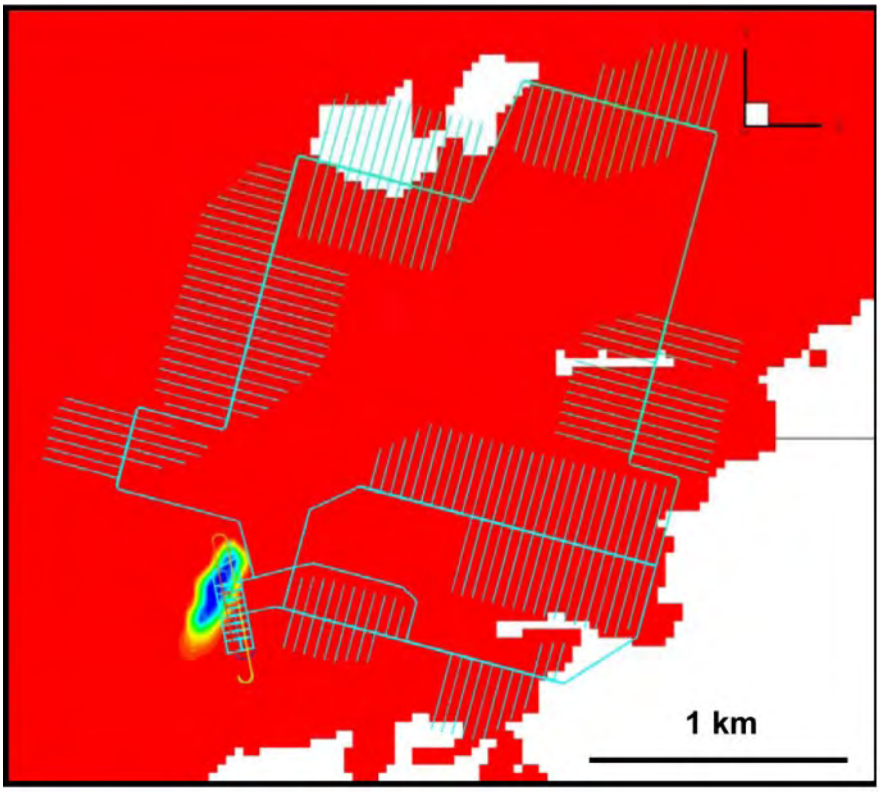


Figure 7-8. *Groundwater level for a grouting factor of 0.01. Horizontal section at level –1 m. Red represents where the groundwater table is at or above level –1 m.*

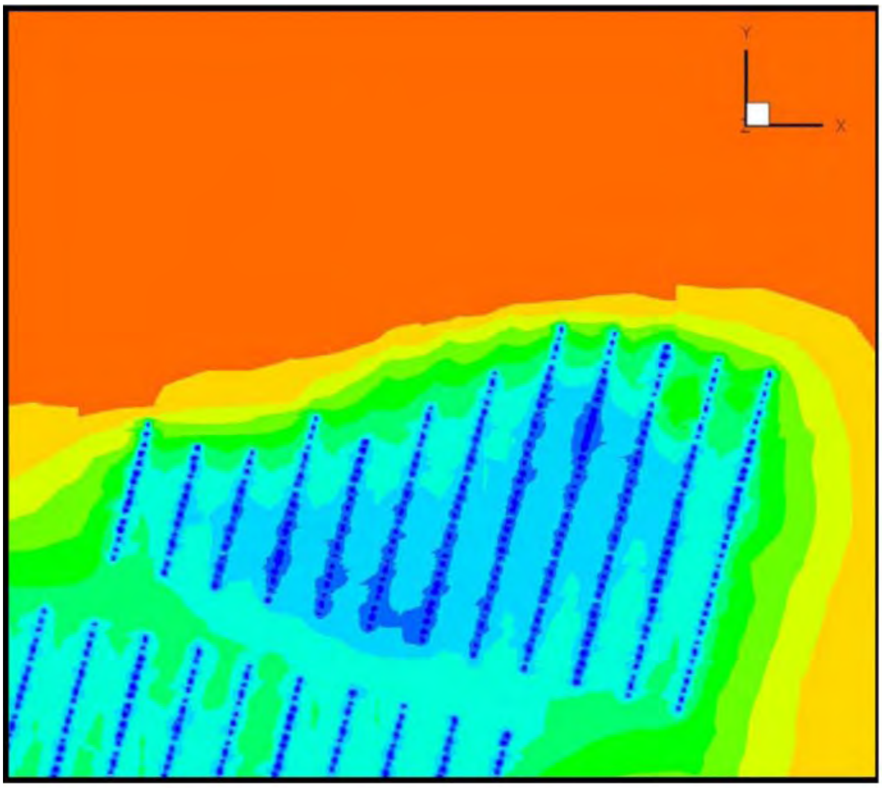


Figure 7-9. *Pressure distribution around the deposition holes for a grouting factor of 0.1. Dark blue represents atmospheric pressure. The figure shows the NE part of the repository at level –515 m.*

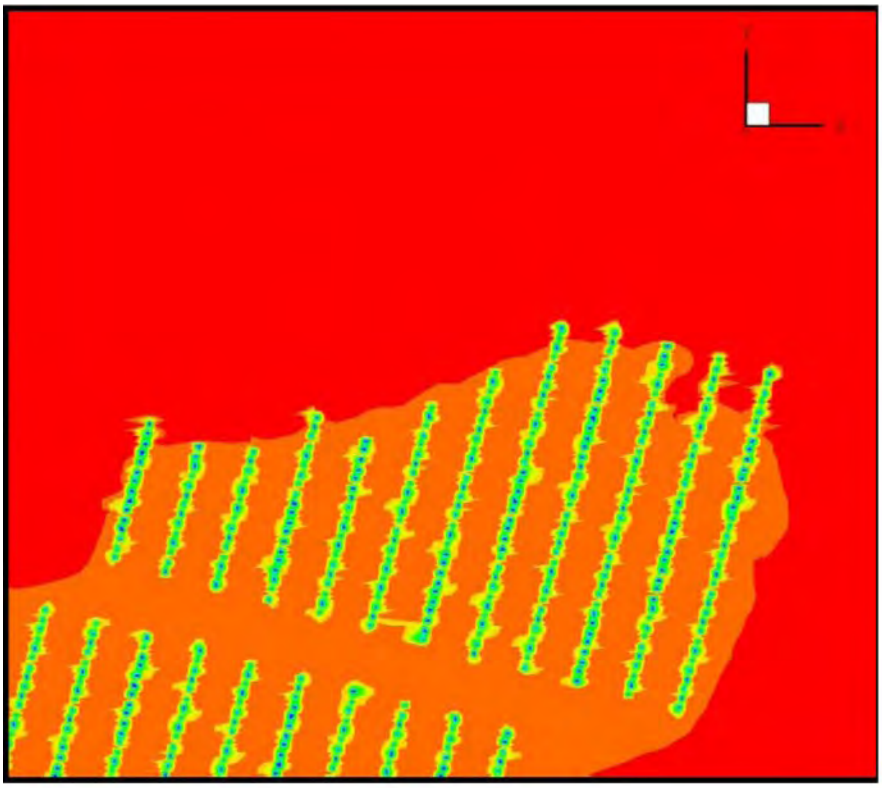


Figure 7-10. *Pressure distribution around the deposition holes for a grouting factor of 0.001. Dark blue represents atmospheric pressure. The figure shows the NE part of the repository at level –515 m.*

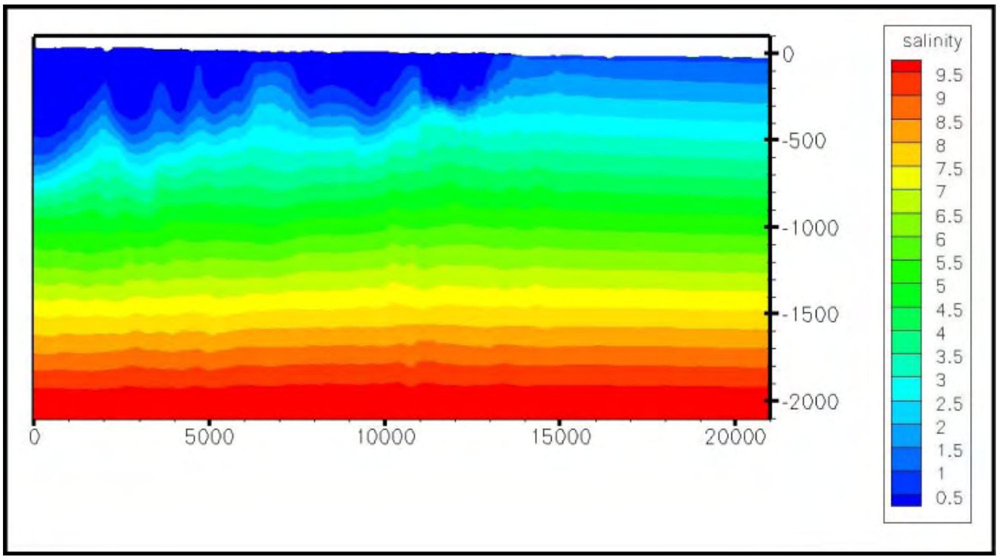


Figure 7-11. *Salinity distribution in the groundwater- undisturbed natural conditions. Vertical profile W-E. Salinity, % Total Dissolved Solids (TDS).*

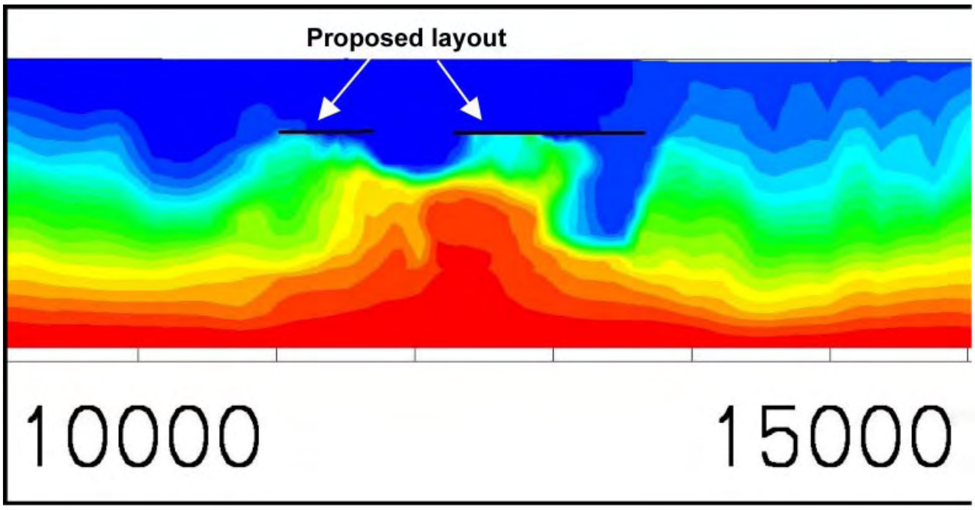


Figure 7-12. *Salinity distribution in the groundwater- grouting factor 0.1. Vertical profile W-E. Salinity, % Total Dissolved Solids (TDS).*

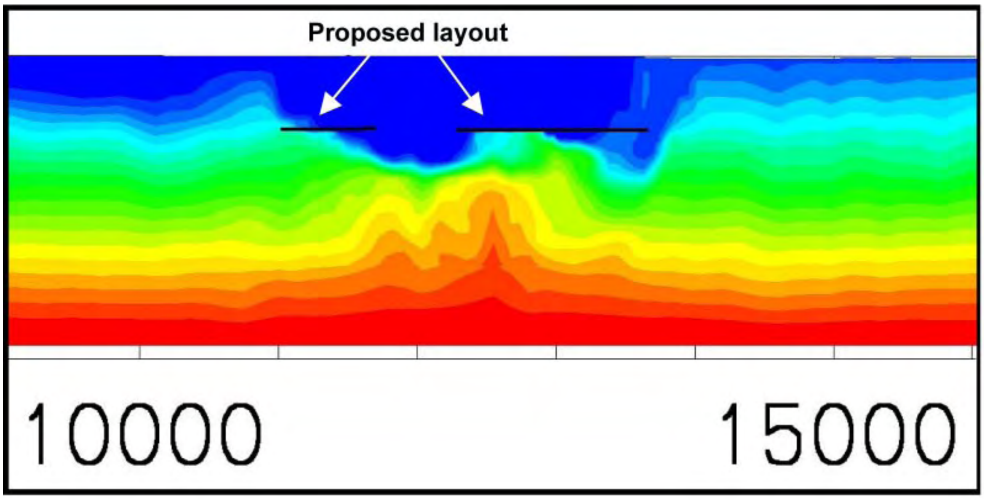


Figure 7-13. *Salinity distribution in the groundwater- grouting factor 0.01. Vertical profile W-E. Salinity, % Total Dissolved Solids (TDS).*

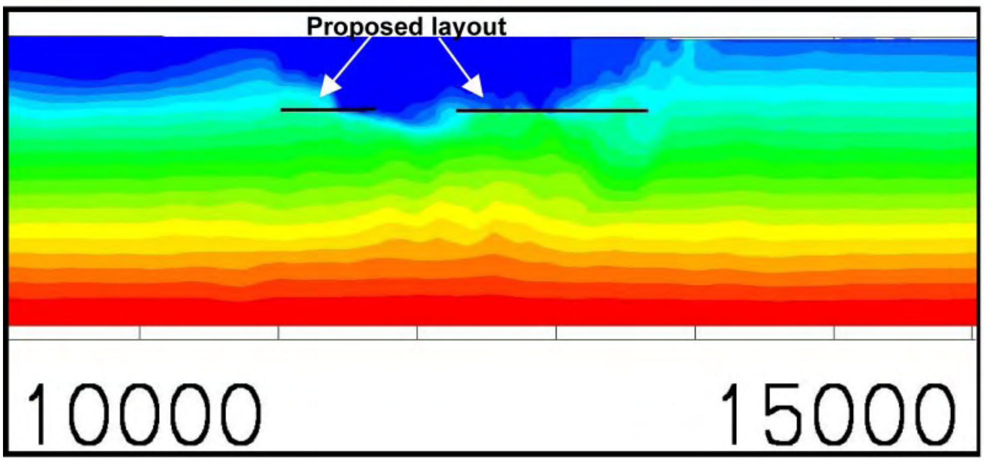


Figure 7-14. *Salinity distribution in the groundwater- grouting factor 0.001. Vertical profile W-E. Salinity, % Total Dissolved Solids (TDS).*

The resulting reduction in seepage levels was relatively small, being from 180 l/s to 160 l/s. The small size of the reduction is due to most seepage being associated with the deformation zone interceptions (hydraulic conductor domains) while the deposition holes have been deliberately confined to the less permeable intervening deposition blocks (hydraulic rock domains) and therefore the exclusion of the deposition holes leads to only a relatively small drop in seepage levels.

Reduced cut off length for the fracture network

The initial analysis assumed a minimum fracture length of L_{min} = 100 m. A complementary analysis was performed whereby additional shorter fractures, with lengths in the range of 10 to 100 m, were included. This was done by attaching a local fracture network, 3,000×3,000×300 m (*L*×*W*×*H*), centred on the repository.

Fracture transmissivity is related to fracture length according to the following relationship /SKB 2005a/:

$$T = 5 \cdot 10^{-8} \left(\frac{L}{100} \right)^2 \quad (\mathrm{m^2/s}) \qquad \text{Equation 7-3}$$

Transmissivities for fractures in the local network lie in the range of $T = 5\times10^{-10}$ to $T = 5\times10^{-8}$ m²/s.

The number of contact points between the simulated fractures and the tunnel geometry increased significantly as can be seen by comparing Figures 7-1 and 7-15. The shorter cut-off length resulted in essentially all of the deposition tunnels having contact with fractures.

However, the total inflow into the repository did not increase but remained at the 180 l/s level for a grouting factor of 0.1. This suggests that the rock mass hydraulic conductivity has not been increased by inclusion of the local network and that the short fractures in the model are not significant seepage pathways.

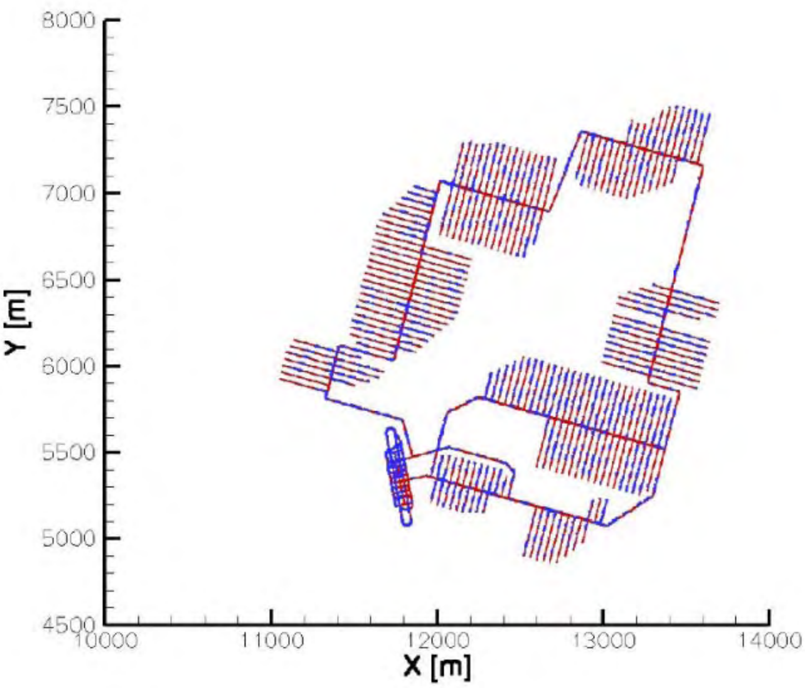


***Figure 7-15.** Contact points (blue) between the tunnel geometry and fractures, L > 10 m.*

7.4 Discussion and conclusions

The analytical approach gives an indication concerning local seepage and the effect of grouting. The significant inflows associated with certain zone passages highlight the need for detailed grouting studies.

The largest water inflows are associated with the passage of the deformation zones where hydraulic conductivities may be 1,000 times higher than in the surrounding rock mass. However, the tunnel sections intercepting deformation zones are relatively short. Major reductions in groundwater pressure due to local seepage at the zone passages are not expected. This appears to be confirmed by the numerical analysis, see Figures 7-4 and 7-5.

Both the analytical and numerical methods indicate that inflow into the repository will be dominated by the deformation zone passages and the larger fractures. It is thus these structures that should be the focus of future studies.

Grouting effectiveness clearly has an effect on the quantity of seepage. If grouting to a resulting hydraulic conductivity of 10^{-7} m/s (level 1) is achieved then this will result in a significant reduction in total seepage. However, the analytical estimation of the necessary seepage reduction in the passages, indicates that grouting to a hydraulic conductivity of 10^{-9} m/s (level 2) will be difficult to achieve for the zones with high hydraulic conductivities.

Groundwater table drawdown due to the development of the repository is moderate and local. The lateral extent of the depressed groundwater table is essentially limited to the area directly over the tunnels.

Saltwater is drawn into the repository, particularly if grouting is limited to the higher grouting factors, which results in an estimated salinity of 2–4% TDS around the repository, see Figure 7-14.

The prediction of seepage quantities by the different methods gives values of similar magnitude and suggests reasonable verification and degree of uncertainty in the results.

There is scope to improve a number of aspects of the analytical methodology that could increase the relevance of the results and generally decrease uncertainty. For example, in future work seepage should be analysed for different deposition units rather than investigating an open repository in its entirety.

The presented numerical analysis was based on the generation of a single discrete fracture network. The first step in any further sensitivity study would be to repeat the analysis for a number of simulated fracture networks.

If it is assumed that seepage is clearly dominated by inflow from the deformation zones, then the quantity of total seepage, drawdown and distribution of salinity should be relatively insensitive to the changes in the discrete fracture network. However, there will be local variations.

Standard design work involving underground facilities often involves specifying an acceptable level of seepage. In order to achieve this seepage requirement, a certain hydraulic conductivity for a deformation zone of a particular thickness needs to be reached, see Equation 7-1 in Section 7.2.1. If these parameters could be included in the numerical model then comparisons between the numerical and analytical models would be facilitated and the numerical model would have potential for wider application.

8 Estimation of rock grouting need

8.1 Input data and assumptions

The aim of Design Task H is to provide an estimate of the necessary rock grouting work needed for the proposed repository layout.

In accordance with the UDP guidelines the estimates were made for the following defined grouting levels:

1. Grouting level 1 – rock mass is sealed to a hydraulic conductivity of approximately $K = 10^{-7}$ m/s within the grouted zone.
2. Grouting level 2 – rock mass is sealed to a hydraulic conductivity of approximately $K = 10^{-9}$ m/s within the grouted zone.

For the purposes of the assessment the grouting work has been considered in two parts. Firstly, grouting of the rock blocks – the general rock mass lying in between the deformation zones and secondly, grouting of the rock associated directly with the deformation zones.

In accordance with UDP guidelines /SKB 2004a/ an assessment has been made of suitable grouting procedures for each grouting level including: number of holes and hole length in the grouting schemes, borehole diameter, look-out angle, number of grouting fans, grout mixes including additives, grouting method and total number of drilling metres for each type of excavation (grouting object).

It is important to have a limitation of high pH in the rock mass around the repository and in the KBS-3 concept /SKB 2000a/ and in the safety analysis it is assumed that grout with a pH value < 11 is used. To fulfil this requirement the calculation of grout quantities has been carried out with a standard mix that is based on a standardized re-calculation of materials used in order to obtain a low pH grout equivalent.

It is assumed that the work is based on a standard pre-grouting program and cement based grouts are used.

The quantity of grout for each grouting object has been estimated based on empirical experience, with the quantities being broken down based on the following divisions: deposition tunnels, main tunnels, ramp, vertical shafts, central area caverns and tunnels, along with transport tunnels outside the central area. The grout quantity estimates have been compared by analytical calculations based on a porosity model.

8.2 Execution

The design guidelines have been further developed by SKB during the course of the work. The resulting variations from UDP guidelines are presented below.

Grout quantity estimates and methods have considered the following issues:

- The use of low-pH cement.
- Estimates of resulting grout quantities remaining in the rock mass after blasting.
- Estimates of grout quantities for each grouting object have been made but have not taken into account timing issues.
- Verification of grout quantities by analytical methods, related to particular grouting techniques, is not considered applicable and is not presented.
- The proposed methods and assessment of quantities have included reference to the reported experience from the excavation of Äspö HRL /SKB 1994a/.
- The predicted quantities have been estimated and verified in accordance with UDP Appendix 2, Section 4.6. Analysis of grout quantities according to a porosity model has been carried out. The porosity is based on porosity estimates from hydraulic conductivity rather than DFN P_{33} values, since this data was not available from the DFN analysis carried out for design task C3.

The deposition blocks and proposed repository layout for the reference level –500 m are presented in Figure 5-1 and the tunnel passages through the deformation zones are presented in Figure 6-1. The geometries and lengths for the different excavation types and deformation zone passages, used in the grout take calculations, are presented in Tables 8-1 to 8-3.

Table 8-1. Summary of dimensions of the various facility parts, excluding the central area and ramp.

Facility part	Width (m)	Height (m)	Cross section (m^2)	Total length (m)
Deposition tunnel	4.9	5.4	25	54,262
Main tunnel	10.0	7.0	66	5,220
Transport tunnel	7.0–10.5	7.0	46–70	5,321[1]
Shaft				
– elevator and skip shaft	5.5	–	24	1,162
– air intake and exhaust shaft	2.5–3.5	–	5–10	1,576

1) Including straight stretches, curves and passages through deformations zones.

Table 8-2. Summary of dimensions of the facility parts for the central area and ramp.

Facility part	Width (m)	Height (m)	Total length (m)
Transport tunnel	7.0	7.0	849
Ramp	5.5	6.0	6,800[1]
Access tunnels	7.0 –10.0	6.8–7.0	266
Other tunnels	3.0	3.0–4.0	1,047
Rock caverns	12.0–15.0	7.0–15.5	512
Rock loading station	3.0–13.0	3.0–7.0	81

1) Including curves, passages and connecting tunnels to the ramp.

Table 8-3. Summary of tunnel passages through deformation zones.

Passage ID no	Deformation zone ID no	Zone thickness (m)	Hydraulic conductivity (m/s)	Dip[1] (degrees)	Interception angle (degrees)	Passage length[2] (m)
P1H	ZSMNE018A	30	9.80E–8	90	63	38
P2L	ZSMNW035A	20	6.30E–07	90	79	22
P3L	ZSMEW023A	20	6.30E–07	90	33	47
P4L	ZSMEW023A	20	6.30E–07	90	73	23
P5H	ZSMNW025A	5	5.20E–08	88	80	7
P6H	ZSMEW004A	30 ± 20	4.20E–07	70 ± 15	80	34
	ZSMEW028A	10		83		
P7H	ZSMNE012A	41	2.59E–06	50 ± 15	53	72
	Part A					18
	Part B					55
P8H	ZSMNS017A	20 (0.5 -10)	3.25E–06	90	31	51
P9L	ZSMNW035A	20	6.30E–07	90	39	40
P10H	ZSMEW004A	30 ± 20	4.20E–07	70 ± 15	46	52
P11H	ZSMNE018A	30	9.80E–08	90	78	32

1) /SKB 2005a/.

2) Value based on the deformation zone thickness, and the proposed layout, see Section 6.3.

The grouting program for the proposed repository needs to address the following specific issues:

- High water pressure within the targeted grout zone.
- High hydraulic conductivity within certain deformation zones.
- Requirement for relatively low acceptable seepage levels.
- Requirement for low pH-values in the rock mass, i.e. minimization of grout quantities.

It is expected that the use of cement grouts, standard grouting techniques, good professional practice and a careful quality control program, will generally yield satisfactory results. However, alternative techniques may need to be employed in particular cases involving the grouting of fine fractures and where a combination of high water pressure and high hydraulic conductivities are encountered. At such locations alternative grout mixes involving additives, the use of local concrete linings and freezing should also be allowed for.

8.2.1 Proposed grout type

No grout mix specifications are included in the current report but rather applications and performance requirements are stated. These requirements will form the basis for developing suitable mixes. However, for the purposes of estimating the grout volumes a grout mix needs to be defined. A mix based on work by /Dalmalm 2004/ has been assumed.

A total of four different grout mixes are proposed, along with a general cavity filling mix. The intended applications are presented in Table 8-4.

The difference between mix 1 and 2 in Table 8-4 is relatively small. Test results indicate that a higher number of mixes can fulfill the 90 μm penetration requirement than the 80 μm penetration requirement and thus mix 1 offers greater flexibility, see reference /Dalmalm 2004/ for test results.

Table 8-4. Proposed grout applications.

Mix	Description
1	Penetrates fractures ≥ 90 µm completely.
	Yield point 5–10 Pa.
2	Penetrates fractures ≥ 80 µm completely.
	Yield point 5–10 Pa.
3	Penetrates fractures ≥ 60 µm completely.
	No specified yield point.
4	Contains accelerator.
5	Backfilling of drill holes.

In the KBS-3 concept for the deep repository and in the safety analysis it is assumed that grout with a pH value < 11 is used. In order fulfil this requirement the calculation of quantities has been performed firstly by:

1. estimation of grout volumes using standard cement grout mix, then
2. by a standardized re-calculation converting these grout volumes to obtain a low pH grout equivalent.

However, grout performance is an issue actively being investigated by SKB and such criteria may be subject to future modification.

Table 8-5. Design mix for low pH grout.

Component	Quantities (kg/m^3)
Water content	599
Silica fume	419
Super plasticizer, SP 40	11
Micro cement OPC	299
wcr	0.83

8.2.2 Grouting method

Grouting criteria involving grout takes, time and pressure will be developed as part of the detailed design work and are not included in the current study. The focus of the grouting methodology in the current work has been to enable estimates to be made of likely grout takes, based on the project defined grouting levels.

The proposed grouting methodology and layout of the grouting schemes for the different parts of the repository are presented in Appendix A.

Rock block

Standard cement grouts and pre-grouting techniques are assumed in order to quantify the grout volumes for the two grouting levels. It is also assumed that the rock mass outside the deformation zones has an overall fairly low hydraulic conductivity /SKB 2005a/. The estimated drilling and grouting quantities are presented in Section 8.3.

The mean hydraulic conductivity of the rock mass with the exclusion of the deformation zones, $K_b = 1.6\times10^{-9}$ m/s /SKB 2005a/, is lower than that required by grouting level 1. This does not mean that grouting will not be required in these rock volumes but rather higher hydraulic conductivities will be encountered only locally and where these intercept the tunnels, grouting will be required.

Zones

The deformation zones have significantly higher fracture intensities and hydraulic conductivities than the surrounding rock mass. The seepage quantities have been investigated by design task G, presented in Section 7 of this report, and it was concluded that seepage into the repository will be dominated by the deformation zones. For the purposes of quantity estimates, standard cement grouts and pre-grouting methods have been assumed. The resulting quantities are presented in Section 8.3.

Grouting effectiveness

Grouting effectiveness has been assessed with the use of the following relationship /Eriksson and Stille 2005/:

$$1-\frac{Seepage\ when\ grouted}{Seepage\ without\ grout} \qquad \text{Equation 8-1}$$

8.3 Estimated drilling quantities

A summary of the estimated drilling quantities for the proposed grouting work for the deposition area is presented in Tables 8-6. Corresponding quantities for the central area and the ramp are presented in Table A-1, Appendix A.

Table 8-6. Summary of grout hole drilling quantities for the deposition area.

	Grouted boreholes (m)	Ungrouted boreholes (plugged) (m)	Probe holes (plugged) (m)	Control holes (plugged) (m)
Rock blocks				
Deposition tunnel				
–Grouting level 1	45,612	91,224	390,686	13,756
–Grouting level 2	466,438	1,053,248	202,578	122,879
Main tunnel				
–Grouting level 1	5,390	10,780	45,936	1,626
–Grouting level 2	56,048	126,560	24,360	14,765
Transport tunnel				
–Grouting level 1	4,620	9,240	39,234	1,393
–Grouting level 2	47,542	107,352	20,598	18,787
Deformation zones[1]				
–Grouting level 1				
*Method A	420	840		127
*Method B	13,410	1,980		2,805
*Method C	18,540	1,760		1,650
–Grouting level 2				
*Method A	2,574	1,584		561
*Method B	52,750	9,900		4,675

1) Passages through deformations zones in the transport tunnels.

8.4 Estimated grout volumes

8.4.1 Introduction

In accordance with UDP guidelines grout quantities have been estimated and thereafter have been verified with analytical calculations by the analytical models. It should be noted that the quantities estimated with the analytical method do not include the volumes of the actual drilled holes.

8.4.2 Estimated grout volume based on an empirical approach

The estimates have taken into account the grout scheme geometry, grout consumption per drill hole metre, number of drill holes grouted, follow-up grouting, the number of grout fans and the required hydraulic conductivity levels. The estimates are based on experience, particularly of grouting work undertaken in the Äspö HRL /SKB 1994a/. The key values used in the estimation of grouted volumes for tunnels within rock blocks and in passages through deformation zones are summarized in Table 8-7. The grouting effectiveness is estimated to be 75% in rock blocks applying grouting level 2 and to 86–92% in passages applying method A and grouting level 2. The calculated quantities are presented for respective repository part in Table A-2 to Table A-5, Appendix A.

Table 8-7. Summarized key values used in the estimation of grouted volumes for tunnels within rock blocks and in passages through deformation zones.

	Grout level	Series	Grout take[2] (%)	Grout volume (l/m)
Rock blocks	1	–	30	20
	2	1	30	20
	2	2	30	15
Deformation zones[1]				
– Method A	1	–	30	20
	2	1	70	15
	2	2	60	15
	2	3	50	15
– Method B	1	1	100	20
	1	2	70	15
	2	1–2	100	20
	2	3–4	80	15
	2	5	50	15
– Method C	1	1–2	100	20
	1	3–4	80	15

1) Passages through deformations zones in the transport tunnels.

2) Number of boreholes with grout take.

8.4.3 Verification by rheological modelling

It has not been considered appropriate to calculate grouting quantities for the entire repository, in accordance with Janson as presented in Appendix 2 of UDP. Due to the high degree of uncertainty and the many assumptions to assess the total grouting volume according to Janson /Janson 1998/ and the lack of verification and references, it was concluded not meaningful to carry out this assessment.

8.4.4 Verification by porosity modelling

The porosity model is based on the assumption that the existing voids in the rock are filled for a certain distance into the rock mass, the so-called grout zone.

Porosity, *p*, can be estimated by the equation below by /Brotzen 1990/:

$$\log_{p} = 0.17 \times \log K_{b} - 1.7 \pm 0.3 \qquad \text{Equation 8-2}$$

Where hydraulic conductivity is given for the 20 m scale, which is considered appropriate in relation to the grout scheme dimensions. The porosity value can then be used to estimate the grout volume per metre of tunnel.

8.4.5 Comparison of results

The grout quantities for excavations lying in the rock blocks and the deformation zone passages, derived from the initial empirical estimates and the attempt at verification using the porosity model, are presented in Tables 8-8 and 8-9. It is considered that the two different methods show reasonable agreement.

Table 8-8. Comparison of initial estimated grout quantities with quantities estimated from the porosity model. Quoted volumes are per facility part lying within rock blocks. They do not include the volumes of the actual drilled holes.

Facility part	Initial estimated grout volume (m^3)		Grout volumes according to the porosity model (m^3)		
Deposition area	Grouting level 1	Grouting level 2	Entire area, no division based on grouting level	10% of the area, –grouting level 1	65% of the area, –grouting level 2
Deposition tunnel	912	8,125	6,779	1,542	7,260
Transport tunnel	108	976	785	177	832
Main tunnel	92	828	992	223	1,052
Central area					
Ramp	88	781	666	153	720
Cavern 1, 2 + main	18	130	150	36	162
Transport + skip	49	433	385	86	404
Shaft 1, 2 and silo	90	370	486	112	529
Total	**1,357**	**11,643**	**10,241**	**2,328**	**10,959**

Table 8-9. Comparison of initial estimated grout quantities with quantities estimated from the porosity model. Quoted volumes are per facility part for where they intercept deformation zones. They do not include the volumes of the actual drilled holes.

Passage	Initial estimated grout volume (m^3)		Grout volumes according to the porosity model (m^3)
Deposition area	Grouting level 1	Grouting level 2	No division based on grouting level
P1H, P5H, P11H	8	39	81
P2L, P3L, P4L, P6H, P9L, P10H	240		227
P7H, P8H	330	931	126
Central area			
Ramp	339	1,085	435
Transport	196	478	193
Skip ramp			31
Total	**1,113**	**2,533**	**1,094**

8.4.6 Overall summary of estimated grout quantities for the proposed repository

An overall summary of the estimated grout quantities for the proposed repository is presented in Tables 8-10 and 8-12.

The quantities presented so far represent the estimated quantities of grout that are injected into the rock mass. However, a certain amount will be removed along with the blasted rock as excavation proceeds. An estimation of the "permanent" grout quantities that remain in the rock mass has been made, based on a porosity model and the results are presented in Tables A-6 and A-7 in Appendix A.

Table 8-10. Overall summary of estimated grout quantities (min–max). Quoted volumes are per facility part lying within rock blocks with plugged volumes of the actual drilled holes as a separate item.

Facility part	Estimated grout volume (m^3)	
Deposition area	**Grouting level 1**	**Grouting level 2**
Deposition tunnel	600–1,700	7,000–8,500
Transport tunnel	60–180	750–1,000
Main tunnel	70–230	800–1,100
Central area		
Ramp	60–180	700–800
Cavern 1, 2 + main	15–35	120–180
Transport + skip	30–90	400–450
Shaft 1, 2 and central silo	75–120	300–550
Plugged volume	1,430	3,865
Total	**2,340–3,965**	**13,935–16,445**

Tables 8-11. Overall summary of estimated grout quantities (max/min). Quoted volumes are per facility part for where they intercept deformation zones with plugged volumes of the actual drilled holes as a separate item.

Passage/facility part	Estimated grout volume (m^3)	
Deposition area	**Grouting level 1**	**Grouting level 2**
P1H, P5H, P11H	5–10	10–35
P2L, P3L, P4L, P6H, P9L, P10H	200–300	250–350
P7H, P8H	300–400	350–450
Central area		
Ramp	300–450	500–900
Transport + skip	150–200	200–300
Plugged volume	55	135
Total	**1,010–1,415**	**1,445–2,170**

Table 8-12. Estimated total amount of grout materials, including plugged volumes of the actual drilled holes.

	Grouted volume (m³)		Plugged volume (m³)		Silica fume (tonne)		SP40 (tonne)		Micro cement OPC (tonne)	
Rock blocks	**Grout level**									
	1	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**
Minimum	910	10,070	1,432	3,863	981	5,838	26	153	700	4,166
Maximum	2,535	12,580	1,432	3,863	1,662	6,890	44	181	1,186	4,916
Zone Passage	**Grout level**									
	1	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**
Minimum	955	1,310	53	133	422	605	11	16	301	431
Maximum	1,360	2,035	53	133	592	908	16	24	422	648

8.5 Discussion and conclusions

The presented estimates should be taken as an initial attempt to assess the scale of the grouting work associated with the development of the repository. The method of estimation involves a large number of uncertainties involving both the character of the rock mass and the production philosophy that will be applied. Due to the physical size of the repository even small changes in these aspects may lead to very large changes in the overall absolute grout quantities. The number of existing excavations, of similar type and depth from which experience can be drawn, is limited.

Future planning should focus on the development of different possible scenarios for the possible rock conditions to be encountered, along with the specific grouting methods and materials to be applied, in order to maximize preparedness prior to the work. In this respect the zone passages require particular attention.

The total grout quantity injected into the rock mass including plugged volume is estimated to be 3,350 to 5,380 m³ for grouting level 1 ($K = 10^{-7}$ m/s) and 15,380 to 18,615 m³ for grouting level 2 ($K = 10^{-9}$ m/s). The deposition tunnels with a total length of 54 km dominate the grouting requirements with 1,590 to 2,690 m³ for grouting level 1 and 9,750 to 11,250 m³ for grouting level 2, all including plugged volume.

9 Estimation of rock support need

9.1 Input data and assumptions

This design task deals with the estimation of the rock support required for a deep repository in the Simpevarp sub area according to the layout presented in Section 5. The estimation of rock support provides input for the analysis of groundwater composition carried out as part of the safety assessment, as well as for cost calculations.

The calculated quantities assume excavation openings according to facility description – Layout E /SKB 2002b/ with a revised generic model /SKB 2004d/.

The assessment assumes tunnel excavation is carried out using conventional drill and blast techniques. The choice of rock support elements follows UDP guidelines that require support in the facility to consist of conventional support elements such as rock bolts, shotcrete and wire mesh.

UDP requirements concerning the durability of rock support and the wish to minimize the use of cement in the deposition tunnels have also been adhered to. The design working life of the load bearing main system in the different parts of the facility is assumed to be $\geq$ 5 years in the deposition tunnels and deposition holes, and $\geq$ 100 in the other excavations. The rock support in the deposition tunnels is primarily based on Swellex bolts, wire mesh and straps to keep the use of cement to a minimum.

In the KBS-3 concept for the deep repository and in the safety analysis it is assumed that mortar with a pH value < 11 is used. In order to fulfil this requirement, the estimation of quantities has been performed by re-calculation of standard material volumes to material quantities based on low pH mortar mixes proposed by SKB.

To minimize the maintenance work in the facility it is assumed that all tunnels, besides the deposition tunnels, are supported by at least unreinforced shotcrete along the tunnel roof. For the same reason, shotcrete and systematic bolting in the roof, and shotcrete on walls are utilized as support in the rock caverns.

It is assumed that the skip shaft is excavated by drill and blast techniques, while raise-boring is used for excavation of the other shafts. It is only the elevator and skip shafts that will have any rock support and are included in the quantities. This implies protection against falling blocks in the lower part of the shafts for ventilation. The skip shaft is assumed to have unreinforced shotcrete as a general minimum rock support.

The rock support for the walls in tunnels and rock caverns is assumed to involve 75% of the total wall area.

9.2 Execution

This design task has been carried out in accordance with SKB guidelines as presented in Section 5.10 of UDP, /SKB 2004a/. The calculated quantities for rock support elements have been broken down into the following facility parts:

- deposition tunnels,
- main tunnels,
- transport tunnels outside the central area,
- ramp,
- tunnels in the central area,
- rock caverns,
- rock loading station,
- shafts.

In addition to quantities for rock support elements, the quantity of concrete and reinforcement for temporary plugging has also been estimated.

The proposed rock support solutions have not been verified in any other way than by empirical methods. The estimation of the rock support need is based mainly on the guidelines of the Q-system, which relate the level of support to the span or height of the opening and to the rock mass quality /Grimstad and Barton 1993/.

It has been assumed that the rock mass quality is in accordance with the background report to the site descriptive model S1.2 /Lanaro 2005/. The classification of the rock mass quality was performed by dividing the rock mass into rock domain A, B, and C, see Figure 2-1, with further subdivisions into competent rock and stochastic deformation zones. The stochastic deformation zones correspond to bore hole sections evaluated as deformation zones in the single-hole interpretation, yet not included in model S1.2 as deterministic deformation zones.

Deterministic deformations zones crossed by a passage in the deposition area are classified separately in accordance with the classification presented in Section 6. Passages outside the deposition area, not considered in Section 6, are included here in the estimation of rock support need.

Where curves and tunnel junctions in the ramp and transport tunnels in proposed layout have coincided with deformation zones, these have been treated as straight tunnel section intersections for the purposes of quantity estimates. It is considered in reality that the ramp geometry will be optimised to minimise such deformation zones intersections. It is further assumed that it is possible to displace the ramp in a northerly direction, compared to the earlier proposed layout, to avoid interception with ZSMNEW023A, while deformation zones ZSMNE018A and ZSMNE021A still intersecting the ramp, see Table 9-2.

Due to the significant number of junctions between the deposition and main tunnels it has been considered necessary to deal with these separately for the calculation of quantities. At such junctions the stress field changes and wedge failures are more likely. This has been accounted for by doubling the span in the Q-system when estimating the level of rock support. The area involving the high parameter level of support is presented in Figure 9-1.

Possible rock support due to spalling or expanding clay has not been considered in the calculation of quantities. The calculation of quantities does not include possible temporary support in zones with bad rock that cannot be utilized as permanent rock support.

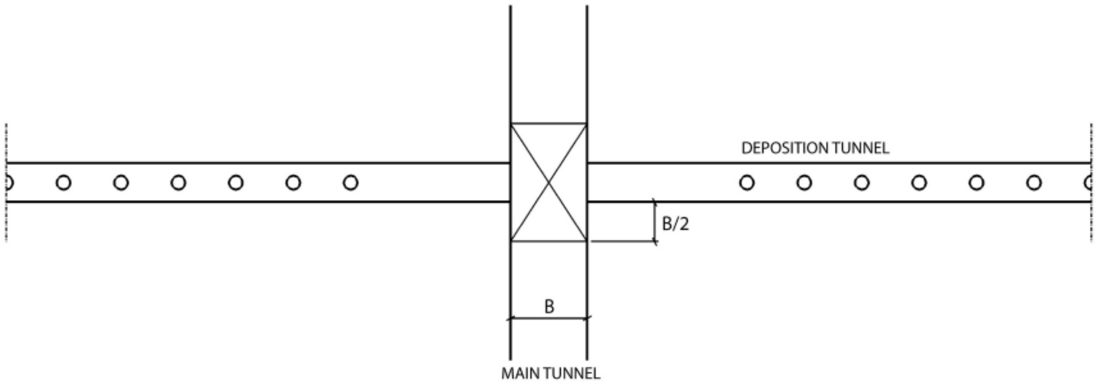


Figure 9-1. *Area for higher level of rock support at tunnel intersections.*

To account for rebound during shotcrete application, correction factors of 1.3 for tunnels and 1.2 for shafts and rock loading station have been assumed. The correction factor is reduced in shafts and the rock loading station since the contour is considered to be more even and the spoil less. The factor is based on *Tolerance 1* for rock excavation according to /Anläggnings AMA 98/. (Mean value of distance between theoretical and excavated rock contour ≤ 0.3 m).

It is further assumed that the portion of rebound associated with wet method shotcrete is in the order of 5 to 15% /Opsahl 1985/. However, it is important to note that the shotcrete area presented in the quantities is based on the theoretical rock contour and not a final as-excavated rock contour.

Where support involves fiber-reinforced shotcrete no thin cover layer of unreinforced shotcrete has been considered in the calculation of quantities.

9.2.1 Proposed rock support layout

The proposed layout is based on the distribution of deposition area as presented in Figure 5-1, Section 5.2, and the intersections between the tunnels and deformation zones in the deposition area shown in Figure 6-1, Section 6.3.

Geometries and lengths for the various parts of the facility and tunnel passages used in the calculations are presented in Tables 8-1 and 8-2, Section 8.2. The quantities concerning tunnel lengths are based on facility description – layout E /SKB 2002b/, the generic model /SKB 2004d/ and on design task F, Chapter 6 of the this report.

9.2.2 Rock mass quality

Rock mass quality, excluding deformation zone passages

Estimation of rock mass quality is based on classification according to the Q-system of the core from drill holes KSH01A, KSH02, KSH03 and KAV01 /Lanaro 2005/.

The rock mass has been divided up into rock domains A, B and C in accordance with the Simpevarp site description version 1.2. The rock mass has been further subdivided into "competent rock" and "stochastic deformation zones". Stochastic deformation zones correspond to bore hole sections that are defined as potential deformation zones in the single hole interpretations but are not included in the deterministic deformation zone model.

Figure 9-2 presents the distribution of competent rock and stochastic deformation zones based on interpretation of cored drill holes within the Simpevarp area and from KLX02 in Laxemar. According to the interpretation, the stochastic zones have core lengths in the order of 20–200 m, amounting to 10 to 20% of the total core length. The proportion of stochastic zones is estimated to be the largest within rock domain B and lowest within domain C. The rock quality (Q-index) as a function of frequency for each rock domain and competent rock/stochastic deformation zone grouping is shown in Figures 9-3 and 9-4.

Q-values from mapping of the drill core have been grouped into four intervals: 0.1–1.0, 1.0–4.0, 4.0–10.0 and > 10. As seen in Figure 9-3 the rock quality in competent rock is best within rock domain A (Ävrögranite).

As can be seen from Figure 9-4 a large portion of the stochastic zones have a rock quality Q > 4, i.e. a rock quality that corresponds to generally *fair* rock conditions.

For competent rock the parameters J_w and *SRF* along with the resulting ratio have assumed values according to Table 9-1. The estimations were made in accordance with the Q-system recommendations /Barton 2002/. For stochastic deformation zones J_w has the same value as for competent rock while *SRF* is dependent on the deformation zone properties.

Of the total tunnel length of 64,800 m in the deposition area see Table 5-2, 1,190 m (2%) are estimated to be within stochastic zones of *very poor* rock quality.

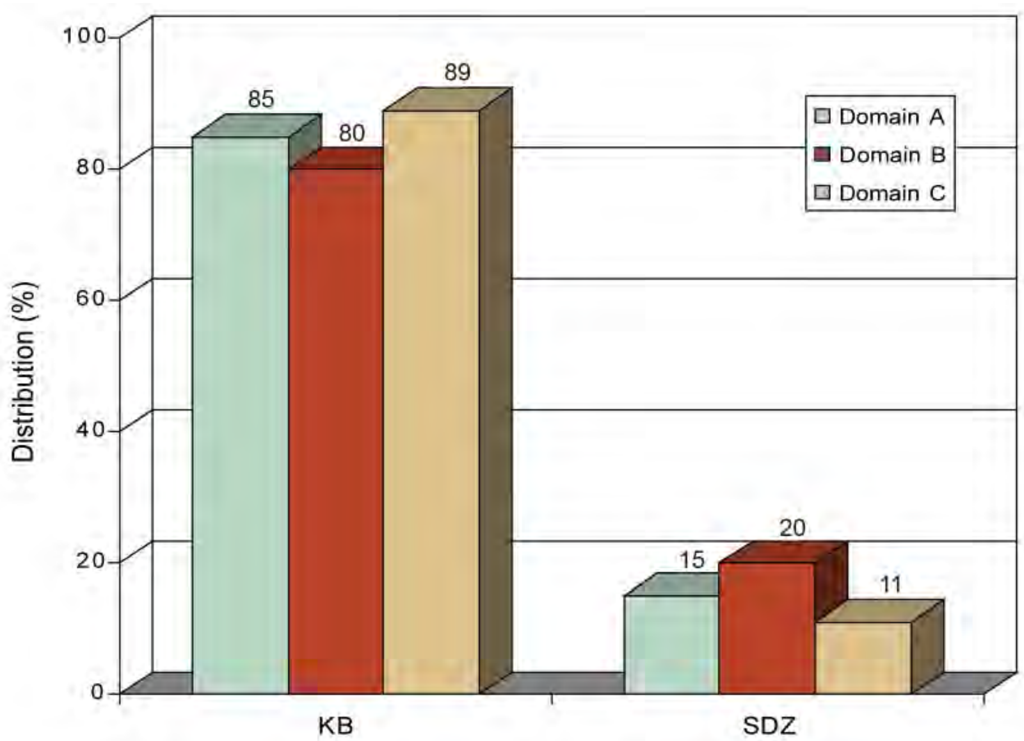


Figure 9-2. *Distribution of competent rock (KB) and stochastic deformation zones (SDZ) based on geological single-hole interpretations of cored drill holes within the Simpevarp area and from KLX02 in Laxemar /Lanaro 2005/.*

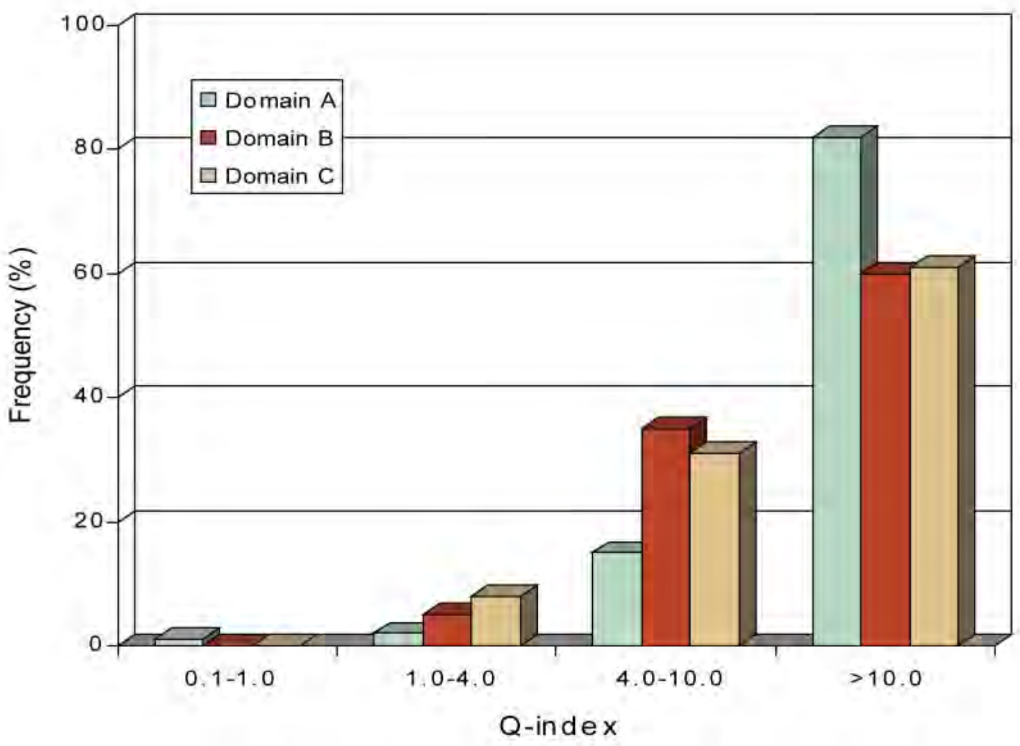


Figure 9-3. *Estimated rock quality in competent rock as a function of frequency and rock domain based on mapping of drill core within the Simpevarp area /Lanaro 2005/.*

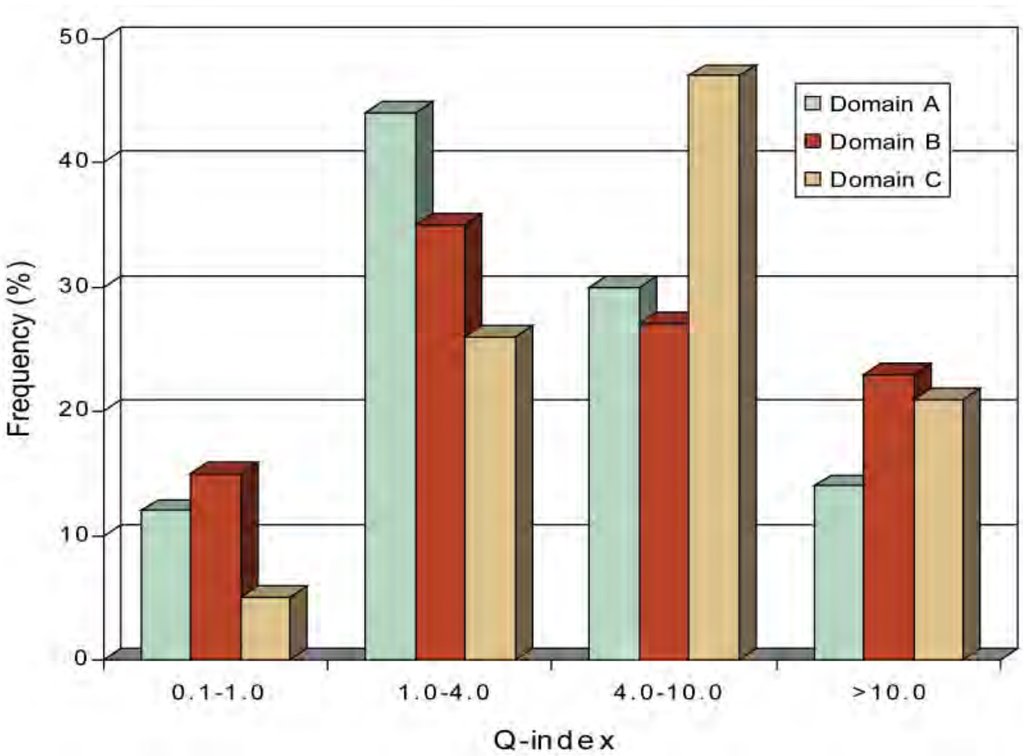


Figure 9-4. *Estimated rock quality in stochastic deformation zones as a function of frequency and rock domain based on mapping of drill core within the Simpevarp area /Lanaro 2005/.*

Table 9-1. Assumed values of parameters Jw and SRF, along with resulting ratio for competent rock. The values are in accordance with the Q-system recommendations /Barton 2002/.

Depth (m)	J_w	SRF	J_w/SRF
0–5	1.00	5.0	**0.20**
5–25	0.66	2.5	**0.26**
25–250	0.50	1.0	**0.50**
> 250	0.33	0.5	**0.66**

Rock quality in deforamtion zone passages in the deposition area

Deterministically interpreted deformation zones that have to be passed in the deposition area in the suggested layout are classified in accordance with design task F, Section 6.3. A summary of estimated rock quality for all passages in the deposition area is shown in Table 6-3.

In Figure 6-4 the total length is shown as a function of rock quality. The results are divided into passages through deformation zones of differing confidence levels. In passage P7H, where the zone is divided into two parts, the length of the passage has been subdivided based on rock mass class.

Of the total eleven passages in the deposition area, the rock quality is considered to be *extremely poor* in one passage (Q < 0.1), *very poor* in eight (Q = 0.1–1) and *poor* in two of the passages (Q = 1–4).

Of the total passage length of 417 m, 132 m (32%) relate to zones with a low level of confidence and 285 m (68%) to zones with a high level of confidence. Of the total passage length, 18 m (5%) are considered to be of extremely *poor*, 337 m (80%) of *very poor* and 62 m (15%) of *poor* rock quality.

Rock quality in deformation zone passages outside the deposition area

Passages through deformation zones outside the deposition area occur in the ramp and in some parts of the central area. The two deformation zones involved are ZSMNE018A (high confidence) and ZSMNE021A ("possible"). In the spiral ramp and central area these deformation zones are repeatedly intercepted. This results in the total passage length in the ramp and transport tunnel becoming relatively large. A summary of estimated rock quality and length of passages outside the deposition area is shown in Table 9-2.

Of the total passage length of 833 m, 272 m (33%) represent passages through zones of confidence "possible" with estimated poor rock quality, and 561 m (67%) through zones of high level of confidence, with estimated *very poor* rock quality.

Table 9-2. Assumed rock quality and length of passages outside the deposition area.

Deformation zone ID no	Level of confidence	Q-index	Facility part, length (m)			
			Ramp	Transport tunnel	Skip ramp	Connecting tunnel
ZSMNE018A	High	0.1–1[1, 2]	367	165	29	0
ZSMNE021A	Possible	1–4[1]	191	60	19	2
Total length			558	225	48	2

1) /SKB 2005a/.

2) /Curtis et al. 2003ab/.

9.2.3 In situ state of rock stress

Preliminary results reported in site description S1.2 indicate that within the Simpevarp area there are probably two different stress domains with significant differences in stress levels /SKB 2005a/.

Stress domain II appears to have a substantially lower stress magnitude than domain I, whereas the orientations of the main stresses are equal in the two stress domains.

Stress domain II is assumed to be delimited in the west by deformation zone ZSMNE012A and in the east by ZSMNE024A.

Reported results indicate that measured initial stresses in the depth range 400–700 m are normal in stress domain I and lower than normal in stress domain II. For the reference depth of 500 m the risk of spalling is estimated to be very small and essentially independent of tunnel orientation, see Section 4.3 of this report. Possible support specifically related to spalling has thus not been included in the calculation of quantities.

9.2.4 Proposed rock support

General

Proposed solutions for support are at this stage mainly based on guidelines given in Figure 9-5 /Grimstad and Barton 1993/. Minor deviations from the guidelines in the diagram have been made by introducing stepwise classes of support regarding bolt density and layer thickness of shotcrete. The requirement for steel-only based support solutions in the deposition tunnels and a general support proposal consisting of a skin of unreinforced shotcrete along the tunnel roof in other tunnels, in order to minimize maintenance, has led to certain deviations from the guidelines.

The level of proposed support is related to the equivalent dimension of the underground opening (D_e) and the estimated quality of the rock mass (*Q-index*). The equivalent dimension of the underground opening is obtained by dividing its width, diameter or height with a factor called *ESR* (*Excavation Support Ratio*).

The value of ESR is related to the utilization of the underground opening and the safety requirements enforced on installed rock support. In /Barton et al. 1974/ they have suggested the values shown in Table 9-3.

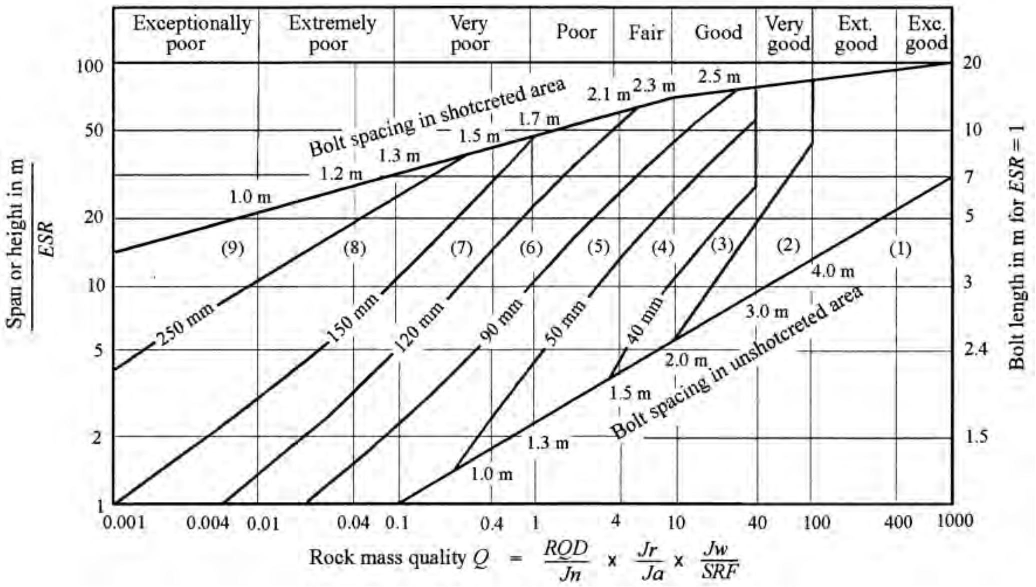
In the deep repository ESR has an applied value of 1.3 in the deposition tunnels and 1.0 in other areas of the facility. The higher ESR-factor in the deposition tunnels has been chosen, as access to these is temporary and limited to 5 years.

Suitable lengths for rock bolts are estimated by using Equation 9-1 /Barton et al. 1980/:

$$L = \frac{2 + 0.15B}{ESR} \qquad \text{Equation 9-1}$$

where,

B = Width or diameter of underground opening.



Rock Support classes

1) Non-reinforced.
2) Selective bolting.
3) Systematic bolting.
4) Systematic bolting and 40–100 mm non-reinforced shotcrete.
5) Systematic bolting and 50–90 mm fibre reinforced shotcrete.
6) Systematic bolting and 90–120 mm fibre reinforced shotcrete.
7) Systematic bolting and 120–150 mm fibre reinforced shotcrete.
8) Systematic bolting and > 150 mm fibre reinforced shotcrete arches.
9) In situ concrete structures.

Figure 9-5. *Nomogram with guidelines for choice of support level based on Q-index. /Grimstad and Barton 1993/.*

Table 9-3. Suggested values of ESR-factor (Barton et al. 1974).

Excavation category	ESR
A. Temporary mine openings	3–5
B. Permanent mine openings, water tunnels for hydropower, pilot tunnels, drifts and headings for large excavations.	1.6
C. Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D. Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E. Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Rock support type

The choice of rock support elements follows UDP guidelines that require support in the facility to consist of conventional support elements such as rock bolts, shotcrete and wire mesh. Furthermore, UDP requirements concerning the durability of rock support and the wish to minimize the use of cement in the deposition tunnels have also been adhered to. In accordance with UDP rock support for the deposition tunnels has been treated separately.

In the KBS-3 concept for the deep repository and in the safety analysis it is assumed that mortar with a pH value < 11 is used. In order to fulfil this requirement the calculation of quantities has been performed with mixes that are based on a standardized re-calculation of materials used in order to obtain a low pH mortar equivalent, see Table 9-4. The calculation for tunnel closures in the deposition tunnels is based on unreinforced shotcrete mix.

Table 9-4. Design mix for low pH grout for bolts and shotcrete.

Component	Bolt grout (kg/m^3)	Shotcrete (kg/m^3)
Water content	696	214
Cement content	596[1]	306[2]
Ballast	0	1,500
Silica fume	255	204
SP 40	2	7
Fibre quantity.	0	70
Density	1,549	2,301

1) White cement, wcr 0.82.

2) Wcr 0.42.

Deposition tunnels

In deposition tunnels suggested support elements consist of rock bolts in combination with wire mesh and straps. The straps consist of two by two connected reinforcement bars in 3 m lengths mounted between the bolts in lateral lines or in squares. The support elements have the following characteristic data:

Rock bolt:	Standard Swellex. Load rating 100 KN.
Mesh:	Galvanized wire ∅3 mm in 80×100 mm squares. Load rating approximately 5 ton/m^2.
Washer:	Galvanized reinforcement bars K500 ∅12 mm.
Mesh:	Galvanized spherical washer ∅150 mm, t = 6 mm

Other facility parts

In the remainder of the facility suggested support elements consist of rock bolts in combination with non-reinforced or fibre-reinforced shotcrete as surface support. In areas with fibre-reinforced shotcrete, rock bolts are installed with a spherical washer and half ball and nut. The support elements have the following characteristic data:

Rock bolt:	Galvanized epoxy coated reinforcement bars K500 ∅25 mm without pre-tension.
Bolt grout:	See Table 9-4
Shotcrete:	See Table 9-4
Washer:	Galvanized spherical washers with half ball and nut ∅150 mm, t = 6 mm.

Rock support classes

In compliance with UDP the reporting of suggested support classes is divided into deposition tunnels and other facility parts. The support classes are based on Q system guidelines presented in Figure 9-6.The density of bolts assumed for selective bolting has been estimated from documented support in the Äspö tunnel /Markström and Erlström 1996/. In the Äspö tunnel, for a length of 3,500 m, there are approximately 240 bolts in the roof and approximately 130 bolts in walls, that have been documented as selective rock bolts.

Certain bolting is performed in combination with unreinforced shotcrete. With an arch length of approximately 6 m and a wall height of approximately 3 m (4×0.75 m) this corresponds to a bolt density of approximately 85 m^2/bolt in the roof and approximately 160 m^2/bolt in the walls.

The conditions in future deposition tunnels can be assumed to correspond relatively well to the conditions in the Äspö tunnel in respect to probable rock conditions and anticipate rock support. Other facility parts are considered to have similar rock conditions but the raised level of minimum support involving a general shotcreting of the tunnel roof should allow a reduced selective bolting density to be implemented.

Deposition tunnels

Suggested support classes regarding rock bolts are shown in Table 9-5 and for mesh and straps in Table 9-6. Support classes based on the Q-index of the rock mass are then applied, see Table 9-7. A principle drawing of the suggested level of support in the deposition tunnels is shown in Figure 9-6

For the rock quality shown in Figure 9-3 and 9-4 the highest support classes are not utilized. The support classes have nevertheless been included to cover future possible requirements.

Table 9-5. Rock support classes regarding rock bolt support in deposition tunnels.

Support class	Selective/systematic	c-distance in the tunnel line (m)	Bolt density (m^2/bolt)
B0	Selective		60 roof/120 wall
B1	Systematic	3.0	4.5
B2	Systematic	1.5	2.25
B3	Systematic	1.0	1.0

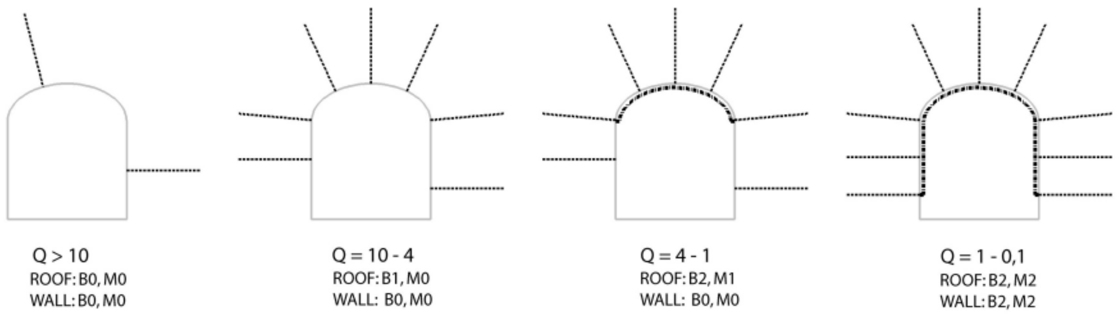


Figure 9-6. *Principle rock support for deposition tunnels.*

Table 9-6. Support classes regarding mesh and straps in deposition tunnels.

Support class		C-distance in the tunnel line (m)	C-distance perpendicular to the tunnel line (m)
M0	No mesh		
M1	Mesh, rock strap, lines	3	1.5
M2	Mesh, rock strap, squares	1.5	1.5
M3	Mesh, rock strap, squares	1.0	1.0

Table 9-7. Rock support levels in deposition tunnels.

Q-value	Designation	Roof	Wall
> 4	Fair rock and better	B0, M0	B0, M0
4–1	Poor rock	B1, M1	B0, M0
1–0.1	Very poor rock	B2, M2	B2, M2

Other facility parts

Suggested support classes regarding rock bolts are shown in Table 9-8 and regarding shotcrete support in Table 9-9. Support classes based on the Q-index of the rock mass follow the same principles as applied for the deposition tunnels, Table 9-7 above, details are not presented here. Principle drawings of suggested level of support for the main tunnels, rock loading stations and shafts are shown in Appendix B. As for the deposition tunnels it is not necessary to utilize the highest support classes at the rock quality shown in Figure 9-3 and 9-4.

Table 9-8. Rock support classes regarding rock bolt support in other facility parts.

Support class	Selective/ systematic	C-distance in the tunnel line (m)	Bolt density (m^2/bolt)
B0	Selective		80 Roof/120 Wall
B1	Systematic	2.0	4
B2	Systematic	1.5	2.25
B3	Systematic	1.0	1

Table 9-9. Rock support classes regarding support with shotcrete in other facility parts.

Support class	Unreinforced/ reinforced	Thickness (mm)
S0	No shotcrete	
S1	Unreinforced	25
S2	Fibre reinforced	50
S3	Fibre reinforced	100
S4	Fibre reinforced	150
S5	Arch	250

9.3 Results

9.3.1 Calculated quantities

Calculated quantities per rock domain and per facility part are presented in Tables B-1 to B-7, Appendix B. Estimated quantities for tunnels in the deposition area, along straight stretches excluding junctions and passages, are presented in Table 9-10.

Table 9-10. Estimated quantities for tunnels in the deposition area, along straight stretches, excluding junctions and passages through deformation zones.

	Unit	Transport tunnels	Main tunnels	Deposition tunnels
Total length	m	4,985[1]	3,441[1]	54,262[2]
Crown – Wall length[3]	m	16.3	18.8	12.2
Rock bolt number	no.	3,160	2,849	21,268
Average bolt density	m^2/bolt	26	23	31
Shotcrete/mesh area[4]	m^2	43,138	23,960	45,999
Shotcrete/mesh area divided by Crown – Wall area[3,4]	%	53	37	7

1) Length of straight tunnel stretch excluding junctions.

2) From main tunnel wall to deposition tunnel end.

3) Involve 75% of the height of the tunnel walls.

4) Based on theoretical rock contour.

The total quantity of bolts in the complete facility is calculated to be 45,000, with approximately 20,000 in the deposition tunnels. The calculated quantities correspond to an average bolt density of approximately 30 m^2/bolt in the deposition tunnels.

The total weight of the steel used for support elements in the deposition tunnels is estimated as190 tonnes. This may be compared with the weight of the concrete reinforcement used for tunnel closure calculated to be approximately 2,100 tonnes.

The area in the deposition tunnels supported by mesh is calculated to be approximately 46,000 m^2. This may be compared with the total area in other parts of the facility that are supported by shotcrete of 201,000 m^2 based on a theoretical rock contour. The calculated area supported by mesh corresponds to 5–10% of the total crown-wall area in the depositions tunnels.

The total amount of unreinforced shotcrete, based on a realistic rock contour including re-bound (5,090 m^3), is approximately 60% greater than the amount of fibre reinforced shotcrete (3,160 m^3). The calculation of quantities results in a total weight of cement for rock support of approximately 2,645 tonnes and a total weight of cement for the tunnel closures of approximately 8,105 tonnes.

9.3.2 Sensitivity analysis

Several of the parameters that are part of the basis for quantity calculations are uncertain and it is difficult to assign values. The following parameters are examples that contain uncertainties and that can have great impact on calculated support quantities.

- The bolt density at selective bolting.
- The conversion factor between theoretical shotcrete quantity and applied quantity on the rock surface, including re-bound.
- The distribution of Q-values in the rock mass.
- The division between competent rock and stochastic deformation zones.
- The rock quality and width of deterministic and stochastic deformation zones.

Parameters that are considered to have the highest level of uncertainty are the bolt density for selective bolting, the distribution of Q-values in the rock mass and division between competent rock and stochastic deformation zones.

For two of the parameters, the bolt density for selective bolting and the distribution between competent rock and stochastic deformation zones, a sensitivity analysis has been performed. The support quantities for deposition tunnels, main tunnels and transport tunnels have been calculated with the following assumptions:

- Doubled bolt density for selective bolting
- Variation in the proportion of stochastic deformation zones by –5% and +10%.

The frequency distribution of the rock quality for competent rock and stochastic deformation zones was not changed during the sensitivity analysis. For reference the results are shown with earlier calculated quantities for the deposition tunnels in Tables B-8 and B-9 and for the main tunnels and transport tunnels in the deposition area in Tables B-10 and B-11, Appendix B.

According to the results a doubled bolt density for selective bolting will require an increase of the bolt quantity by 36% in the deposition tunnels and a total increase in main and transport tunnels of 8%. Thus the bolt density for selective bolting has a relatively large impact on the bolt quantities in the deposition tunnels whereas the changes in main and transport tunnels are limited. The reason for this being that the support in the deposition tunnels mainly consists of selective bolting, which is not the case in the main and transport tunnels.

A reduction by 5% in the proportion of stochastic zones creates a reduction of 13% in the bolt quantity and a reduction of 21% in the mesh area in the deposition tunnels, whereas an increase by 10% in such zones creates an increase of 26% in the bolt quantity and an increase of 41% in the mesh area.

Correspondingly, in the main and transport tunnels a reduction in stochastic zones by 5% creates an 8% reduction in the bolt quantity, < 1% reduction in the quantity of unreinforced shotcrete and 10% reduction in the quantity of fibre reinforced shotcrete. An increase by 10% of such zones creates an increase of 16% of the bolt quantity, < 1% increase in the quantity of unreinforced shotcrete and 20% increase in the quantity of fibre reinforced shotcrete.

The reason for the relatively minor change in unreinforced shotcrete is that the deformation zones normally are supported by fibre reinforced shotcrete.

In general, based on the actual parameter study, it seems fair to assume a variation interval of at least –15 to +40% around the calculated quantities due to uncertainties in the underlying parameters. Furthermore, it is likely that the interval of uncertainty had been even greater if the distribution of Q-value in the rock masses had been included in the parameter study.

9.3.3 Discussion

The chosen solution for support, with general shotcreting of the tunnel roof in order to minimize the maintenance in the facility, causes additional shotcrete quantities compared to the guidelines in the Q-system. At a later stage it may be relevant to compare costs for these quantities with costs for future maintenance including scaling the tunnel roofs in the facility. It may also be motivated to compile quantities and costs that have been generated by deviations from the guidelines in the Q-system regarding, for example, bolt density, bolt lengths and shotcrete layer thickness.

One reason that the distribution between competent rock and stochastic deformation zones is judged as an uncertain parameter is that the evaluation is mainly performed in vertical holes, whereas most facility parts will be laid out horizontally. It is thus possible that the number of zones has been underestimated by the core drilling at the same time as the zone widths are overestimated compared to tunnels and caverns.

The ratio Jw/SRF may possibly be neutral (= 1) as water inflows are largely dealt with by pre-grouting. However, in the actual calculation of quantities the ratio follows the recommendations of the Q-system.

The ESR value, which is related to the utilization of the rock volume and the safety requirements on installed support, should be more closely assessed in connection with further application of the Q-method. In certain facility parts the parameter value appears to have a great impact on the calculated quantities.

As stated in Section 6, the rock mass quality presented for the identified passages is considered to be somewhat conservative. The reason for this is the simplified description of the deformation zones assuming the geological properties to be uniform and neither vary with depth nor along the passage length.

9.3.4 Conclusions

A preliminary estimate has been calculated for required support quantities in the repository. Due to uncertainties in the underlying parameters it seems reasonable to assume a variation interval from –15 to +40% around the calculated quantities.

It is furthermore probable that the quantities may change substantially due to changed design criteria and assumptions in later stages of the design.

The total quantity of bolts in the complete facility is calculated to be 45,000, with approximately 20,000 being in the deposition tunnels. The area in the deposition tunnels supported by mesh is calculated to be approximately 46,000 m^2. This may be compared with the total area in other parts of the facility that are supported by shotcrete of 201,000 m^2, based on a theoretical rock contour.

The total amount of unreinforced shotcrete, based on a realistic rock contour including re-bound (5,090 m^3), is approximately 60% greater than the amount of fibre reinforced shotcrete (3,160 m^3). The calculation of quantities results in a total weight of cement for rock support of approximately 2,645 tonnes and a total weight of cement for the tunnel closures of approximately 8,105 tonnes.

10 Technical risk assessment

10.1 Purpose and scope

As part of design task K a technical risk assessment has been performed in accordance with UDP guidelines to *"establish a feedback between the design results and the goals of rock engineering in design step D1. The purpose of the feedback is to ensure that the premises comprising the design basis are illuminated from several aspects with a view towards the aforementioned goals."*

"The technical risk assessment was limited to the completed design in design step D1 and provides proposals for measures aimed at preventing the occurrence of undesirable events. The technical risk assessment does not include events that are associated with the construction and operating phases or the post-closure phase."

10.2 Execution

The two main questions for the risk assessment are:

1. Can the repository be accommodated within the assigned area?
   A detailed description of procedures, input data and results is given in Section 10.3.
2. What can go wrong with the applied design methodology?
   A detailed description of procedure and results is given in Section 10.4.

Based on the answers from these two questions feedback can be given to:

- The design organisation concerning the need for further studies and changes in working methods.
- The site organisation concerning possible need for further investigations.
- Safety estimates.

The carrying out of design task K for Simpevarp has concentrated on the main tasks and with data mainly collected from the results of design tasks A–E, i.e. Chapter 3–5 in this report.

10.3 Can the repository be accommodated within the assigned area?

The assigned area for the task is limited to the Simpevarp interest area /SKB 2003/, see Figure 10-1.

The strategy to be able to answer this question is to build a model that can investigate the possibility for the repository to be contained within the assigned area. The aims of the modelling are to:

- Estimate the probability that the repository can accommodate 6,000 canisters.
- Estimate the required rock volume for the repository.
- Give a complete picture of the influence of different factors of uncertainty.
- Identify controlling factors.

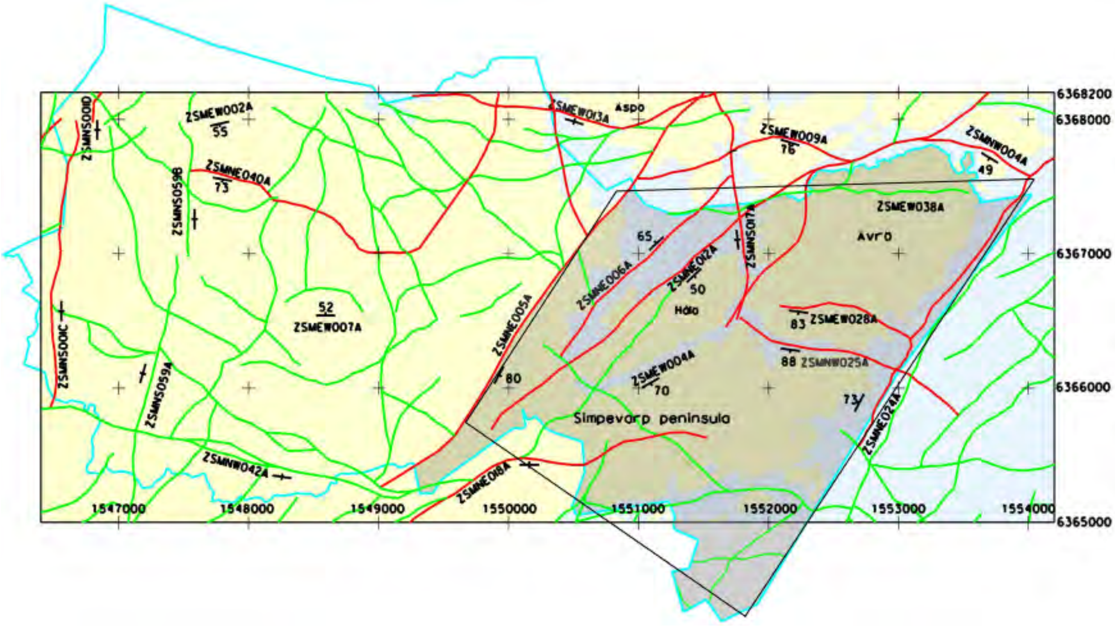


Figure 10-1. *Area for deposition (Simpevarp) and the simplified area used in the mode. (Ground level).*

In this section are described the methodologies used to answer the different questions that together comprise the main question.

Stochastic input data

For estimation of the distribution, principles given by e.g. /Stille et al. 2003/ have been used. When there have been no physical reasons or data to base the choice of model on, entropy-principles have been applied. These principles give a suitable distribution based on the state of knowledge. This will often lead to a triangular distribution, which is the distribution corresponding to the assessor’s willingness to give minimum, maximum and most likely value.

10.3.1 Global methodology

The total number of canisters, N_T, which can be accommodated can be calculated using the following formula:

$$N_T = \frac{A_{Net}^{-500} \cdot (1-k)}{A_S^{Adjusted}} \qquad \text{Equation 10-1}$$

A_{Net}^{-500}	=	the total available area for deposition on the studied level (–500 m) after reductions for zones, margins for zones and central area,
k	=	a loss factor that gives the number of geometrically possible hole positions that are lost according to certain criteria,
$A_S^{Adjusted}$	=	specific hole area, i.e. the total required area per deposition hole in relation to distance between hole and tunnel and adjoining part of the tunnel system area.

A calculation model according to Equation 10-1 above has been made in Excel and simulations according to the MonteCarlo method by using Crystal Ball /Crystal Ball 2000/.

By using the model, in addition to the total number of canisters that can be accommodated in the area, the probability that at least 6,000 canisters can be accommodated and the expected total rock volume for deposition of 6,000 canisters have been calculated. In Appendix C the execution and input for the following parameters are presented:

1. Estimation of available deposition area.
2. Estimation of hole loss factor.
3. Estimation of specific hole area.
4. Estimation of rock spoil from blasting for deposition of 6,000 canisters.

The results of the calculations to answer the question "Can the repository be accommodated within the assigned area?" are described in Section 10.3.2.

Extent and limitations

The factors that have been considered and are included in the model are:

- Dip of outer boundary lines.
- Existence of deformation zones.
- Respect distance (RD).
- Margin for excavation (MFE).
- Loss of deposition holes due to
  - Fractures
  - Water
  - Wedge breakout
  - Spalling
- Hole spacing.
- Main and transport tunnels.
- Central area.

Factors or conditions that have not been considered are:

- Overrating of eliminated areas at overlapping zones.
- Detailed geometries (e.g. that certain areas in reality are too small to be able to accommodate deposition tunnels).
- Actual tunnel layout (e.g. for geometrical reasons deposition tunnels cannot be made to full length).

The model is considered, even in view of these limitations, to contain all relevant factors that are needed in order to answer the put questions with sufficient accuracy.

10.3.2 Results

The model has been simulated according to the MonteCarlo method for the parameters in Section "10.3.1 Global methodology" and Appendix C. The number of simulation steps has been 100,000. The most important result from the presented simulations is that there is a very high probability that 6,000 canisters can be accommodated within the studied area at level –500 m. As indicated in Figure 10-3 there is a very high probability to accommodate 6,000 canisters, and the calculations show a 99.75% probability that at least 6,000 canisters can be accommodated within the area. Probably more than 9,000 canisters can be accommodated. The lowest number of canisters that was calculated during any of the 100,000 simulations was 4,069.

The loss factor, expressed as (1–*k*), states the number of hole positions that for various reasons must be disapproved. The total hole loss is 13% in average but can in extreme cases approach 60%, see Table 10-1 and Figure 10-5.

Table 10-1. Summary of the most important results for Question 1.

Forecast	Unit	Mean	Median	Range minimum	Range maximum
A_{Gross}^{-500}	km²	6.6	6.6	5.6	7.5
A_{Net}^{-500}	km²	3.5	3.5	2.0	5.6
Factor (1–k)	–	0.9	0.9	0.4	1.0
$A_S^{Adjusted}$	m²	332	329	281	443
Number of approved canister positions	no s	9,254	9,196	4,069	15,866
Volume of rock spoil from blasting (previous forecast)	m³ (f m)	3,262,095	3,241,582	1,868,444	5,024,418
Volume of rock spoil from blasting for 6,000 approved canister positions (more precisely: slumped rock volume for between 5,900 and 6,100 canister positions)	m³ (f m)	2,607,169	2,564,739	2,098,270	3,424,908

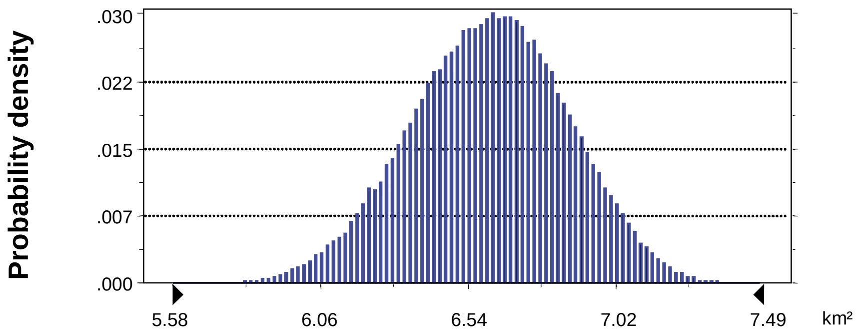


Figure 10-2. *Forecast A_{Gross}^{-500}, Unit: km². Frequency Chart, 100,000 trials.*

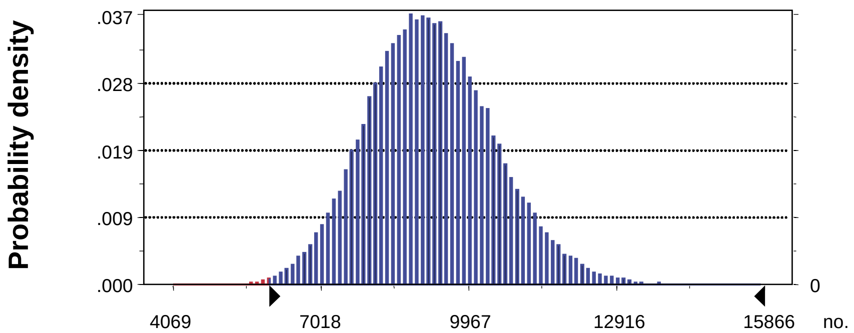


Figure 10-3. *Forecast: Number of canister positions, Frequency Chart, 100,000 trials. Certainty is 99.75% from 6,000 to +Infinity.*

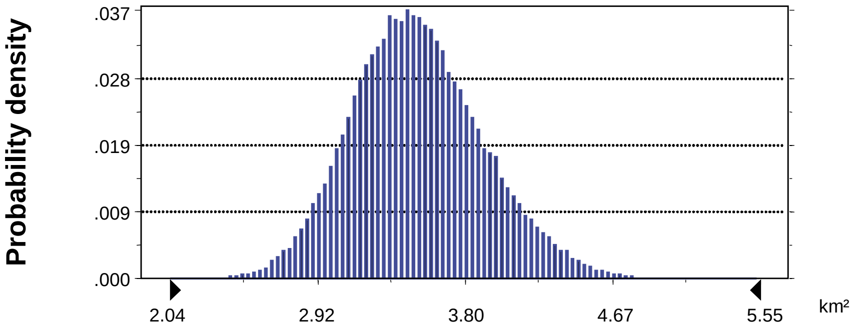


Figure 10-4. *Forecast:* A_{Net}^{-500}*, Unit: km*^{2}*. Frequency Chart, 100,000 trials.*

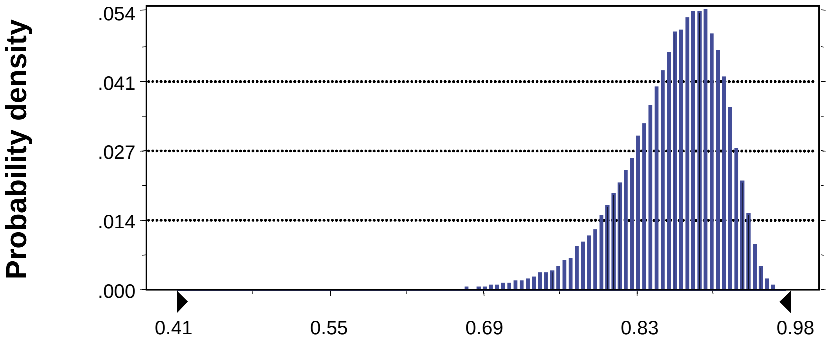


Figure 10-5. *Forecast: (1–k). Frequency Chart, 100,000 trials.*

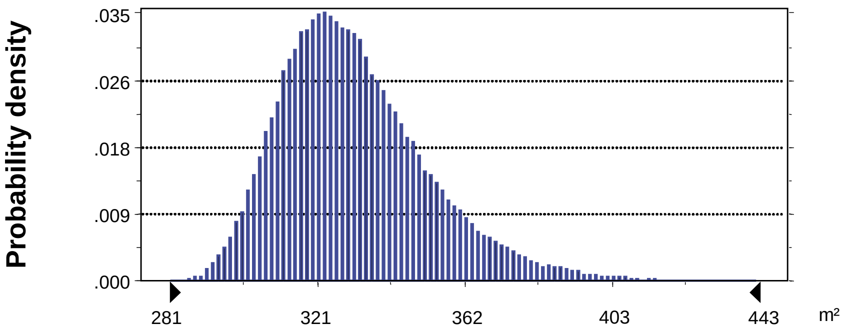


Figure 10-6. *Forecast:* $A_S^{Adjusted}$*, Unit: m*^{2}*. Frequency Chart, 100,000 trials.*

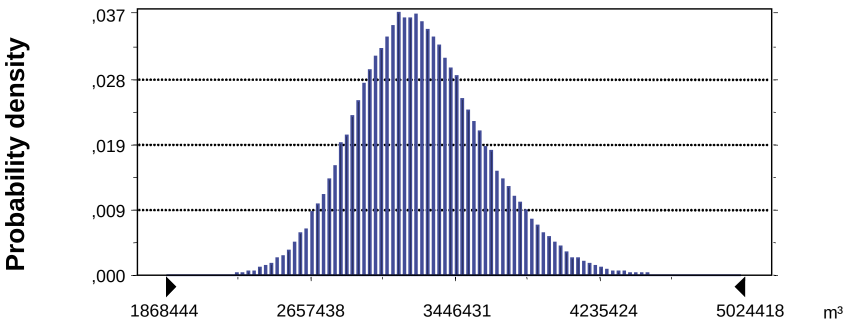


Figure 10-7. *Forecast: Volume of rock spoil from blasting, Unit: m*^{3}*. Frequency Chart, 100,000 trials.*

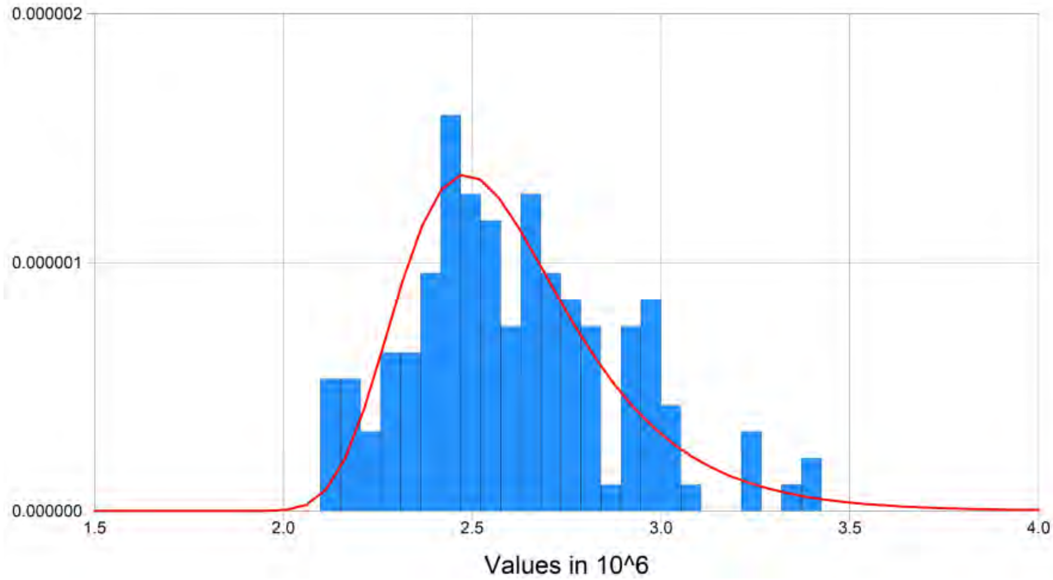


Figure 10-8. *Forecast: Probability density function of volume of rock spoil from blasting, 6,000 canister positions, Unit: tfm3. Histogram based on 141 values. An adapted extreme value distribution is also shown (Mode 2.48E+6; Scale 2.17E+5).*

In order to illustrate which factors have the largest impact on the results of the presented simulation, the factors have been ranked according to their relative contribution to the uncertainty (the variance). The 10 highest ranked factors are presented in Figure 10-9 and discussed below.

- **Hole distance, Domain A**: The reason for this factor being placed highest is that Domain A covers the largest area and that thermal properties influence the hole distance that controls $A_S^{Adjusted}$. Variation and scale effects of thermal properties need to be further investigated.
- **Loss percentage due to larger fractures:** Has a major influence and should be investigated (see earlier comments under Section Input data).
- **Dip of outer borderlines:** These control directly the gross area at level –500 m and is therefore of major importance. The dip of the zones cannot be influenced but greater knowledge can reduce the uncertainty. It should be investigated whether it is economically viable to reduce the uncertainty, for example, by further site investigations. A lesser variation (uncertainty) reduces the probability that < 6,000 canisters can be accommodated.
- **MFE:** Has a relatively large influence.
- **Likelihood of long zones**: Long zones, especially with RD of 100 m or large MFE, create a large area reduction. As for outer borderlines: it should be investigated whether it is economically viable to reduce the uncertainty.
- **RD:** This factor does not appear in the top 10 list as it is not stochastic. It is however of major importance and it should be considered to introduce a more varied evaluation of RD.

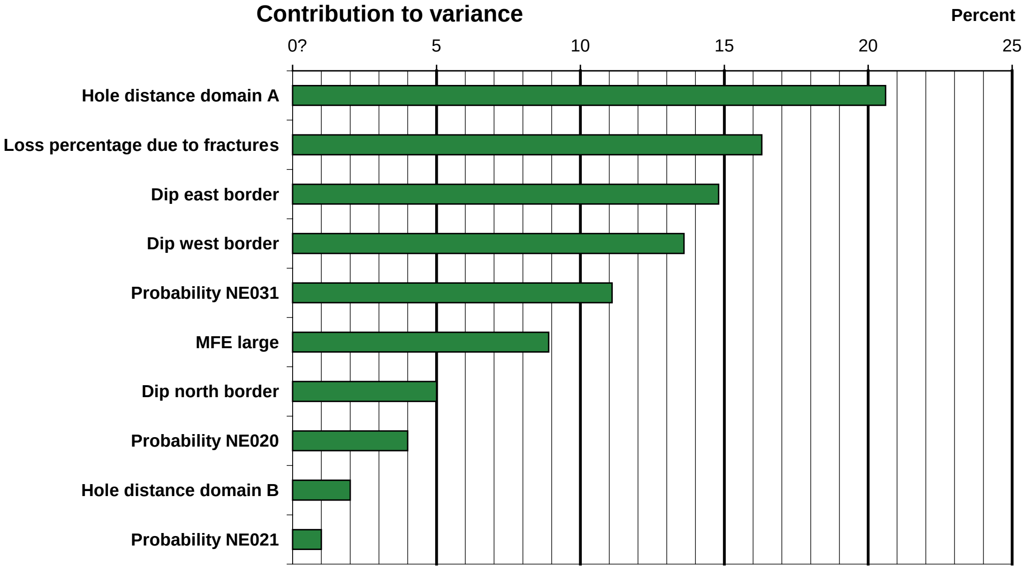


Figure 10-9. *Factors ranked according to their relative contribution to the variance.*

10.3.3 Conclusion and recommendations

Most important results concerning the available rock volume:

- There is a very high (99.75%) probability that 6,000 canisters can be accommodated within the studied area at level –500 m.
- The total hole loss, loss factor (1–*k*), is 13% on average.
- The average area needed to host the 6,000 canisters at –500 m is 3.5 km² with a range of 2–5.6 km². This is within the limits of what was found in the layout studies in Chapter 5, where the area 4.5 km² was needed according to that example. It should be noted that the layout study was not optimised.
- The three factors that have the largest impact on the uncertainty are:
  - Hole spacing due to thermal properties.
  - Loss percentage due to fractures with R > 100 m.
  - Dip outer border deformation zones.

Recommendations related to the question "Can the repository be accommodated within the assigned area?"

- The variation of thermal properties of the rock mass and scale effects should be further investigated.
- The loss criteria "Large adjacent fractures" should be further analysed.
- The credibility and properties (e.g. dip) of zones cannot be influenced but greater knowledge can reduce uncertainty. It should be investigated whether it is economical to reduce uncertainty (e.g. by further site investigations). A means of help could be to further use the built simulation model.

10.4 Risk assessment of Design methodology

The design methodology for the final repository for spent nuclear fuel is described in UDP /SKB 2004a/. Prior to design phase D2 the design methodology will be subject to a risk assessment. The goal is to review the design methodology and, in case faults are discovered, to suggest improvements in the work methodology. The results of the risk assessment may be used for:

- Change of methods and work methodology.
- Showing need for additional data and further investigations.

The intention of the analysis is to identify possible weaknesses and faults in the design methodology that may lead to significant problems during the construction phase and where possible, make recommendations regarding measures to deal with the risk will be given.

From the "Underground Design Premises" /SKB 2004a/:

The technical risk assessment is performed to establish a feedback between the design results and the goals of rock engineering in design step D1 according to Section 2.4. The purpose of the feedback is to ensure that the premises comprising the design basis are illuminated from several aspects with a view towards the aforementioned goals. Technical risk analyses will be carried out in later design steps.

The technical risk assessment shall be limited to completed design in design step D1 and shall include providing proposals for measures aimed at preventing the occurrence of undesirable events. The technical risk assessment shall not include events that are associated with the construction and operating phases or the post-closure phase.

Proposals for preventive measures may consist of recommendations for further studies and investigations.

In the UDP /SKB 2004a/ certain directives are given for the design, while certain other parts are decided by the design engineer.

10.4.1 Methodology

For the analysis an '*event tree*' method has been chosen to illustrate possible consequences that may appear when using the current design methodology. A '*fault tree*' is connected to the event tree, being used to analyse possible chains of events that may lead to sub-events in the event tree.

With regard to time and cost limitations, the analysis gives qualitative descriptions of consequences and shows possible chains of events that may lead to these consequences.

10.4.2 Structuring and identification of risks

For the current risk identification it has been assumed that the project to some extent is not cost sensitive. As an example, the event "The repository could have been built at a lower price" has not been included in the analysis. The identification and structuring (with trees) have been made in parallel so that the chains of events have been analysed at the same time as a risk has been identified.

It should also be noted that there are differences between different parts of the repository, e.g. demand on working life, tolerance to extensive support etc. The demand on working life for deposition tunnels is considerably less than for the main tunnels, at the same time as the demand for restrained grouting and rock support is greater for the deposition tunnels.

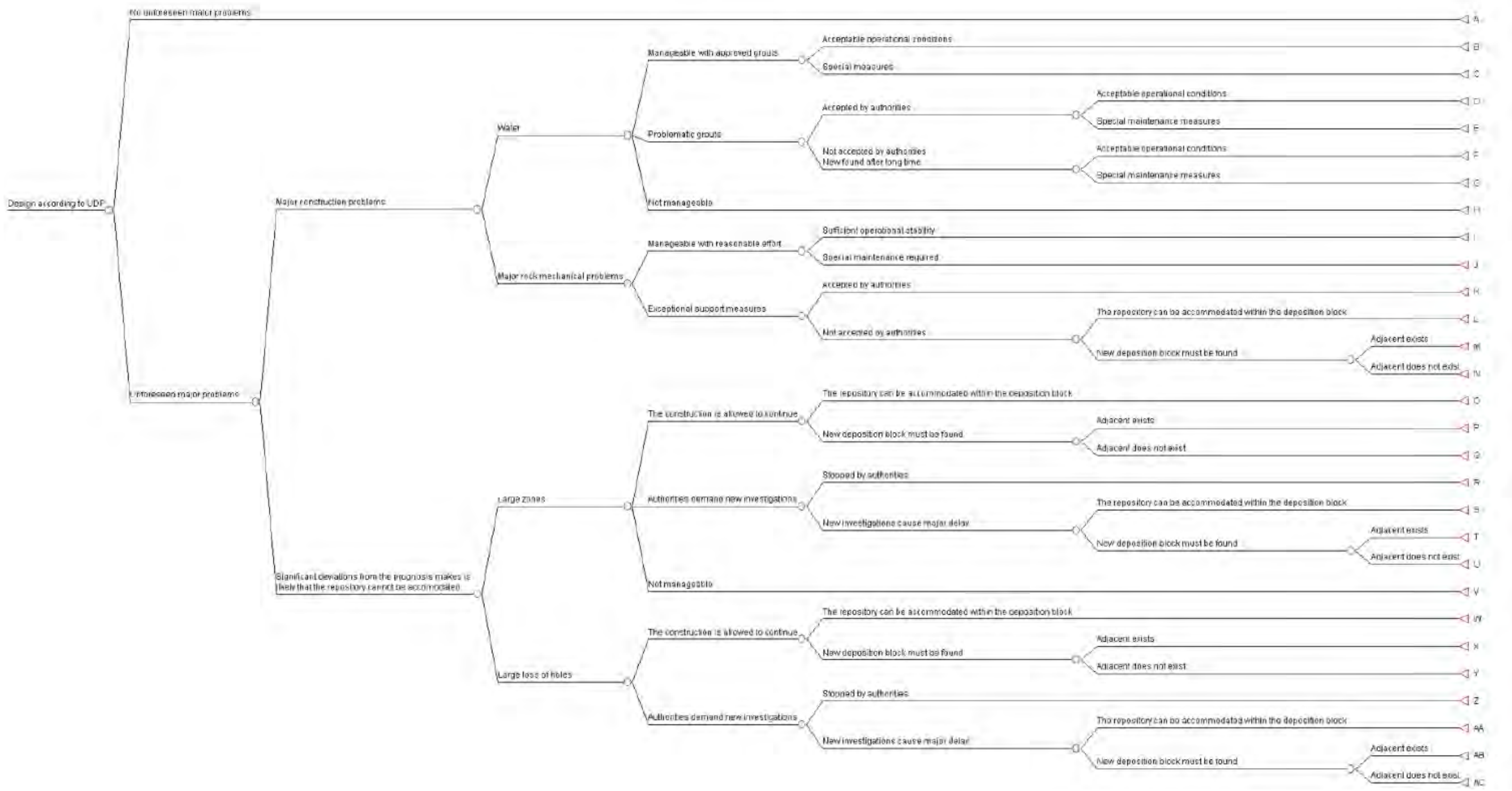


Figure 10-10. *Event tree showing possible results of a starting event "Design according to UDP"*

In the tree there are a great number of possible results. These are summarised in Table 10-2 (see also comments after the table).

Table 10-2. Description of possible results according to the event tree.

Name	Event description	Additional cost type	Comment
A	No unforeseen problems.	Construction costs	
B	Manageable water problems.	Construction costs + additional grouting costs	
C	Manageable water problems but demands for measures during continued service time.	Construction costs + additional grouting costs +maintenance cost (water)	
D	Water problems manageable with special grouting products.	Construction costs + grouting costs special products	
E	Do. but demands for maintenance.	Construction costs + grouting costs special products +maintenance cost (water)	
F	Water problems. Not manageable with available products. Acceptable product can be procured.	Construction costs +cost developing grouting products + waiting time +grouting costs special products	
G	Do. but demands for maintenance.	Construction costs +cost developing grouting products + waiting time +grouting costs special products + maintenance cost (water)	
H	Not manageable water problems.	Construction stop New site	H
I	Manageable stability problems.	Construction costs + additional rock support	
J	Do. but demands for maintenance.	Construction costs + additional rock support + additional maintenance (stability)	
K	Exceptional rock support required. Accepted by authority.	Construction costs + cost for exceptional rock support	
L	Exceptional rock support required. Not accepted by authority .	Construction costs + cost for substitution area	
M	Exceptional rock support required. Not accepted by authority. The repository cannot be accommodated within intended deposition block. There are deposition blocks in the vicinity.	Construction costs + cost for substitution area + cost for tunnels to adjacent deposition block	
N	Exceptional rock support required. Not accepted by authority The repository cannot be accommodated within intended deposition block There are no deposition blocks in the vicinity.	Construction costs + cost for substitution area + cost for facility in other deposition block	N
O	Unforeseen large zones. The construction may continue. The repository can be accommodated within intended deposition block.	Construction costs + cost for additional tunnels	
P	Unforeseen large zones. The construction may continue. The repository cannot be accommodated within intended deposition block. There are deposition blocks in the vicinity.	Construction costs +cost for additional tunnels + cost for tunnels to adjacent deposition block	
Q	Unforeseen large zones. The construction may continue. The repository cannot be accommodated within intended deposition block. There are no deposition blocks in the vicinity.	Construction costs + cost for additional tunnels + cost for facility in other deposition block	Q

R	Unforeseen large zones. Authorities disapprove capacity of prognosis. Construction extremely delayed.	Construction costs (part) + extreme delay+ possible new site	R
S	Unforeseen large zones. Authorities disapprove capacity of prognosis. New investigations. The repository can be accommodated in the deposition blocks.	Construction costs + investigation costs + delay	
T	Unforeseen large zones. Authorities disapprove capacity of prognosis. New investigations. The repository cannot be accommodated in deposition blocks. There are deposition blocks in the vicinity.	Construction costs + investigation costs + delay + cost for tunnels to adjacent deposition blocks	
U	Unforeseen large zones. Authorities disapprove capacity of prognosis. New investigations. The repository cannot be accommodated in deposition blocks. There are no deposition blocks in the vicinity.	Construction costs + investigation costs + delay + cost for facility in other deposition block	U
V	Unforeseen large zones. Cannot be managed.	Construction stop. New site.	V
W	Major loss of deposition positions. The construction may continue. The repository can be accommodated in the deposition block.	Construction costs + cost for additional tunnels	
X	Major loss of deposition positions. The construction may continue. The repository cannot be accommodated in the deposition block. There are deposition blocks in the vicinity.	Construction costs + cost for additional tunnels+ cost for tunnels to adjacent deposition blocks	
Y	Major loss of deposition positions. The construction may continue. The repository cannot be accommodated in the deposition block. There are no deposition blocks in the vicinity.	Construction costs +cost for additional tunnels + cost for facility in other deposition block	Y
Z	Authorities disapprove capacity of prognosis. Construction extremely delayed.	Construction costs (part) + extreme delay + possible new site	Z
AA	Major loss of deposition positions. Authorities disapprove capacity of prognosis. New investigations will give permission. The repository can be accommodated in the deposition block.	Construction costs + investigation costs + delay	
AB	Major loss of deposition positions. Authorities disapprove capacity of prognosis. New investigations will give permission. The repository cannot be accommodated in the deposition block. There are deposition blocks in the vicinity.	Construction costs + investigating cost + delay +cost for additional	
AC	Major loss of deposition positions. Authorities disapprove capacity of prognosis. New investigations will give permission. The repository cannot be accommodated in the deposition block There are no deposition blocks in the vicinity.	Construction costs + investigation costs + delay + cost for additional tunnels + cost for facility in other deposition block	AC

The result according to Table 10-2 will incur large or ***very large consequences*** accordingly:

H. Major construction problems due to unmanageable water problems. May be that intermittent seepage, e.g. at passages of a zone, becomes so great that it is unmanageable. Less likely. May also be that the general seepage cannot be managed with acceptable grouting products. Major economical, prestige and goodwill loss. Loss of know-how due to quitting personnel. May threaten the whole project.

N. Exceptional support required but cannot be accepted by authority. Should be applied mainly to the deposition blocks where there are requirements for limitation of support. At the same time, the time requirement is less in these parts. It becomes apparent that the repository cannot be accommodated in the intended deposition block (in spite of efforts to change the layout). There are no deposition blocks in the vicinity. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit).

Q. Encountered large (wide and long) zones heavily exceed the predicted number so that it is assumed that the repository cannot be accommodated. The authorities disapprove the prognosis but allow the construction work to continue. It becomes apparent that the repository cannot be accommodated in the intended deposition block. There are no deposition blocks in the vicinity. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit).

R. Encountered large zones heavily exceed the predicted number so that it is assumed that the repository cannot be accommodated. The authorities disapprove the prognosis methodology and require new investigations. The construction becomes extremely delayed. The consequence is that a new site must be found. Major economical, prestige and goodwill loss. Loss of know-how due to quitting personnel. May threaten the whole project.

U. Encountered large zones heavily exceed the predicted number so that it is assumed that the repository cannot be accommodated. The authorities disapprove the prognosis methodology. New and extensive investigations are made and the work is allowed to continue. It becomes apparent that the repository cannot be accommodated in the deposition block. There are no deposition blocks in the vicinity. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit).

V. Non-predicted zones encountered. The zones cannot be managed (in view of constructability or other). The construction is stopped or becomes extremely delayed. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit).

Y. Loss of deposition holes heavily exceeds the prognosis. The authorities allow further construction. It becomes apparent that the repository cannot be accommodated in the intended deposition blocks. There are no deposition blocks in the vicinity. The construction becomes extremely delayed. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit). Major economical, prestige and goodwill loss. Loss of know-how due to quitting personnel.

Z. Loss of deposition holes heavily exceeds the prognosis. The authorities disapprove the prognosis methodology and require new investigations. Major economical, prestige and goodwill loss. Loss of know-how due to quitting personnel. May threaten the whole project.

AC. Loss of deposition holes heavily exceeds the prognosis. The authorities disapprove the prognosis methodology and require new investigations. It becomes apparent that the repository cannot be accommodated in the intended deposition block. There are no deposition blocks in the vicinity. The construction becomes extremely delayed. The consequence is judged to be that parts of the repository must move to a completely new site (new central unit). Major economical, prestige and goodwill loss. Loss of know-how due to quitting personnel.

Fault tree

In order to analyse the events that lead up to the event "Unforeseen major problems" a fault tree has been made, see Figure 10-11. Construction faults have not been analysed as the present analysis only regards the design phase.

Comments to certain parts of the fault tree:

"Design methodologies problems"

- Certain methodologies have been approved/prescribed by SKB in UDP /SKB 2004a/. There is a possibility that faults have passed the SKB control.
- The design engineer can also choose to use his own methodology that may be less appropriate.

"Input data faults"

- The data that are the basis for calculations come from sparse test points. Such lack of data increases the uncertainty especially if there are only vague à-priori information that can be combined with the test information (using Bayes' theoreme).

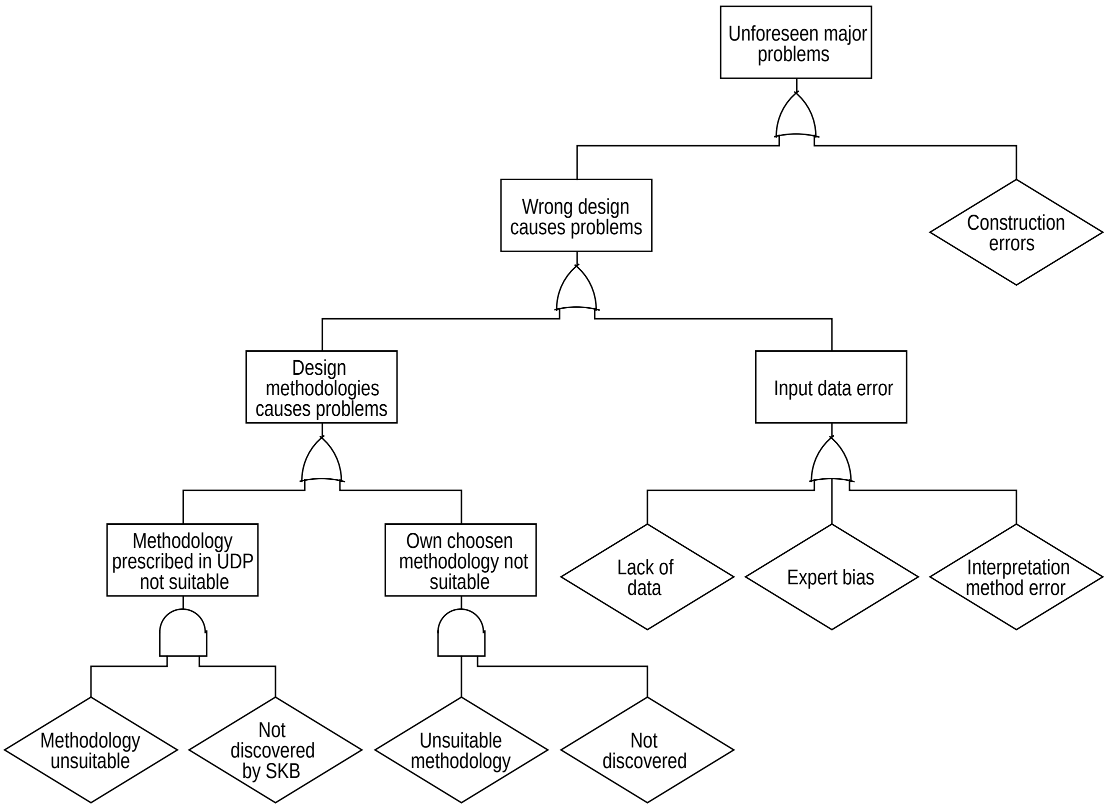


Figure 10-11. *Fault tree showing how the event "Unforeseen major problems" can arise.*

- The use of Bayesian (subjective) probability theory is unavoidable.
  - One danger with this is associated with so called psychological bias of experts that take part in the setting of uncertainties. Often experts unintentionally give over optimistic judgements of uncertainty since they will unconsciously want to show themselves to be experts by appearing to be certain.
  - Another problem that may occur is when some experts with a powerful image can dominate working groups, even outside their own area of expertise.
- Another source of uncertainty is to be found in the stochastic zones in the DFN model. These have a great impact but can for obvious reasons not be validated.

“Interpretation method”

- Data to be used are sometimes interpreted with indirect methods and from small samples. The incurred uncertainty may lead to faults in the design.

10.4.3 Results

Tree analyses with regard to cost and time frames were both superficial and qualitative, however, they allow certain conclusions to be drawn.

Major consequenses

It can be seen from the event tree, Figure 10-10, that the most serious consequences involve the repository being forced to move and be restarted:

Branch H: Major construction problems due to the inability to manage the water problems, e.g. allowable grouting products.

Branch R: It is discovered that the prognosis is very inaccurate with regard to presence of major zones. The authorities do not allow further work.

Branch Z: The same, when loss of canister positions are concerned.

Branch V: Large construction problems since due to lack of knowledge and capacity to handle large unforeseen zones.

Serious consequences, not involving an entire shift in the repository site, are those where accommodation is not possible within the intended deposition block(s) and an additional central unit etc has to be built, although within an acceptable distance so that the same deposition area can be used. This is applicable on branches N, Q, U, Y and AC.

Possible reasons for “Unforeseen major problems”

Causes assumed to create the greatest dangers are:

- “Own chosen methodology inappropriate”
  - May arise mainly through lack of knowledge.
- “Expert bias”
  - This danger should be obvious, especially as procedures to get expert knowledge and also, which is essential, understanding and knowledge of Bayesian statistics are lacking.

10.4.4 Conclusion and recommendations

The performed analyses are very general and a further more thorough review is definitely possible. However, it is considered that the current results are relevant and identify the greatest areas of risk.

Since a risk assessment should not be a kind of review procedure but should work in parallel with the design itself we propose that further assessment should wait until design phase 2 (D2) but should be prepared well in advance. However, it is considered that the current results are relevant and identify the greatest areas of risk.

Most important results from the Design methodology risk assessment:

Essential major consequences:

- Demands that the repository site must be changed due to the inability to manage the water problems with regard to allowed grouting products.
- Problems to get sufficient degree of utilization because the number of major zones has been wrongly predicted.
- Problems to get sufficient degree of utilization due to greater loss of canister positions (thermal properties, vicinity to fracture zones and water).

Essential reasons for design faults:

- Lack of knowledge.
- "Expert bias" and problems to manage subjective probabilities.

Recommendations related to evaluation of Design methodology

Design methodology is a wide expression and design consists of a very large amount of creativity. To draw up a strict framework therefore seems neither possible nor desired. Nevertheless, it is necessary to have the possibility to judge whether different parts of the used/suggested methodology may constitute a danger to the project.

The following measures are suggested:

- The upcoming design is performed explicitly based on probability/risk.
- The preparation of a risk assessment for D2, where the assessment is done in parallel with the design. The preparations shall consist of choice of structure of the assessment, implementation of methodology for quantifying (also collection of damage data), etc.
- As a first step further simulations should be made to illustrate the sensitivity of different factors as well as an attempt to quantify the tree analysis to gain experience in the handling of subjective uncertainties.
- Continued work with grouting methods, grouting products and calculation methods to build experience and allow a flexible approach.
- A compilation covering existing experience and worked examples of the use of numerical methods in probability based design should be made. This should include consideration of spatially dependent variables.
- A compilation covering existing experience and education in the assessing of subjective probabilities should be made including the issue of 'expert bias'.

11 Conclusions

11.1 Outcome from design tasks

11.1.1 Layout

The conclusion of the analyses carried out as prerequisites for the layout studies are summarised in Table 11-1. The analyses concerns identifying a relevant depth, determine spacing between deposition tunnels and between deposition holes, orientate the deposition tunnels in an optimised direction, and finally determine the expected loss of deposition holes based on a number of factors.

The D1 design layout at level –500 m shows sufficient space and volume are available at the site for the anticipated number of 6,000 canisters. The anticipated volume for the Deposition area is approximately 2 million m³ including 65 km of tunnels and deposition holes.

Table 11-1. Summary of prerequisites for the layout studies evaluated in Chapters 3-4.

Prerequisite	Description of prerequisite	Chapter/Section in this report
Location of the repository	The repository should be located within the “Simpevarp interest area” and is further restricted by taking deformation zones ZSMNE005A and ZSMNE024A to mark the western and eastern boundaries respectively.	3
Depth of the repository	The repository should be located to 500 m depth.	3 and 4.5
Distance between deposition holes and between deposition tunnels	The distance between deposition holes should be 7.5 m and between deposition tunnels 40 m.	4.2
Orientation of deposition tunnels	Deposition tunnels should be oriented N015 or N105.	4.3
Loss of deposition holes	Loss of deposition holes is 10% for the layout presented.	4.4

Table 11-2. Summary of key data for the proposed layout.

Key data	Value
Enclosed area for deposition (m^2).	4,120,000
Total length of main tunnels (m).	5,220
Total length of transport tunnels in the deposition area (m)	5,320
Number of deposition tunnels.	213
Total length of deposition tunnels (m).	54,260
Total number of canister positions, including an excess of 10% for loss of deposition holes.	6,600
Number of canister positions, allowing for 10% loss of deposition holes*.	6,000
Excavated volume including central area, ramp and shafts but excluding deposition holes (m^3)	2,368,000

*) Loss of canister positions according to analytical method carried out after the layout was presented, was 13%.

The proposed repository layout involves eleven passages through deformation zones. Of the total passage tunnel length approximately 20 m are judged to pass extremely poor rock, approximately 335 m very poor rock and approximately 60 m poor rock. Of the total passage tunnel length, approximately 125 m have a risk for *high* water inflows, 215 m risk for *medium* water inflows and approximately 75 m risk for *low* water inflows.

The proposed rock support is to a large degree based on recommendations from the Q system and a standard methodology of reducing the tunnel advance distance along with the installation of rock bolts and shotcrete. The proposed grouting activities are focused on a strict program of probing, grouting and control holes of sufficient length using cement-based grouts. Freezing and the installation of a local concrete lining is proposed as an alternative method for passages with potentially high water inflows and very to extremely poor rock conditions /Chang et al. 2005/.

It should be noted that since the geological properties of any particular deformation zone are assumed to be uniform and not vary with depth, the length of the passages most probably are overestimated. Similarly the properties presented for the identified passages are clearly considered to be somewhat conservative concerning both rock mass quality and hydraulic properties.

11.1.2 Hydrogeological results and rock support systems

Seepage and hydrogeology

The largest water inflows are associated with the passage of the deformation zones where hydraulic conductivities may be 1,000 times higher than in the surrounding rock mass. However, the tunnel sections intercepting deformation zones are relatively short. Major reductions in groundwater pressure due to local seepage at the zone passages are not expected. The calculated seepage into the repository is according to Table 11-3, and a prerequisite for this analysis is full access of groundwater from the sea via deformation zones.

Groundwater table drawdown due to the development of the repository is moderate and local. The lateral extent of the depressed groundwater table is essentially limited to the area directly over the tunnels.

Saltwater is drawn into the repository, particularly if grouting is limited to the higher grouting factors, which results in an estimated salinity of 2–4% TDS around the repository, see Figure 7-14.

Table 11-3. Summary of seepage into the repository (analytical analysis).

Grouting level (m/s)	Seepage (l/s)*
No grouting (in situ conductivity)	150
10^{-7}	49
10^{-9}	9

*) The seepage is based on 20 deposition tunnels and all main and transport tunnels open. Ramp tunnel, shaft and central area is not included in the analysis.

Grouting

Future planning should focus on the development of different possible scenarios for the possible rock conditions to be encountered, along with the specific grouting methods and materials to be applied, in order to maximize preparedness prior to the work. In this respect the zone passages require particular attention.

Table 11-4 summarize estimated grout quantities for the repository.

Table 11-4. Summary of grout quantities injected into the rock mass.

Grouting level (m/s)	Grout quantity (m^3)
10^{-7}	Total repository: 3,350–5,380
10^{-9}	Total repository: 15,380–18,615
10^{-7}	Deposition tunnels: 1,590–2,690
10^{-9}	Deposition tunnels: 9,750–11,250

Rock support

A preliminary estimate has been calculated for required support quantities in the repository. Due to uncertainties in the underlying parameters it seems reasonable to assume a variation interval from –15 to +40% around the calculated quantities.

The total quantity of bolts, mesh and shotcrete is presented in Table 11-4.

Table 11-4. Summary of grout quantities injected into the rock mass.

Item	Quantity
Bolts in the compelte facility	45,000 no.s
Bolts in deposition tunnels	20,000 no.s
Mesh in deposition tunnels	46,000 m^2
Shotcrete in facility, excl deposition tunnels	201,000 m^2
Shotcrete, unreinforced	6,420 m^3*
Shotcrete, fibre reinforced	3,215 m^3*
Total cement for rock support	2,645 tonnes
Total cement for tunnel closures	8,105 tonnes

*) Included re-bound from the walls and roof.

11.1.3 Technical risk assessment

The model has been simulated according to the MonteCarlo method for different parameters. The number of simulation steps has been 100,000. The most important result from the presented simulations is that there is a very high probability that 6,000 canisters can be accommodated within the studied area at level –500 m. The calculations show a 99% probability that at least 6,000 canisters can be accommodated within the area.

The main factors with largest impact on the results is the MonteCarlo simulation are:

- Hole distance for Domain A. Variation and scale effects of thermal properties need to be further investigated.

- Loss percentage due to large fractures.
- Dip of east and west borderlines. These control directly the gross area and is therefore of major importance for the area available for deposition holes.

The loss factor, expressed as (1–*k*), states the number of hole positions that for various reasons must be disapproved. The total hole loss is 13% in average but can in extreme cases approach 60%.

Concluding remarks regarding design issues

As a very general result the following important issues have been identified:

- Demands that the repository site must be changed due to the inability to manage the water problems with regard to allowed grouting products.
- Problems to get sufficient degree of utilization because the amount of major zones has been wrongly predicted.
- Problems to get sufficient degree of utilization because the amount of major zones has been wrongly predicted.
- Problems to get sufficient degree of utilization due to great loss of canister positions (thermal properties, vicinity to fracture zones, water).
- Lack of knowledge
- "Expert bias" and problems to manage subjective probabilities

11.2 Critical issues

For the Simpevarp site, one of the more important issues that that will need extra attention is the relatively high number of deformation zones crossing the area. If major zones are not predicted correctly, this may lead to a false degree of utilisation for the site in the design phase. Also, the deposition tunnels need to be placed in several rock blocks with long transport tunnels in between.

Another aspect regarding the major deformation zones is the possibility to pass them at repository depth. Thorough site investigations in order to describe and classify the major zones will be necessary to facilitate safe construction and to minimise risks.

11.3 Recommendations

11.3.1 Feedback to design

The design D1 Simpevarp is based on the site conditions presented in SDM 1.2 Simpevarp. During the time the design work preceded a similar site description task was carried out for the adjacent area in Laxemar. This resulted in a remodelling of several deformation zones in the Simpevarp area, which gave some rather important changes to the base for the layout in the Simpevarp area. Some deformation zones were reclassified from "possible" zones to "high confidence" zones and new deformation zones are added. An additional study of the possibility to accommodate the repository at the Simpevarp location based on the remodelled deformation zones in SDM 1.2 for Laxemar has been carried out, see Appendix D. The result of the study showed that the repository can be accommodated if the eastern border of the available area is the same as the Interest area, but not if the eastern border is the same as deformation zone ZSMNE024A.

In the area there are two existing major underground facilities designed, constructed and owned by SKB – Clab and Äspö HRL. Existing hydrogeological data may be extracted from studying documentation from the construction phase and present status. Finding the optimal orientation of deposition tunnels (design task C3) in the design is largely based on DFN data. The hydrogeological base data for the DFN modelling is from a small amount of data from boreholes. For this study supplementary data from an existing underground facility may be valuable information.

Supplementary engineering geological input from existing facilities is also valuable for design tasks "passages through deformation zones", "estimation of rock grouting need" and "estimation of rock support need".

11.3.2 Feedback to site investigation and modelling

Feedback to site investigation and modelling will be covered in a following report. However, some recommendations concerning further investigations and improvements of the site description model are included here.

The hydraulic conductivity in different directions within the rock mass is one of the parameters used to choose the orientation of deposition tunnels. In SDM Simpevarp 1.2 the anisotropy in the hydraulic conductivity is evaluated by DFN-analyses. The rock mass hydraulic anisotropy evaluated by this manner stands in sharp contrast to the interpretation based on direct measurements in different directions in the spiral tunnel of Äspö HRL /Rhén et al. 1997/. This strongly suggest that the DFN-models should be verified and calibrated to data from existing facilities in the investigation area.

SDM Simpevarp 1.2 has no clear account for possible differences in the rock mass hydraulic conductivity in the plane between the rock domains or with depth. Spatial difference in the rock mass hydraulic conductivity is a factor of great importance for the location of the repository. Therefore, it is recommended that the site investigation is directed towards increased knowledge of possible rock domain dependence and depth dependence of the rock mass hydraulic conductivity.

The geometry of the deformation zones, the length, width and orientation, have an enormous influence on the area available for deposition. Since these parameters are very uncertain for several deformation zones is it recommended that the site investigation should include an increased number of short boreholes that cross critical deformation zones to confirm the assumed zone geometry.

In a future repository the passage of the deformation zones is a critical issue that needs detailed descriptions of the geological conditions of the zones. The geological information available of the deformation zones in SDM Simpevarp 1.2 is not very detailed and the amount of information varies considerably from one zone to another. As the case for the zone geometry, an increased number of short boreholes that cross critical deformation zones to confirm the assumed zone characterisation is recommended.

The present rock mechanics characterisation result in a large amount of tunnelling through stochastic zones of *very poor* rock quality. Since the basis for the interpreted rock mass quality is judged to be somewhat uncertain, is it recommended that rock mass classification is verified by a review and control directly on the drill cores.

Estimation of rock support requirements shall according to UDP be carried out by means of empirical methods /SKB 2004a/. The proposed procedure necessitates that the SDM includes properties that can be used for rock mass classification, or rock mass classification as a direct parameter. The recommendation is to expand the section covering rock mass classification in the succeeding site description models. It is also considered to be appropriate to make use of accounts from rock mass classification in existing facilities in the Simpevarp subarea.

The current report disposition of SDM Simpevarp 1.2, with the scientific subjects in separate chapters, has improved the clearness of the site information noticeably, in comparison with the previous model version. However, it is possible to facilitate the design work even further by developing a special compressed chapter that focuses exclusively on the design parameters required for to performing the design work according to UDP.

11.3.3 Feedback to Safety Assessment

It is preferable that the following issue is addressed within the R & D programme or the safety assessment.

Applying the concept for respect distance as defined in SR-Can Interim /SKB 2004c/ results in large reductions of the area available for deposition, especially for deformation zones with large extensions and a respect distance of 100 m. If possibilities to formulate less restrictive criteria through further studies can be foreseen, such studies should preferably be undertaken

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Appendix A

Rock grouting

A1 Rock blocks

A1.1 Tunnels and caverns

Tunnels in the deposition area, central area and ramp all pass through the rock blocks as well as the deformation zones, see main report Figure 5-2. The rock caverns are confined to the rock blocks according to the proposed layout. The proposed grouting procedure for tunnels and rock caverns within the rock blocks is presented below.

A1.1.1 Grouting level 1

The rock blocks' mean hydraulic conductivity is less than that prescribed by grouting level 1, $K = 10^{-7}$ m/s, however, higher hydraulic conductivities will be encountered locally and where these intercept the tunnels, grouting will be required.

The Simpevarp site description v 1.2 presents the distribution of hydraulic conductivity within the rock blocks for different directions. The analysis indicates that approximately 10% of the rock blocks' volume has a hydraulic conductivity that exceeds $K = 10^{-7}$ m/s. The representative hydraulic conductivity for this rock volume that requires grouting is estimated to be approximately $K_{eff} = 2 \times 10^{-7}$ m/s.

Selective pre-grouting is proposed with 10% of the theoretical number of grouting fans assumed to be drilled and grouted. A grouting fan with follow-up holes is assumed to be the normal case. Grout mix 1 from Table 8-4 is assumed as standard, along with mix 5 used to backfill the drill hole volumes for both probing and grouting.

Grouting sequence:

1. Probe drilling: the number of probe holes is assumed to be 1/3 of the number of bore-holes in the planned scheme.
2. Hydraulic testing (Q) based on established test criteria.
3. Drilling of the scheme holes if Q exceeds test criteria.
4. Hydraulic testing of all holes in the scheme.
5. Grouting of scheme holes based on test results.
6. Control testing and additional drilling and grouting if required.

The proposed form of the grouting scheme is presented in Figure A-1.

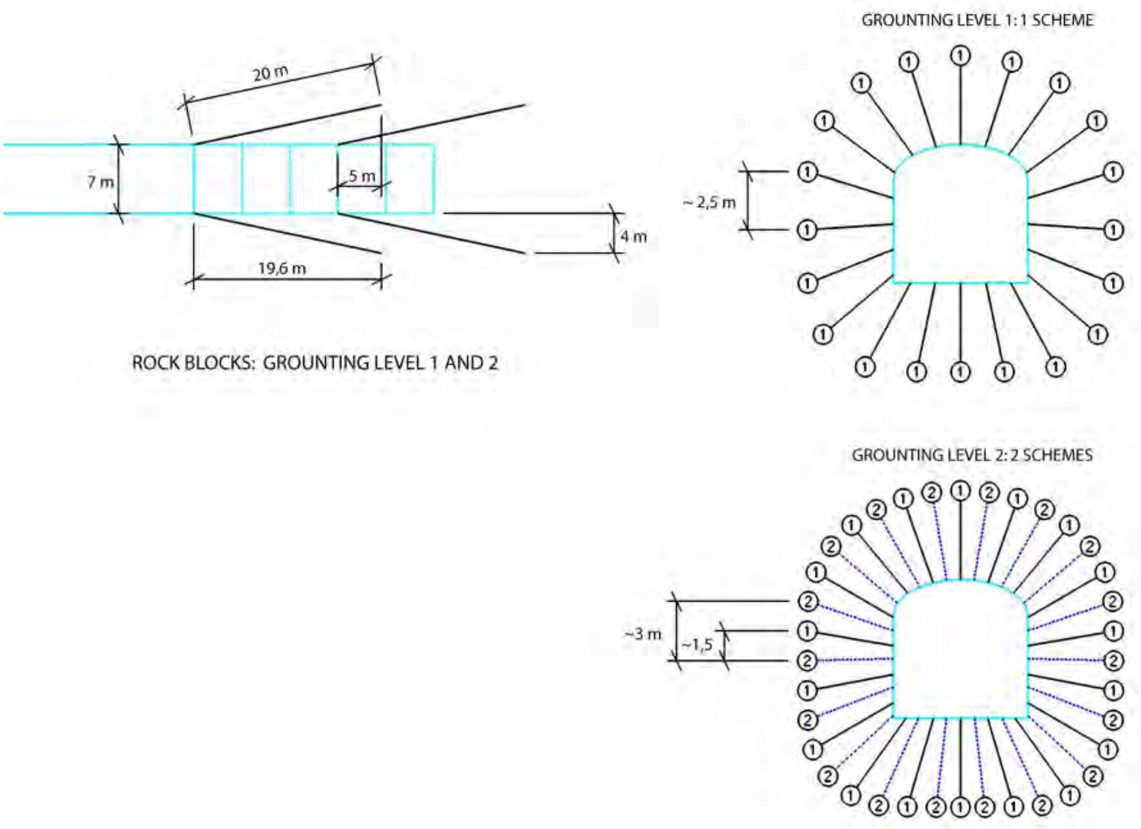

Figure A-1. *Proposed layout of the grouting scheme for transport tunnels located in rock blocks, grouting levels 1 and 2.*

A1.1.2 Grouting level 2

Grouting level 2 involves the establishment of a grout zone with a hydraulic conductivity of 10^{-9} m/s. The hydraulic conductivity of the undisturbed rock mass is of the same order as that prescribed with a mean value of $K_{mean} = 1.3 \times 10^{-9}$ m/s, /SKB 2005a/. Based on the Simpevarp site description v 1.2 it is estimated that 65% of the rock mass has a hydraulic conductivity of approximately 3×10^{-8} m/s and will need to be grouted.

The grouting effectiveness is estimated as approximately 75% and is focused on individual fractures. It is judged that with selective pre-grouting approximately 65% of the theoretical pre-grouting fans will need to be drilled and grouted.

Two stages of grouting with two grout types are assumed as the standard case. The first stage involves grout type 1 and the second stage grout type 2, in accordance with Table 8-4 in the main report. Grout mix 5 is used to backfill the drill hole volumes for both probing and grouting.

Grouting sequence:

1. Probe drilling: the number of probe holes is assumed to be 1/2 of the number of bore-holes in the planned scheme.
2. Hydraulic testing (Q) based on established test criteria.
3. Drilling of the scheme holes (stage 1) if Q exceeds test criteria.
4. Section hydraulic testing of all holes in the scheme.

5. Grouting of scheme holes based on test results. Use of double packer techniques.
6. Control testing and additional stage 2 drilling, testing and grouting as required.

A1.2 Shaft

According to the proposed layout the shafts are confined to the rock blocks and do not intercept the deformation zones. The proposed grouting scheme is preliminary and does not take into consideration any production issues. However, since the grouting holes are likely to be long and drilled in a ring around the shaft, deviations in the borehole alignments need to be monitored. To achieve an acceptable borehole deviation, in the range of 0.5–1%, it will most probably be necessary to drill in stages of 100–200 m and employ high precision drilling techniques.

A1.2.1 Grouting level 1

The same assumptions regarding rock quality and grouting of the tunnels and caverns also apply to the shafts. The rock mass has an estimated mean hydraulic conductivity significantly lower than the level prescribed for grouting level 1. As for the tunnels and caverns it is estimated that approximately 10% of the rock volume has a hydraulic conductivity of 2×10^{-7} m/s or more and will require grouting.

The proposed grout mix for stage 1 is mix 1, with mix 2 used for stage 2, in accordance with Table 8-4 in the main report. Grout mix 5 is used to backfill the drill hole volumes for both probing and grouting.

Grouting sequence:

1. Probe holes are drilled in a ring outside the planned shaft position.
2. Hydraulic testing (*Q*) based on established test criteria in 20 m intervals.
3. Grouting in 20 m intervals (Stage 1).
4. Drilling of secondary grouting holes.
5. Grouting (Stage 2).
6. Control hole drilling, testing and further grouting as required.

The proposed grouting scheme drill hole layout is presented in Figure A-2.

A1.2.2 Grouting level 2

The same assumptions regarding rock quality and grouting of the tunnels and caverns also apply to the shafts. The rock mass has an estimated mean hydraulic conductivity that is of the same magnitude as prescribed for grouting level 2. However, ca 65% of the rock volume is assumed to have a hydraulic conductivity of 5×10^{-8} m/s or more and will require grouting. Grouting will be focused on individual fractures with an estimated grouting effectiveness of 75%.

The proposed grout mix for stage 1 is mix 1, with mix 3 being used for stage 2, in accordance with main report Table 8-4. Grout mix 5 is used to backfill the drill hole volumes for both probing and grouting.

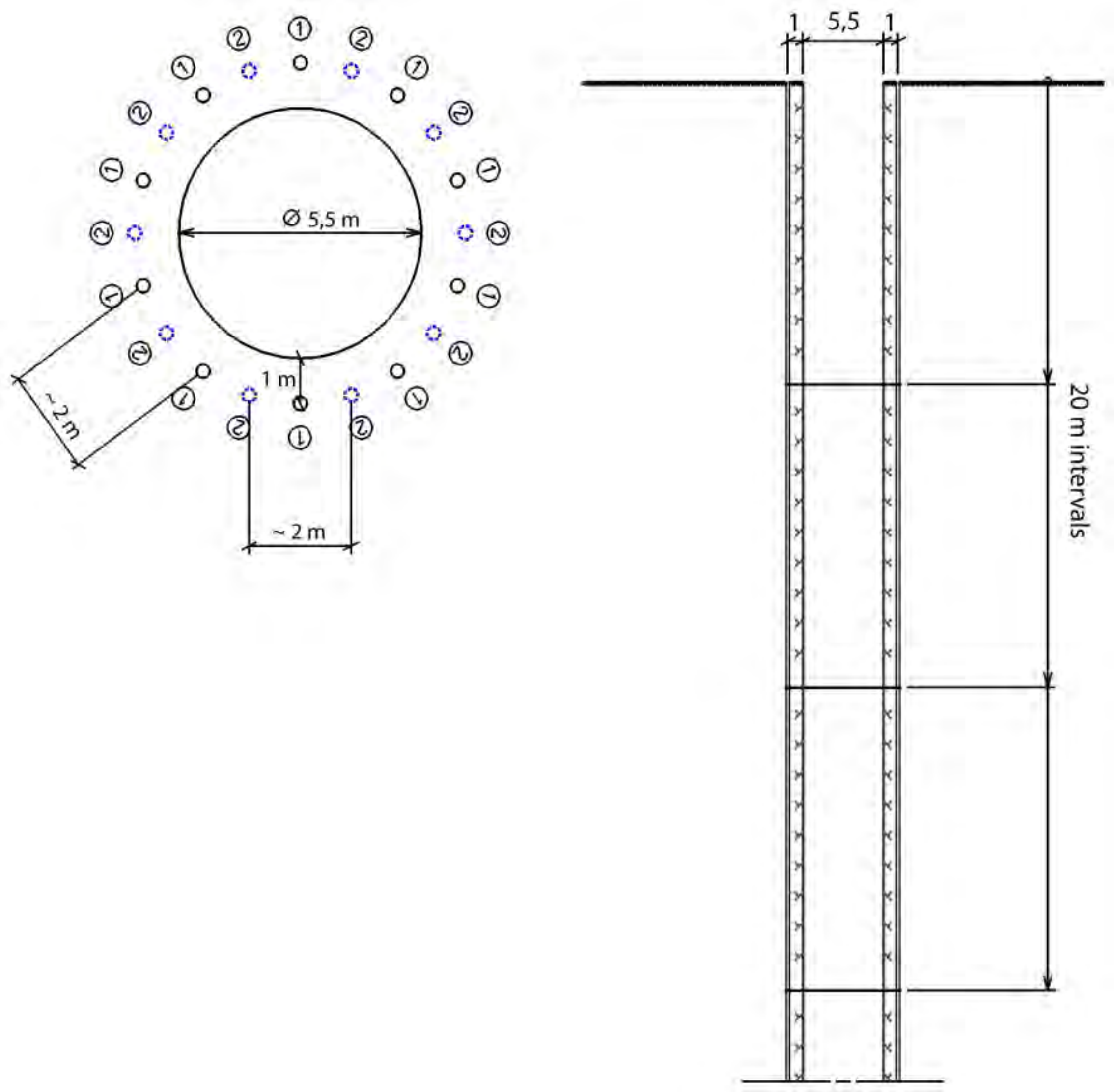

Figure A-2. *Proposed grouting scheme drill hole layout for shafts in rock blocks, grouting level 1.*

Grouting sequence:

1. Probe holes are drilled in a ring outside the planned shaft position.
2. Hydraulic testing (*Q*) based on established test criteria in 5 m intervals.
3. Grouting in 5 m intervals (Stage 1).
4. Drilling of secondary grouting holes.
5. Grouting (Stage 2).
6. Control hole drilling, testing and further grouting as required.

The proposed grouting scheme drill hole layout is presented in Figure A-3.

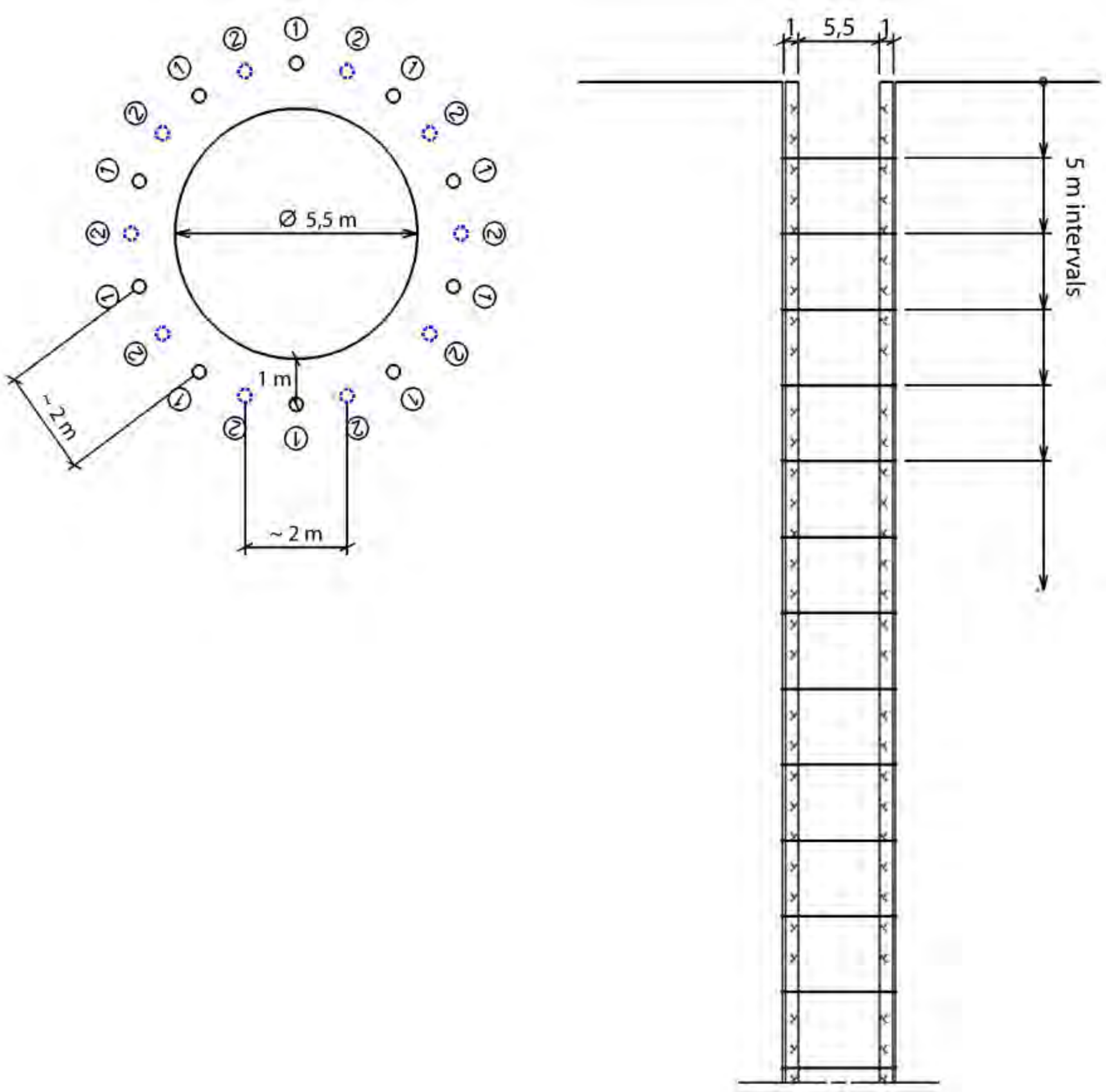

Figure A-3. *Proposed grouting scheme drill hole layout for shafts in rock blocks, grouting level 2.*

A2 Deformation zones

A2.1 Tunnels

The proposed layout requires the transport tunnels to pass through the deformation zones in 11 locations, see main report Figure 6-1. The zones have dramatically higher hydraulic conductivities and seepage levels are expected to be significantly higher than within the rock blocks, see Table 8-3 in the main report. The total seepage for all 11 passages is estimated to be in the order of 140 l/s if left ungrouted, 50 l/s if grouted to grouting level 1 and 1 l/s if grouted to level 2.

The grouting methodology outlined below assumes a simplistic approach for all the passages and is only intended to allow a preliminary estimation of grouting work and quantities required. During the detailed design stage alternative approaches will be considered that are developed for specific zone passages. However, it can be assumed that a flexible approach will be needed including preparation for the use of a varied selection of grout mixes, drilling patterns and hole lengths, specifically adapted to the fracture orientations and rock conditions encountered, with the ability to deal with a combination of both high water pressures and high hydraulic conductivities.

To achieve the seepage requirements in passages with high water inflow, the extension of the grouted rock volume around the tunnel must be larger as compared with tunnel sections in rock blocks. This, in combination with a stiff grout, requires a larger borehole angle in the proposed drill hole layout for passages compared to tunnel sections in rock blocks.

A2.1.1 Grouting level 1

The grouting effectiveness is estimated to be 30–80%, which means that normal pre-grouting should result in the hydraulic conductivity prescribed for grouting level 1 of $K = 10^{-7}$ m/s.

Three different methods are proposed A, B and C, related to the different zone hydraulic conductivities and expected degree of difficulty.

Method A

Grouting of passages P1H, P5H and P11H, with hydraulic conductivities lower than grouting level 1, is performed in accordance with the method previously described for tunnels and caverns in rock blocks.

Method B

It is proposed that grouting of passages P2L, P3L, P4L, P6H, P9L and P10H is performed with continuous pre-grouting. The zones are judged to be moderately to strongly water-bearing.

Two to three stages of grouting, involving two grout mixes are assumed as standard. Grout type 1 is a stiff grout with the addition of an accelerator. Grout type 2 has better penetration properties. A further cavity filling, standard grout would also be used.

Grouting sequence:

1. Drilling of grouting scheme holes for stage 1 as well as face drilling.
2. Hydraulic testing of every drill hole in the scheme to determine the grouting sequence.
3. Grouting of all scheme holes.
4. Drilling of grouting scheme holes for stage 2.
5. Hydraulic testing to determine grouting needs and sequence.
6. Grouting of drill holes based on test results.
7. Control-hole drilling and further grouting as necessary.

The proposed grouting scheme drill hole layout is presented in Figure A-4.

Method C

Continuous pre-grouting is assumed for passages P7H and P8H with the highest hydraulic conductivities. Around 4 to 5 stages of grouting, using two different grout mixes are assumed as standard. Grout type 1 is a stiff grout with the addition of an accelerator. Grout type 2 has better penetration properties. A further cavity filling, standard grout would also be used.

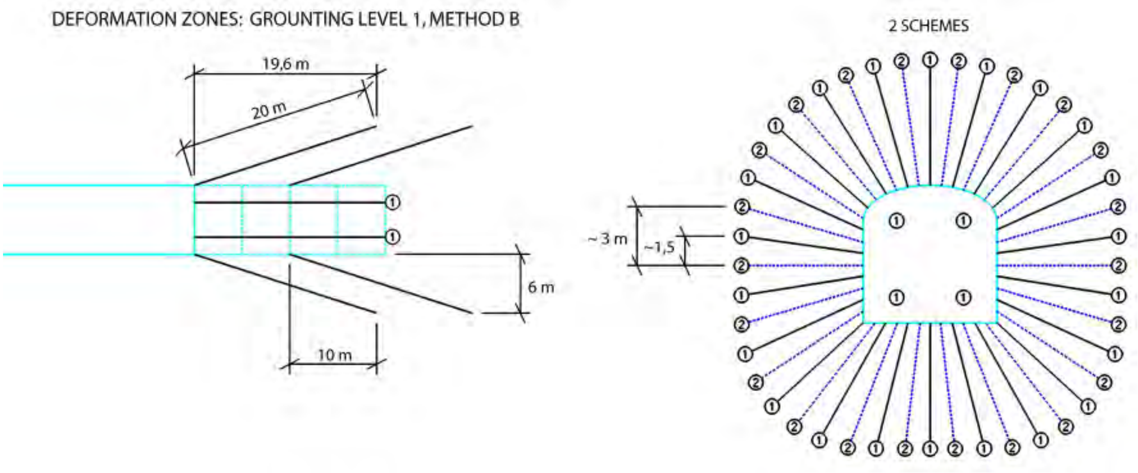

__Figure A-4.__ Proposed grouting scheme drill hole layout for transport tunnel-deformation zone intercepts, in accordance with method B and grouting level 1.

Grouting sequence:

1. Drilling of grouting scheme holes for stage 1 as well as face drilling.
2. Hydraulic testing of every drill hole in the scheme to determine the grouting sequence.
3. Grouting of all scheme holes.
4. Drilling of grouting scheme for stage 2.
5. Hydraulic testing to determine grouting needs and sequence.
6. Grouting of drill holes based on test results.
7. Drilling of grouting scheme for stage 3 and continue with steps 5 and 6.
8. Drilling of grouting scheme for stage 4 and continue with steps 5 and 6.
9. Controlhole drilling and further grouting as necessary.

The proposed grouting scheme drill hole layout is presented in Figure A-5.

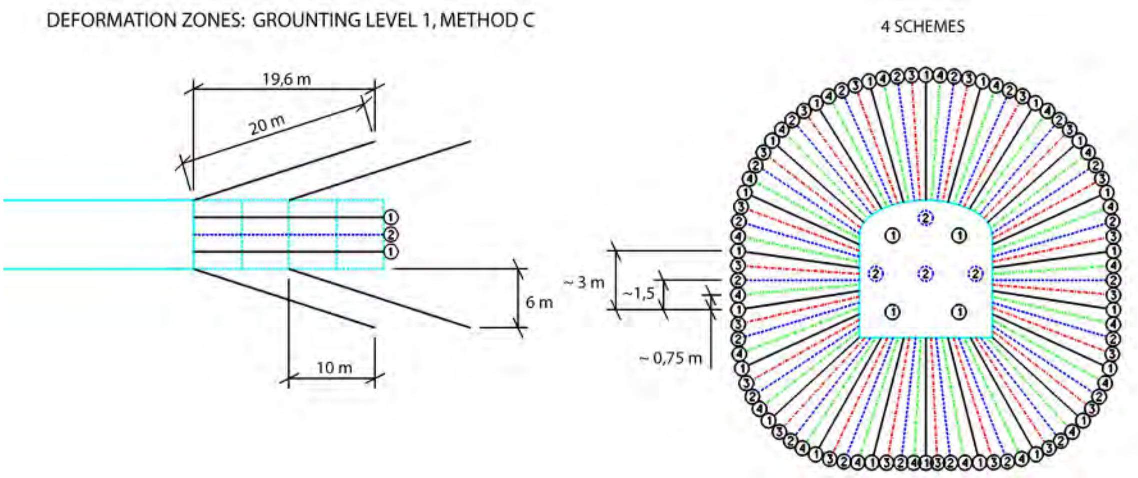

__Figure A-5.__ Proposed grouting scheme drill hole layout for transport tunnel-deformation zone intercepts, in accordance with method C and grouting level 1. Four stages of grouting presented.

A2.1.2 Grouting level 2

The 11 passages are divided into two groups, which are grouted using two different methods. Method A is used for the passages with lower hydraulic conductivities, P1H, P5H and P11H. It is considered realistic to achieve a grouting effectiveness of 86–92% resulting in the prescribed hydraulic conductivity of $K = 10^{-9}$ m/s for grouting level 2.

The other 8 passages with higher hydraulic conductivities require a grouting effectiveness of 98.7–99.8% in order to reach the stipulated level. Method B is used for these passages and it is judged to be very difficult to reach such a level of effectiveness with cement grouts.

Method A

Continuous pre-grouting for passages P1H, P5H and P11H. Three grouting stages are assumed. It is judged that to reach the required grouting level 2 will be time consuming and alternatives to cement grouts should be considered.

Three grout types are recommended. Grout type 1 is a stiff grout with the addition of an accelerator. Grout type 2 has good penetration properties. Grout type 3 has excellent penetration properties. A further cavity filling, standard grout would also be used to backfill tight holes.

Grouting sequence:

1. Drilling of grouting scheme holes for stage 1.
2. Stepped hydraulic testing of every drill hole in the scheme to characterize the fractures and grouting sequence.
3. Grouting of drill holes based on test results. Use of double packers as required.
4. Drilling of grouting scheme holes for stage 2 and continue with steps 2 and 3.
5. Drilling of grouting scheme holes for stage 3 and continue with steps 2 and 3.
9. Controlhole drilling and further grouting as necessary.

Method B

Continuous pre-grouting for passages P2L, P3L, P4L, P6H, P7H, P8H, P9L and P10H. It is judged that to reach the required grouting level 2 will be time consuming and alternatives to cement grouts as well as alternative lining-based methods should be considered. Five to six grouting stages are assumed.

Three grout types are recommended. Grout type 1 is a stiff grout with the addition of an accelerator. Grout type 2 has good penetration properties. Grout type 3 has excellent penetration properties. A further cavity filling, standard grout would also be used to backfill tight holes.

Grouting method:

1. Drilling of grouting scheme holes for stage 1 including face drilling.
2. Hydraulic testing to determine grouting needs and sequence.
3. Grouting of all scheme holes.
4. Drilling of grouting scheme holes for stage 2 including face drilling and continue with steps 2 and 3.
5. Drilling of grouting scheme holes for stage 3.

6. Hydraulic testing to determine grouting needs and sequence.
7. Grouting of drill holes based on test results.
8. Drilling of grouting scheme for stage 4 and continue with steps 6 and 7.
9. Drilling of grouting scheme for stage 5.
10. Stepped hydraulic testing of every drill hole in the scheme to characterize the fractures and grouting sequence.
11. Grouting of drill holes based on test results. Use of double packers as required.
12. Control-hole drilling and further grouting as necessary.

A3 Estimated drilling quantities

Table A-1. Summary of grout hole drilling quantities for the central area and ramp.

	Grouted boreholes (m)	Ungrouted boreho-les (plugged) (m)	Probe holes (plugged) (m)	Control holes (plugged) (m)
Rock blocks				
Ramp				
– Grouting level 1	4,410	8,820	37,627	1,330
– Grouting level 2	44,838	101,248	19,510	11,812
Transport tunnel				
– Grouting level 1	1,680	3,360	14,484	507
– Grouting level 2	17,410	39,312	7,604	4,586
Skip ramp				
– Grouting level 1	784	1,568	6,394	236
– Grouting level 2	7,465	16,856	3,264	1,967
Other tunnels				
– Grouting level 1	154	308	1,111	46
– Grouting level 2	1,240	2,800	589	327
Rock caverns				
– Grouting level 1	742	1,484	5,619	224
– Grouting level 2	6,200	14,000	2,723	1,862
Shaft				
– Grouting level 1	6,088	28,157		
– Grouting level 2	19,405	14,839		
Silo				
– Grouting level 1	349	388		
– Grouting level 2	247	189		
Deformation zones				
Ramp				
– Grouting level 1				
*Method A	3,024	6,048		931
*Method B	15,580	2,280		3,230
– Grouting level 2				
*Method A	28,860	17,760		6,290
*Method B	36,860	6,840		3,230
Transport tunnel				
– Grouting level 1				
*Method A	1,386	2,772		417
*Method B	4,920	720		1,020
– Grouting level 2				
*Method A	12,480	7,680		2,720
*Method B	11,640	2,160		1,020

	Grouted boreholes (m)	Ungrouted boreholes (plugged) (m)	Probe holes (plugged) (m)	Control holes (plugged) (m)
Skip ramp				
– Grouting level 1				
*Method A	224	448		1,020
*Method B	1,344	192		272
– Grouting level 2				
*Method A	1,872	1,152		408
*Method B	3,200	576		272

A4 Estimated grout volume based on an empirical approach

Table A-2. Summary of estimated grout quantities within rock blocks.

Repository part	Grouting level 1					Grouting level 2				
	Grouted volume (m³)	Plugged volume (m³)	Silica fume (tonne)	SP40 (tonne)	Micro cement OPC (tonne)	Grouted volume (m³)	Plugged volume (m³)	Silica fume (tonne)	SP40 (tonne)	Micro cement OPC (tonne)
Deposition tunnel	912	991	797	20.9	569	8,125	2,752	4,557	120	3,252
Main tunnel	108	117	94	2.5	67	976	331	548	14	391
Transport tunnel	92	100	80	2.1	57	828	293	470	12	335
Ramp	88	96	77	2.0	55	781	265	438	12	313
Rock caverns	18	18	15	0.4	11	130	45	73	2	52
Tunnels central area	49	53	43	1.1	30	433	147	243	6	173
Shaft	90	57	62	1.6	44	370	30	168	4	120
Total	**1,357**	**1,432**	**1,169**	**31**	**834**	**11,643**	**3,863**	**6,497**	**171**	**4,636**

Note. "Plugged volume" refers to the volume of the actual drilled holes.

Table A-3. Summary of estimated grout quantities for passage of deformation zones, in accordance with grouting Method A.

Repository part	Grouting level 1					Grouting level 2				
	Grouted volume (m³)	Plugged volume (m³)	Silica fume (m³)	SP40 (tonne)	Micro cement OPC (tonne)	Grouted volume (m³)	Plugged volume (m³)	Silica fume (tonne)	SP40 (tonne)	Micro cement OPC (tonne)
Transport tunnel	8	2	4	0.1	3	39	4	18	0.5	13
Ramp	60	14	31	0.8	22	433	48	202	5.3	144
Tunnels central area	84	5	37	1.0	27	215	24	100	2.6	71
Total	**152**	**21**	**72**	**2**	**52**	**687**	**76**	**320**	**8**	**228**

Note. "Plugged volume" refers to the volume of the actual drilled holes.

Table A-4. Summary of estimated grout quantities for passage of deformation zones, in accordance with grouting Method B.

Repository part	Grouting level 1 Grouted volume (m³)	Plugged volume (m³)	Silica fume (m³)	SP40 (tonne)	Micro cement PC (tonne)	Grouting level 2 Grouted volume (m³)	Plugged volume (m³)	Silica fume (tonne)	SP40 (tonne)	Micro cement OPC (tonne)
Transport tunnel	240	10	105	2.8	75	931	29	402	10.6	287
Ramp	279	11	122	3.2	87	652	20	282	7.4	201
Tunnels central area	112	4	49	1.3	35	263	8	114	3.0	81
Total	**631**	**25**	**275**	**7**	**196**	**1,846**	**57**	**797**	**21**	**569**

Note. “Plugged volume” refers to the volume of the actual drilled holes.

Table A-5. Summary of estimated grout quantities for passage of deformation zones in accordance with grouting Method C.

Repository part	Grouting level 1 Grouted volume (m³)	Plugged volume (m³)	Silica fume (m³)	SP40 (tonne)	Micro cement OPC (tonne)
Transport tunnel deposition area	330	7	141	3.7	101

Note. “Plugged volume” refers to the volume of the actual drilled holes.

A5 Estimated grout quantities that remain in the rock mass after excavation

Table A-6. Overall summary of estimated grout quantities (max/min) that remain in the rock mass after excavation. Quoted volumes are per excavation type lying within rock blocks. They do not include the volumes of the actual drilled holes.

Excavation type Deposition area	Estimated ‘permanent’ grout volume (m³) Grouting level 1	Grouting level 2
Deposition tunnel	522–1,479	6,090–7,395
Transport tunnel	49–146	608–810
Main tunnel	53–175	608–836
Central area		
Ramp	52–155	602–688
Cavern 1, 2 + main	10–24	81–122
Transport + skip	24–73	324–365
Shaft 1, 2 and central silo	58–93	233–426
Total	**768–2,144**	**8,545–10,641**

Table A-7. Overall summary of estimated grout quantities (max/min) that remain in the rock mass after excavation. Quoted volumes are per excavation type for where they intercept deformation zones. They do not include the volumes of the actual drilled holes.

Passage/tunnel type Deposition area	Estimated 'permanent' grout volume (m^3) Grouting level 1	 Grouting level 2
P1H, P5H, P11H	4.6–9.2	9.2–32
P2L, P3L, P4L, P6H, P9L, P10H	184–276	230–322
P7H, P8H	276–368	322–414
Central area		
Ramp	276–414	460–828
Transport + skip	138–184	184–276
Total	**879–1,251**	**1,205–1,872**

Appendix B

Rock support

B1 Rock support drawings

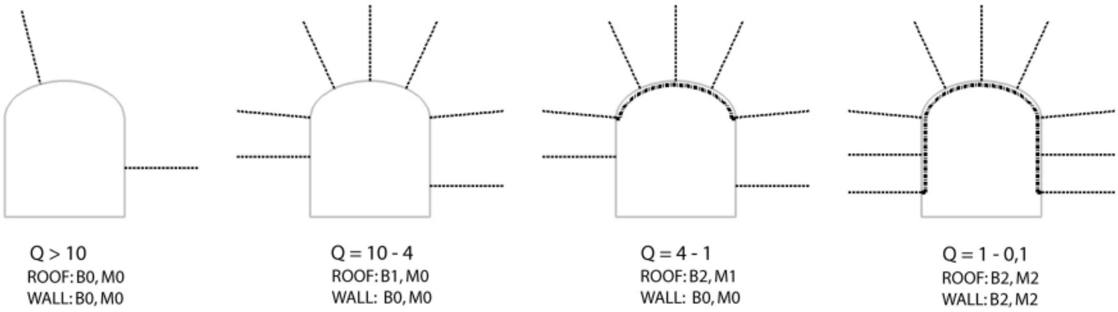

Figure B-1. *Principle drawing of support for deposition tunnels.*

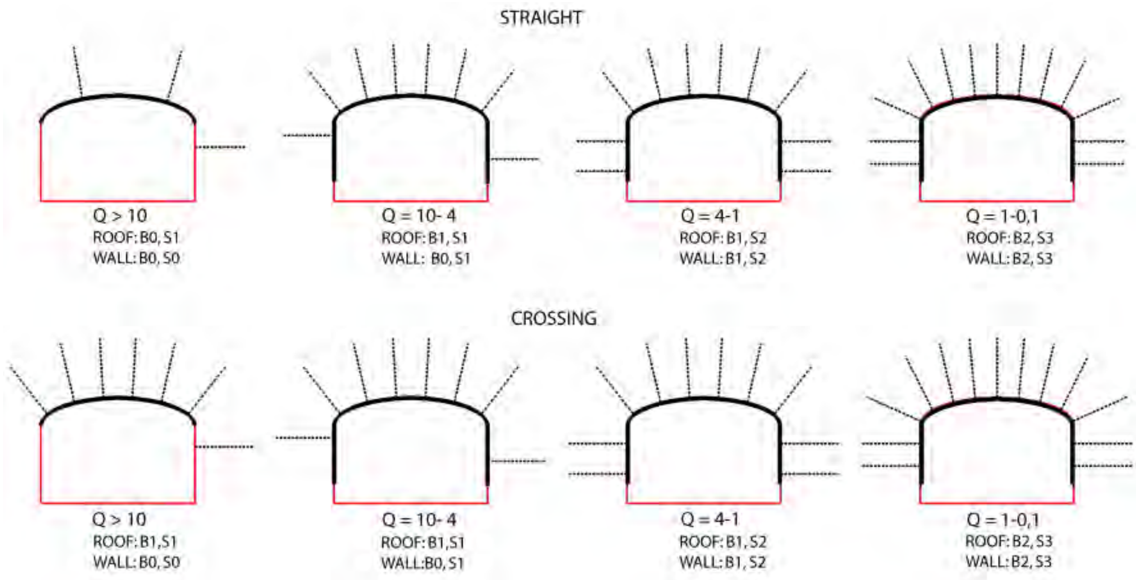

Figure B-2. *Principle drawing of support for main tunnels.*

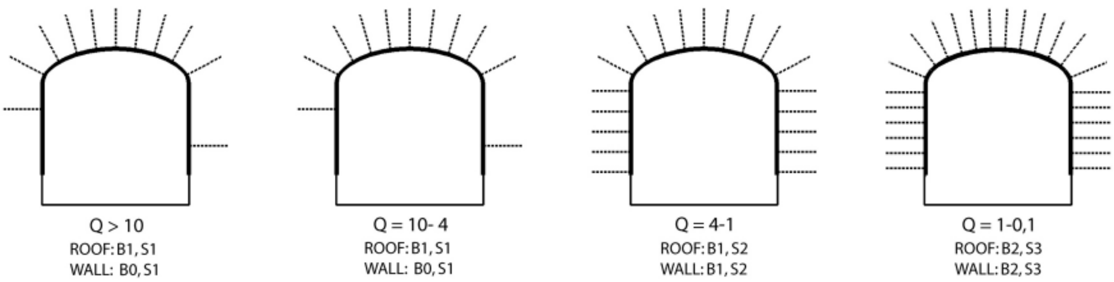

Figure B-3. *Principle drawing of support for rock halls.*

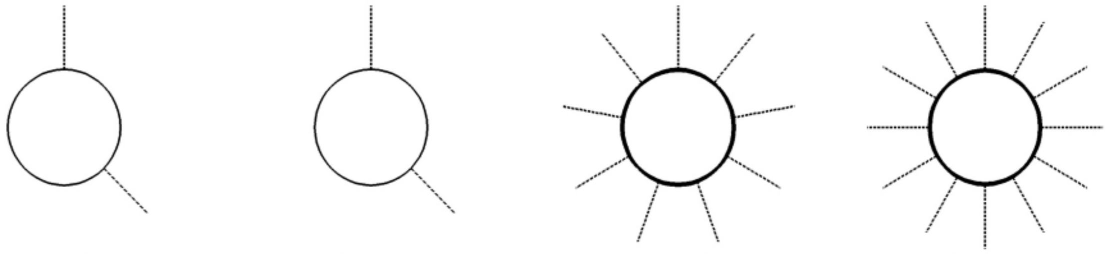

Figure B-4. *Principle drawing of support for elevator shafts.*

B2 Calculated quantities

Table B-1. List of quantities for deposition tunnels.

Support element	Unit	Domain A	Domain B	Domain C	Sum
Swellex number	no.	13,976	4,466	2,826	**21,268**
Swellex weight	tonne	34	11	7	**52**
Mesh area	m^2	29,368	10,364	6,267	**45,999**
Mesh weight	tonne	50	18	11	**79**
Strap length	m	21,121	7,074	2,702	**30,897**
Strap weight	tonne	30	10	4	**44**
Washer number	no.	9,044	3,107	1,529	**13,680**
Washer, half ball	tonne	8.4	2.9	1.4	**13**
Total weight	tonne	122	42	23	**188**

Table B-2. List of quantities for tunnel closures.

Element	Unit	Prerequisites	Total quantity
Concrete plug number	no.	213	–
Tunnel area	m^2	25	**–**
Plug length	m	5	**–**
Concrete volume	m^3	–	**26,487**
Cement content	kg/m^3	306	**–**
Cement	tonne	–	8,105
Silca fume content	kg/m^3	204	**–**
Silca fume	tonne	–	**5,403**
Ballast content	kg/m^3	1,500	**–**
Ballast	tonne	–	**39,730**
SP 40 content	kg/m^3	7	–
SP 40	tonne	–	**185**
Reinforcement content	kg/m^3	78.5	
Reinforcement	tonne	–	**2,079**

Table B-3. List of quantities for main tunnels.

Support element	Unit	Straight stretch A	B	C	Junction A	B	C	Sum
Rock bolt								
Bolt number	no.	1,699	990	160	1,817	1,255	133	**6,054**
Bolt weight	tonne	23	13	2	28	19	2	**87**
Washer, half ball	tonne	0.9	0.5	0.1	0.4	0.3	0.0	**2**
Bolt grout								
Volume	m³	14.6	8.5	1.4	17.9	12.4	1.3	**56**
Cement	tonne	8.7	5.1	0.8	10.7	7.4	0.8	**33**
Silica fume	tonne	3.7	2.2	0.4	4.6	3.2	0.3	**14**
SP 40	kg	29	17	3	36	25	3	**112**
Shotcrete								
Supported area	m²	17,054	5,860	1,046	7,839	6,112	662	**38,574**
Volume, unreinforced	m³	448	134	26	213	161	18	**1,000**
Volume, reinforced	m³	265	137	17	104	91	7	**620**
Cement	tonne	218	83	13	97	77	8	**496**
Silica fume	tonne	145	55	9	65	51	5	**331**
Ballast	tonne	1,069	406	65	475	378	37	**2,430**
SP 40	tonne	5.0	1.9	0.3	2.2	1.8	0.2	**11**
Fibre	tonne	18.5	9.6	1.2	7.3	6.4	0.5	**43**

Table B-4. List of quantities for transport tunnels.

Support element	Unit	Straight stretch A	B	C	Curve A	B	C	Passage	Sum
Rock bolt									
Bolt number	no.	1,993	896	271	290	107	271	2,988	**6,815**
Bolt weight	tonne	23.0	10.4	3.1	3.9	1.4	3.6	34.5	**80**
Washer, half ball	tonne	1.5	0.7	0.2	0.1	0.1	0.2	2.8	**6**
Bolt grout									
Volume	m³	14.7	6.6	2.0	2.5	0.9	2.3	22.1	**51**
Cement	tonne	8.8	3.9	1.2	1.5	0.6	1.4	13.2	**31**
Silica fume	tonne	3.8	1.7	0.5	0.6	0.2	0.6	5.6	**13**
SP 40	kg	29	13	4	5	2	5	44	**102**
Shotcrete									
Supported area	m²	27,974	10,828	4,336	2,830	1,427	4,336	6,809	**58,539**
Volume, unreinforced	m³	867	334	140	100	44	67	0	**1,552**
Volume, reinforced	m³	84	36	3	15	4	3	838	**982**
Cement	tonne	291	113	44	35	15	21	256	**775**
Silica fume	tonne	194	75	29	23	10	14	171	**517**
Ballast	tonne	1,427	555	213	173	73	104	1,257	**3,801**
SP 40	tonne	6.7	2.6	1.0	0.8	0.3	0.5	5.9	**18**
Fibre	tonne	5.9	2.5	0.2	1.1	0.3	0.2	58.6	**69**

Table B-5. List of quantities for ramp.

Support element	Unit	Straight stretch A	Straight stretch B	Curve A	Curve B	Passage	Connection	Sum
Rock bolt								
Bolt number	no.	1,016	1,497	223	329	2,260	97	**5,422**
Bolt weight	tonne	11.7	17.3	3.0	4.4	26.1	1.3	**64**
Washer, half ball	tonne	0.7	1.1	0.2	0.2	2.1	0.1	**4**
Bolt grout								
Volume	m³	7.5	11.1	1.9	2.8	16.7	0.8	**41**
Cement	tonne	4.5	6.6	1.1	1.7	10.0	0.5	**24**
Silica fume	tonne	1.9	2.8	0.5	0.7	4.3	0.2	**10**
SP 40	kg	15	22	4	6	33	2	**82**
Shotcrete								
Supported area	m²	18,936	24,663	4,316	5,595	5,962	1,723	**61,196**
Volume, unreinforced	m³	588	762	135	174	0	54	**1,712**
Volume, reinforced	m³	55	79	11	17	645	5	**811**
Cement	tonne	197	257	45	58	197	18	772
Silica fume	tonne	131	172	30	39	132	12	**515**
Ballast	tonne	964	1,262	219	285	967	88	**3,785**
SP 40	tonne	4.5	5.9	1.0	1.3	4.5	0.4	**18**
Fibre	tonne	3.8	5.5	0.8	1.2	45.1	0.3	**57**

Table B-6. List of quantities for rock loading stations and tunnels in central area.

Support element	Unit	Caverns	Rock station	Other tunnels	Sum
Rock bolts					
Bolt number	no.	2,213	226	2,866	**5,305**
Bolt weight	tonne	34	3	33	**71**
Washer, half ball	tonne	0.4	0.1	2.1	**3**
Bolt grout					
Volume	m³	21.8	2.0	21.3	**45**
Cement	tonne	13.0	1.2	12.7	**27**
Silica fume	tonne	5.6	0.5	5.4	**12**
SP 40	kg	44	4	43	**90**
Shotcrete					
Supported area	m²	12,892	975	17,374	**31,242**
Volume, unreinforced	m³	373	22	383	**778**
Volume, reinforced	m³	114	11	588	**713**
Cement	tonne	149	11	297	**457**
Silica fume	tonne	99	7	198	304
Ballast	tonne	731	52	1,456	**2,238**
SP 40	tonne	3.4	0.2	6.8	**10**
Fibre	tonne	8.0	0.9	41.1	**50**

Table B-7. List of quantities for shafts.

Support element	Unit	Elevator shaft A	Elevator shaft B	Skip shaft A	Skip shaft B	Sum
Rock bolts						
Bolt number	no.	358	23	390	0	**771**
Bolt weight	tonne	4.1	0.3	4.5	0	**9**
Washer, half ball	tonne	0.3	0.0	0.3	0	**0.6**
Bolt grout						
Volume	m³	2.6	0.2	2.9	0	**6**
Cement	tonne	1.6	0.1	1.7	0	**3**
Silica fume	tonne	0.7	0.0	0.7	0	**1**
SP 40	kg	5.3	0.3	5.8	0	**11**
Shotcrete						
Supported area	m²	1,028	69	10,229	0	**11,326**
Volume, unreinforced	m³	23.4	1.6	25.5	0	**50**
Volume, reinforced	m³	14.9	1.0	16.3	0	**32**
Cement	tonne	11.7	0.8	12.8	0	**25**
Silica fume	tonne	7.8	0.5	8.5	0	**17**
Ballast	tonne	57.5	3.8	62.6	0	**124**
SP 40	tonne	0.3	0.0	0.3	0	**1**
Fibre	tonne	1.1	0.1	1.2		**2**

B3 Sensitivity analysis

Table B-8. List of quantities for deposition tunnels for doubled bolt density at selective bolting (Roof 30/Wall 60 m²/bolt). Only the bolt quantity is included in the table as only this is changed in comparison with the chapter 9 reference estimate.

Support element	Unit	Reference	Change vs reference Quantity	Change vs reference Percent
Swellex number	no	21,268	28,855	36
Swellex weight	tonne	52	71	36

Table B-9. List of quantities for deposition tunnels at 5% reduction resp. 10% increase of stochastic deformation zones.

Support element	Unit	Reference	Change vs reference Quantity –5%	Quantity +10%	Percent –5%	Percent +10%
Swellex number	no	21,268	18,510	26,784	–13	26
Swellex weight	tonne	52	46	66	–13	26
Mesh area	m²	46,000	36,471	65,057	–21	41
Mesh weight	tonne	78	62	111	–21	41

Table B-10. List of quantities for main tunnels and transport tunnels in the deposition area for doubled bolt density at selective bolting (Roof 40/Wall 80 m²/bolt). Only the bolt quantity and grouting quantity are included in the table as only this is changed in comparison with the chapter 9 reference estimate.

Support element	Unit	Reference	Change vs reference	
			Quantity	Percent
Rock bolt number	no	12,870	13,962	8
Rock bolt weight	tonne	168	182	8
Bolt grout volume	m³	107	116	8

Table B-11. List of quantities for main tunnels and transport tunnels in the deposition area at 5% reduction respective 10% increase of stochastic deformation zones.

Support element	Unit	Reference	Change vs reference			
			Quantity		Percent	
			–5%	+10%	–5%	+10%
Rock bolt number	no	12,870	11,865	14,880	–8	16
Rock bolt weight	tonne	168	155	193	–8	16
Bolt grout volume	m³	107	99	124	–8	16
Shotcrete area	m²	97,113	95,007	101,325	–2	4
Volume, unreinforced	m³	2,552	2,546	2,564	–0.2	0.5
Volume, reinforced	m³	1,602	1,439	1,927	–10	20

Appendix C

Technical risk assessment

C1 Estimation of available deposition area

C.1.1 Simplified surface geometry

The total available area for deposition at ground level (see the shaded area in Figure C-1) is measured as 7.61 km^2. The geometry of the area has for practical reasons been simplified in the model to a 4-sided polygon with the same area.

The geometry of the simplified area is shown in Figure C-2. The borderline of the area to the north was initially set as the interest area but is here replaced by deformation zone ZSMEW038A. The size of the area has been adjusted to give exactly the same size as the actual interest area. Coordinates for corner points are given in Table C-1.

Table C-1. Coordinates used for the simplified area (size of area = 7.61 km^2).

Point	X-coordinate (km)	Y-coordinate (km)
X_1, Y_1	0.00	0.00
X_2, Y_2	2.60	0.00
X_3, Y_3	2.60	3.90
X_4, Y_4	0.00	1.96

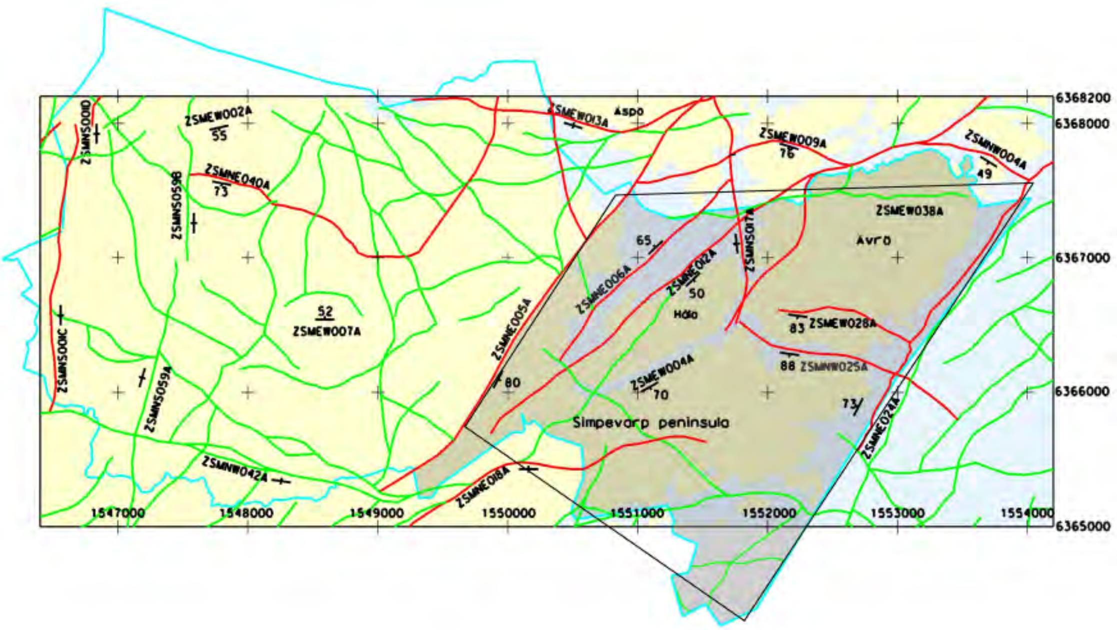

Figure C-1. Area for deposition (Simpevarp) and the simplified area used in the model (Ground level).

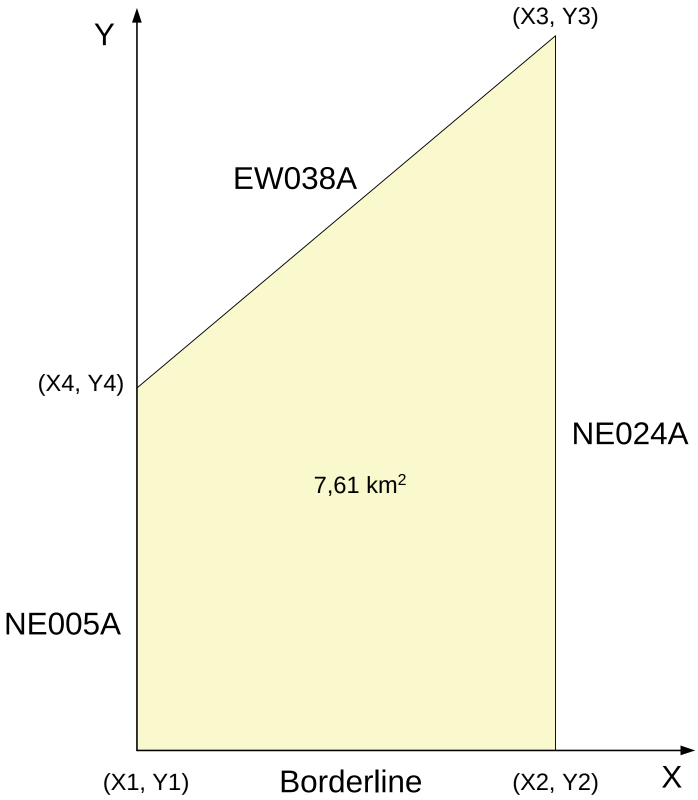

Figure C-2. *Schematic geometry in the model representing the area for deposition at Simpevarp (ground level).*

C1.2 Estimation of gross area at level –500 m

The outer boundaries of the simplified area according to Figure C-2 are assumed to have the following properties:

Pt.1 to Pt.2: Vertical borderline without variation

Pt.2 to Pt.3: Properties equal to zone NE024A

Pt.3 to Pt.4: Properties equal to zone EW038A

Pt.4 to Pt.1: Properties equal to zone NE005A

When knowing the dip of these borderlines the geometry of the deposition area at level –500 m can be calculated (A_{Gross}^{-500}).

Input data for estimation of gross area at level –500 m

Input data for calculation of gross area at level –500 m is the dip of the outer border lines that surround the studied area. Assumptions made are shown in Table C-2.

Table C-2. Stochastic input data for estimation of gross area at level –500 m.

Border line	Geometrical properties	Probability density function	Dip	Unit
Pt.1 to Pt.2:	Vertical border line without variation	–	–	–
Pt.2 to Pt.3:	Properties equal to zone NE024A	Triangular	Minimum 50.0 Likeliest 65.0 Maximum 75.0	Degrees
Pt.3 to Pt.4:	Properties equal to zone EW038A	Triangular	Minimum 65.0 Likeliest 80.0 Maximum 95.0	Degrees
Pt.4 to Pt.1:	Properties equal to zone NE005A	Triangular	Minimum 75.0 Likeliest 90.0 Maximum 105.0	Degrees

C1.3 Estimation of net area at level –500 m

The net area available for deposition at level –500 m is estimated by reducing the gross area by subtraction of the area occupied by the deformation zones (both those known and those unknown) and by the Central area. The area occupied by the known zones has been calculated according to Equation C-1:

$$A_{Zone}^{Known} = p \cdot L \cdot MAX(RD; MFE) \cdot 2 \qquad \text{Equation C-1}$$

p = confidence in existence, estimated individually for each zone

L = measured zone length within the simplified area (km²)

RD = Respect Distance (RD), see main report Section 1.5.6

MFE = Margin for Excavation (MFE), see main report Section 1.5.6 .

Table C-3. Assumptions for known zones (properties for zones marked with a line have been assumed to be valid for the area border lines, see Figure C-2).

Name	Degree of confidence	Possible occurrence** (%)	Respect distance (m)	Length of zone within the area (km)	Margin for Excavation
ZSMEW004A	High	99	100	3.57	MFE Large
ZSMEW009A	High	99	–	0.54	MFE Large
ZSMEW023A	Low	80	–	1.69	MFE Small
ZSMEW028A	High	95	–	0.89	MFE Small
ZSMEW038A	~~Low~~	~~0~~	~~–~~	~~3.30~~	~~MFE Small~~
ZSMEW316A	Low	50	–	0.93	MFE Large
ZSMNE005A	~~High~~	~~0~~	~~100~~	~~2.12~~	~~MFE Large~~
ZSMNE006A	High	99	–	1.53	MFE Large
ZSMNE012A	High	99	100	3.49	MFE Large
ZSMNE016A	High	95	–	0.81	MFE Large
ZSMNE018A	High	95	100	1.48	MFE Large
ZSMNE020A	Low	50	100	0.99	MFE Small
ZSMNE021A	Low	50	100	0.49	MFE Small

Name	Degree of confidence	Possible occurrence** (%)	Respect distance (m)	Length of zone within the area (km)	Margin for Excavation
ZSMNE024A	High	0	100	~~2.39~~	~~MFE Large~~
ZSMNE031A	Low	80	100	2.57	MFE Large
ZSMNE034A	Low	50	–	0.48	MFE Large
ZSMNS017A	High	75	–	1.25	MFE Large
ZSMNW004A	High	95	–	0.18	MFE Large
ZSMNW025A	High	95	–	1.21	MFE Small
ZSMNW035A	Low	50	–	1.81	MFE Small
ZSMNW321A	Low	50	–	0.22	MFE Large
Border SW*	Known	100	–	2.33	MFE Large
Border SE*	Known	100	–	3.96	MFE Large
Border N*	Known	100	–	3.03	MFE Large
Border NW*	Known	100	–	2.02	MFE Large

*) For all inner zones the greatest value of RD resp. MFE is multiplied by two (Equation C-1) as these distances are measured one-sided from the zones centre. For those zones that make up outer borderlines to the area this doubling shall not be made. As the border lines are fictive and do not correspond exactly to actual zones, RD have not been applied to these lines.

**) Possible occurrence for penetration of the designated facility volume.

The probability of the existence of zones that have not yet been identified has also been estimated. There is a certain probability of finding an unknown zone, a lesser probability of finding two unknown zones etc. The size of an unknown zone is calculated accordingly:

$$A_{Zon}^{Unknown} = L^{Unknown} \cdot MFE_{Small} \cdot 2 \qquad \text{Equation C-2}$$

where,

$L^{Unknown}$ = Estimated zone length based on the distribution of measured lengths of known zones within the area (km)

MFE_{Small} = The Margin for Excavation (MFE) is always supposed to be “Small” for unknown zones. MFE is measured from zone centreline (km).

The size of the central area is 0.085 km^2 /SKB 2002a/.

The net area for deposition at level –500 m may now be calculated accordingly:

$$A_{Net}^{-500} = A_{Gross}^{-500} - \sum A_{Zone}^{Known} - \sum A_{Zone}^{Unknown} - A_{Central\ area} \qquad \text{Equation C-3}$$

Input data for estimation of net area at level –500 m

Input data for estimation of net area at level –500 m are margins to zones (with regard to constructability etc), possible existence of zones and properties of possible currently now unknown zones, within the studied area. Assumptions made are shown in Table C-4.

Table C-4. Stochastic input data used for estimation of net area at level –500 m.

Assumption	Probability density function	Parameters		Unit
MFE large	Log normal	Mean 33.35		m
		Standard Dev.	21.28	
MFE small	Log normal	Mean 10.0		m
		Standard Dev.	5.0	
No. unknown zones	Poisson	Rate 0.2		–
Length unknown zone	Weibull	Location 0.0		m
		Scale 1.2		
		Shape 1.4		
Possible zones	Custom (J/N)	(See Table 10-2)		–

C2 Estimation of hole loss factor

According to UDP Design Task C4, Section 4.4, deposition holes can be disapproved for four different reasons:

1. Less than 2 m distance between the periphery of the deposition hole and stochastic fractures/fracture zones with a radius R > 100 m.
2. Water seepage in a deposition hole > 10 l/s.
3. Wedge breakout > 0.15 m^3/deposition hole.
4. Spalling in deposition hole.

In the simulation the loss factor is handled by the following expression:

$$(1-k) = \frac{(100-k_{fractures})\cdot(100-k_{water})\cdot(100-k_{Spalling})}{3\cdot 100} \qquad \text{Equation C-4}$$

where,

k = loss factor.

In the following sections is described how the different components of the loss factor k have been estimated in the analysis. From UDP design task according to C4, Section 4.4, it is however apparent that loss of deposition holes due to potential wedge breakout in the holes can be discounted in Simpevarp. Therefore this parameter is not further treated here.

C2.1 Loss due to adjacent large fractures

Loss due to adjacent large fractures has been estimated by means of data from DFN analysis. The data describes the estimated percentage loss of deposition holes for various tunnel orientations.

The whole table has been used to create a distribution of probability density. The motivation for this being that the simulation model does not take directions into account (see further discussions in the section on input data below.) The distribution has then been used to slump a loss percentage for one and each of the possible deposition blocks in Simpevarp, see Figure C-3.

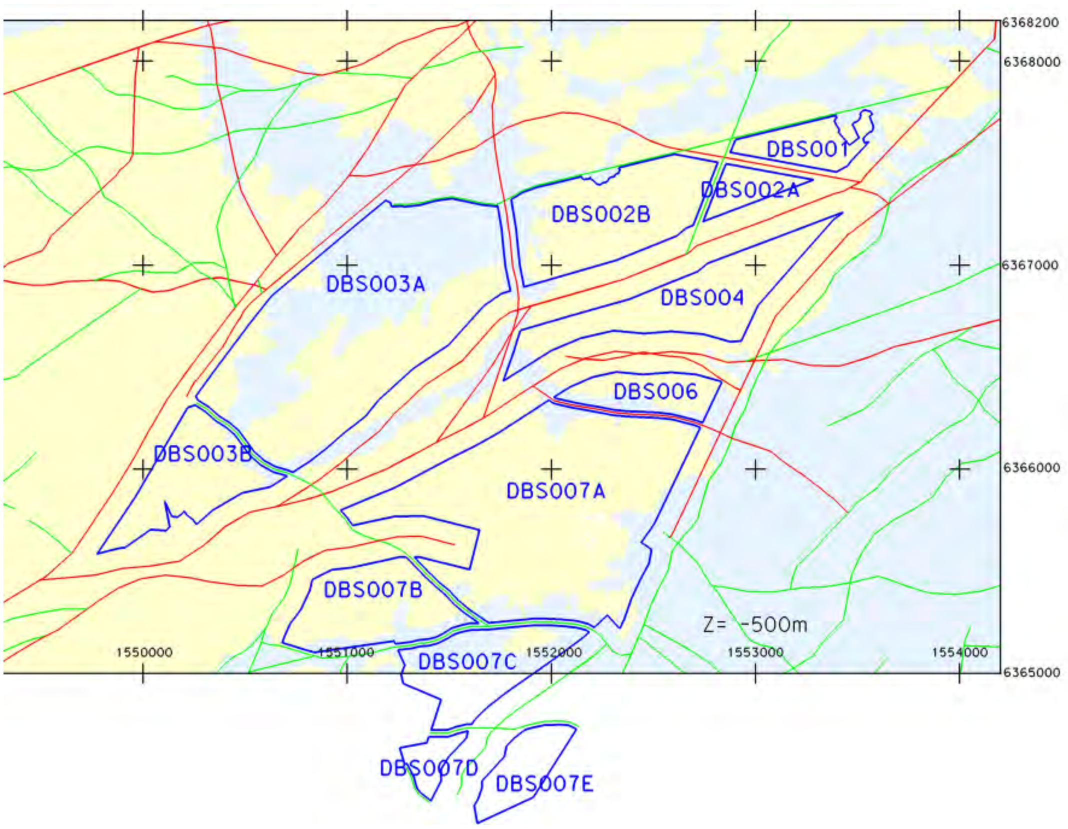

***Figure C-3**. Areas for possible deposition blocks in Simpevarp at level –500 m.*

The loss percentage for each deposition block has then been multiplied with the size of the deposition blocks. The weighted loss factor is calculated as the sum of these weighted (by area) losses, see Table C-5.

Table C-5. Methodology for deciding a loss factor (with regard to adjacent fractures), which is weighted by the deposition block areas.

Deposition block	Area (km^2)	Part of total area	Loss factor*	Weighted loss
DBS001	0.11	0.03	k_{tot}	k_{tot}×Part
DBS002A	0.06	0.02	k_{tot}	k_{tot}×Part
DBS002B	0.39	0.10	k_{tot}	k_{tot}×Part
DBS003A	1.08	0.26	k_{tot}	k_{tot}×Part
DBS003B	0.26	0.06	k_{tot}	k_{tot}×Part
DBS004	0.33	0.08	k_{tot}	k_{tot}×Part
DBS006	0.13	0.03	k_{tot}	k_{tot}×Part
DBS007A	1.07	0.26	k_{tot}	k_{tot}×Part
DBS007B	0.26	0.06	k_{tot}	k_{tot}×Part
DBS007C	0.25	0.06	k_{tot}	k_{tot}×Part
DBS007D	0.05	0.01	k_{tot}	k_{tot}×Part
DBS007E	0.12	0.03	k_{tot}	k_{tot}×Part
SUM	4.11		SUM	$k_{weighted}$

*) Stochastic. Simulated out of assumed distribution.

Input data for estimation of loss due to adjacent large fractures

With regard to the possibility that an earthquake might damage the canisters, a limitation has been introduced that a fracture with a radius larger than 100 m is not allowed within 2 m of the periphery of a deposition hole.

These fractures are so called stochastic fractures as their presence cannot be observed. Therefore, the analyses are based on fractures modelled as a network, DFN (Discrete fracture network) see /Munier 2004/.

A calculation based on DFN is given in UDP design task C4 "Loss of deposition holes", Section 4.4. In the DFN analysis carried out the loss percentage due to close proximity of such fractures is shown for various orientations of deposition tunnels. These values are based on 20 simulations, thus the number of samples is small. For this reason and because all orientations of the deposition tunnels in this report are considered as equally probable, all data from the table have been used when judging which statistical distribution shall be used in the simulation. By means of the program BestFit /BestFit 1996/ the distribution has been decided as log normal with a mean value 12.7% and standard deviation 13.1%. This gives a relatively large loss of deposition holes.

As it is hardly probable that the distribution of loss is identical over the whole area, each deposition block has been studied separately and the loss calculated as the weighted sum of all deposition blocks, where the loss within each deposition block has been given the mentioned distribution. Also the result, later used in the simulation, is a log normal distribution with similar mean value 12.44% but substantially lower standard deviation, 4.56%. See Figure C-4.

Another method for deciding the percentage loss is shown in a not yet published SKB report "An analytical method for estimating the probability of canister/fracture intersections in a KBS-3 repository" /Hedin 2005/. With a DFN model for Forsmark a mean value of approximately 1% is obtained. This result points to the need to further investigate this percentage loss.

It should be noted that in both analyses it is assumed that the canister position is not adjusted if fractures are discovered. This makes the analyses conservative.

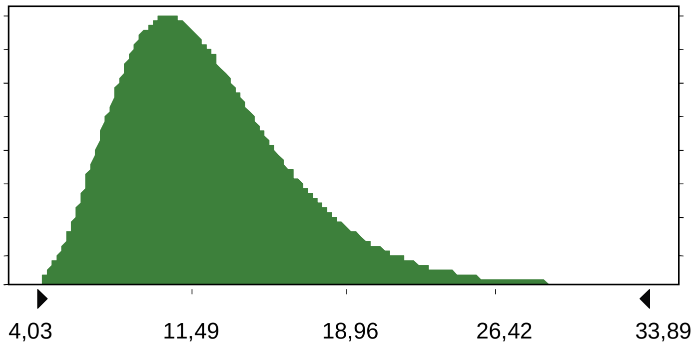

Figure C-4. *Assumed distribution of percentage loss of deposition holes due to adjacent large fractures.*

C2.2 Loss due to unacceptable seepage levels

The loss percentage due to unaccepted water inflow has been estimated by means of data from a DFN analysis. The estimation is based on the criteria 10 l/min, irrespective of direction.

Input data for estimation of loss due to high seepage levels

The applied distribution for estimation of loss due to unacceptably high seepage levels is log normal with a mean value of 0.53% and standard deviation of 1.37%, see Figure C-5.

C2.3 Loss due to spalling

The loss percentage due to spalling has been estimated by following the results shown in Table 4-16, design task C4, Section 4.4. In Simpevarp two stress domains have been identified, Domain I and Domain II. In the Report for Design task C4 it has been shown that the risk of spalling for Domain II and level –500 m is negligible. The total percentage loss factor due to spalling has been estimated with the following expression:

$$k_{Spalling} = A_{DomainI} \cdot S_{DomainI} \cdot Test[V] \qquad \text{Equation C-5}$$

$A_{Domain\ I}$ = percentage share of deposition holes in stress domain I

$S_{Domain\ I}$ = anticipated share of holes with spalling problems in stress domain I

V = expected fallout volume with spalling problems in stress domain I

$Test[V]$ = if $V \geq 0.15\ m^3$ the value is 1, otherwise 0.

Input data for estimation of loss due to spalling

Assumptions made regarding input data for the calculation of the percentage loss of deposition holes due to spalling are shown in Table C-6.

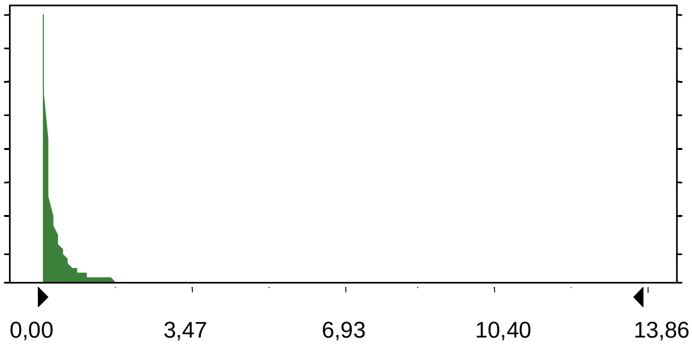

***Figure C-5.** Assumed distribution for percentage loss of deposition holes due to too much in-leaking water.*

Table C-6. Assumed distributions for percentage loss of deposition holes due to spalling problems.

Assumption	Probability density function	Parameters	Unit
Part stress domain	Triangular	Minimum 0.2 Likeliest 0.3 Maximum 0.4	–
Part spalling domain I	Triangular	Minimum 0.0 Likeliest 12.0 Maximum 25.0	–
Fallout vol./hole domain I	Triangular	Minimum 0.0 Likeliest 0.05 Maximum 0.2	m^3

C3 Estimation of specific hole area

A_S is the specific hole area, defined as the total area per deposition hole with regard to distance between holes and tunnels and share of the tunnel system area. Among other things, the hole spacing depends on the thermal properties of the rock. For Simpevarp four different rock domains have been identified with varying thermal properties.

A weighted hole spacing for the whole area was derived by weighting the hole distances in each domain against the domains share of the total area. The hole spacing in the different rock domains are presented in UDP design task C2, see Section 4.2 of this report.

Table C-7. Methodology for deciding an average hole spacing, which is weighted against areas of rock domains.

Rock type	Rock domain	Share of area*	Hole distance*	Weighted hole distance
Ävrö granite	A	$1–S_B–S_C–S_D$	C_A	$(1–S_B–S_C–S_D)×C_A$
Fine grained dioritoid	B	S_B	C_B	$S_B×C_B$
Mixture of quartz-monzodiorite/ Ävrö granite	C	S_C	C_C	$S_C×C_C$
Quartz-monzodiorite	D	S_D	C_D	$S_D×C_D$
			Sum	$C_{weighted}$

* Stochastic. Simulated out of assumed distribution.

$$A_S^{Adjusted} = C_{Weighted} \cdot 40 \cdot f$$
$$f = 1 + \frac{(25 + 8 - C_{Weighted})}{300} \qquad \text{Equation C-6}$$

$A_S^{Adjusted}$ = specific hole area, i.e. the total area per deposition hole with regard to distance between holes and tunnels (40 m) and share of the tunnel system area

$C_{Weighted}$ = weighted hole distance (see Table C-7)

f = modification factor with regard to end effects in deposition tunnels (see also Figure C-6). Deposition and main tunnels are included in $A_S^{Adjusted}$. Transport tunnels are not included as they mostly pass through areas outside A_{Net}.

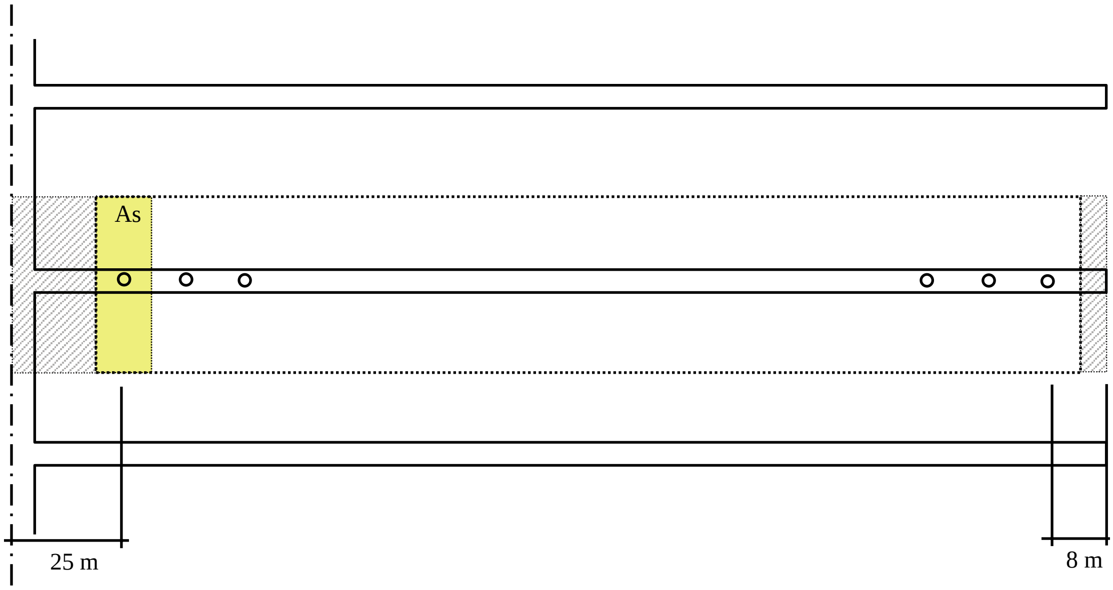

Figure C-6. *Background for modification factor with regard to end effects in deposition tunnels.*

Input data for estimation of specific hole area

Assumptions made regarding stochastic input data for calculation of specific hole area are shown in Table C-8.

Table C-8. Assumed distributions for estimation of specific hole area.

Assumption	Probability density function	Parameters	Unit
Part rock domain B	Triangular	Minimum 0.10	–
		Likeliest 0.17	
		Maximum 0.30	
Part rock domain C	Triangular	Minimum 0.03	–
		Likeliest 0.07	
		Maximum 0.10	
Part rock domain D	Triangular	Minimum 0.00	–
		Likeliest 0.00	
		Maximum 0.02	
Hole distance domain A	Extreme value	Mode 7.30	m
		Scale 0.50	
Hole distance domain B	Extreme value	Mode 7.50	m
		Scale 0.60	
Hole distance domain C	Extreme value	Mode 7.50	m
		Scale 0.65	
Hole distance domain D	Extreme value	Mode 8.00	m
		Scale 0.75	

C4 Estimation of rock spoil from blasting for deposition of 6,000 canisters

By means of the created simulation model estimation can also be made of the expected rock spoil from blasting for deposition of 6,000 canisters. The calculation regards firm masses.

For each simulation step the model generates a result in the form of the number of deposition holes that can be accommodated within the net area at level –500 m. Based on this number the corresponding total rock spoil from blasting can be calculated, see Table C-9.

Table C-9. Methodology for calculation of rock spoil from blasting corresponding to a certain number of deposition holes.

Part volume	Methodology
Deposition hole	See Equation C-7. Supposedly adjacent large fractures and risk of spalling problems will lead to disapproval of the position prior to excavation of the hole.
Deposition tunnels	See Equation C-9.
Main tunnels	See Equation C-10. Assumed 40 m ST per pair of 300 m long DT. To be multiplied by 1.5 with regard to all DT not being 300 m long.
Transport tunnels	See Equation C-11. Assumed length of TT is 10% and length DT (based on layout studies).
Central area	Constant value /SKB 2002a/.
Other (vent., shafts, etc)	Constant value /SKB 2002a/.

$$\sum V_{hole} = N_T \cdot \left(1 + \frac{k_{water}}{100}\right) \cdot V_{hole} \qquad \text{Equation C-7}$$

$$L_{DT} = N_P \cdot \frac{A_S^{Adjusted}}{40} \qquad \text{Equation C-8}$$

$$V_{DT} = L_{DT} \cdot A_{DT} \qquad \text{Equation C-9}$$

$$V_{ST} = 1.5 \cdot \frac{L_{DT} \cdot 40}{2 \cdot 300} \cdot A_{ST} \qquad \text{Equation C-10}$$

$$V_{TT} = 0.1 \cdot L_{DT} \cdot A_{TT} \qquad \text{Equation C-11}$$

N_T = total no.s canisters that can be accommodated

k_{water} = percentage loss of hole positions due to water in-leakage

V_{hole} = volume of one deposition hole (ø1.75 m, depth 8 m)

L_{DT} = total length of deposition tunnels

N_P = total number of deposition positions, equal to no.s canisters + disapproved positions

$A_S^{Adjusted}$ = specific hole area, i.e. the total area per deposition hole with regard to distance between holes and tunnels and share of the tunnel system area

A_{DT} = cross section, deposition tunnels

A_{ST} = cross section, main tunnels

A_{TT} = cross section, transport tunnels.

Appendix D

Supplementary update based on Site Description Model Laxemar v 1.2

Summary

The underground design in the main report is based on the site descriptive model SDM Simpevarp 1.2 /SKB 2005a/. After the delivery of the report new information on the site is now available and these changes are judged to have an impact on the main question as to whether the repository can be accommodated within the site or not. In order to have a just foundation for decision making when choosing between the sites at Simpevarp and Laxemar, SKB has assigned FB Engineering to update those parts of the report that affect the actual question based on SDM Laxemar 1.2 /SKB 2006a/.

Deformation zones have been both added and deleted in connection with the new site descriptive model SDM Laxemar 1.2. In Simpevarp there are now a total of 22 deformation zones to be considered when accommodating the final repository, 5 zones with medium and 17 zones with a high level of confidence. This is to be compared with the former number of deformation zones which according to SDM Simpevarp 1.2 was a total of 21. 9 zones with confidence level "possible" and 12 zones with high level of confidence. In addition several deformation zones have different estimated lengths than previously which in several cases affects Respect Distances and Margins for Excavation. In all, the total available area for repository is reduced by 15% to 3.5 km².

A revised Geo-DFN and a revised approach to long fractures means that the loss of deposition holes increases from 10% to 15%, i.e. to accommodate 6,000 canisters in the repository 6,900 canister positions are required.

Layout studies show that the total potential of the area allows the repository to be accommodated with a total of 6,935 canister positions being theoretically available. With a modified area bounded in the east by deformation zone ZSMNE024A, instead of the interest area border, the repository can not be accommodated as only 5,430 canister positions are available.

A technical risk evaluation by stochastic methods was performed in order to clarify the total potential and this resulted in 6,070 canisters being accommodated. If the area confined in east by deformation zone ZSMNE024A is analyzed, instead of the interest area border, the repository cannot be accommodated as there is only room for 4,040 canisters.

It can be concluded that the analysis shows that the repository can be accommodated but there are no margins in addition to those in the analysis

1 Background

The underground design in the main report is based on the site descriptive model SDM Simpevarp 1.2 /SKB 2005a/.

After completion of the report new information on the site is now available that is judged to have an impact on the main question as to whether the repository can be accommodated within the site or not. In order to have a just foundation for decision making when choosing between the sites at Simpevarp and Laxemar, SKB has assigned FB Engineering to update those parts of the report that affect the actual question based on SDM Laxemar 1.2 /SKB 2006a/.

2 Deterministic deformation zones

Deterministically interpreted deformation zones within Simpevarp according to SDM Simpevarp 1.2 are shown in Figure D2-1 and according to SDM Laxemar 1.2 in Figure D2-2. Zones with a high level of confidence are marked with red lines and zones with medium and low levels of confidence with green lines.

In SDM Simpevarp 1.2 deterministically interpreted deformation zones were characterized by two different levels of confidence, "high" and "possible". These were in the original D1 design for Simpevarp denominated with "high" resp. "possible" levels of confidence. For SDM Laxemar 1.2 there are three different levels of confidence, "high", "medium" and "low". In accordance with the design that has been performed for Laxemar consideration is taken to zones in Simpevarp with high and medium levels of confidence.

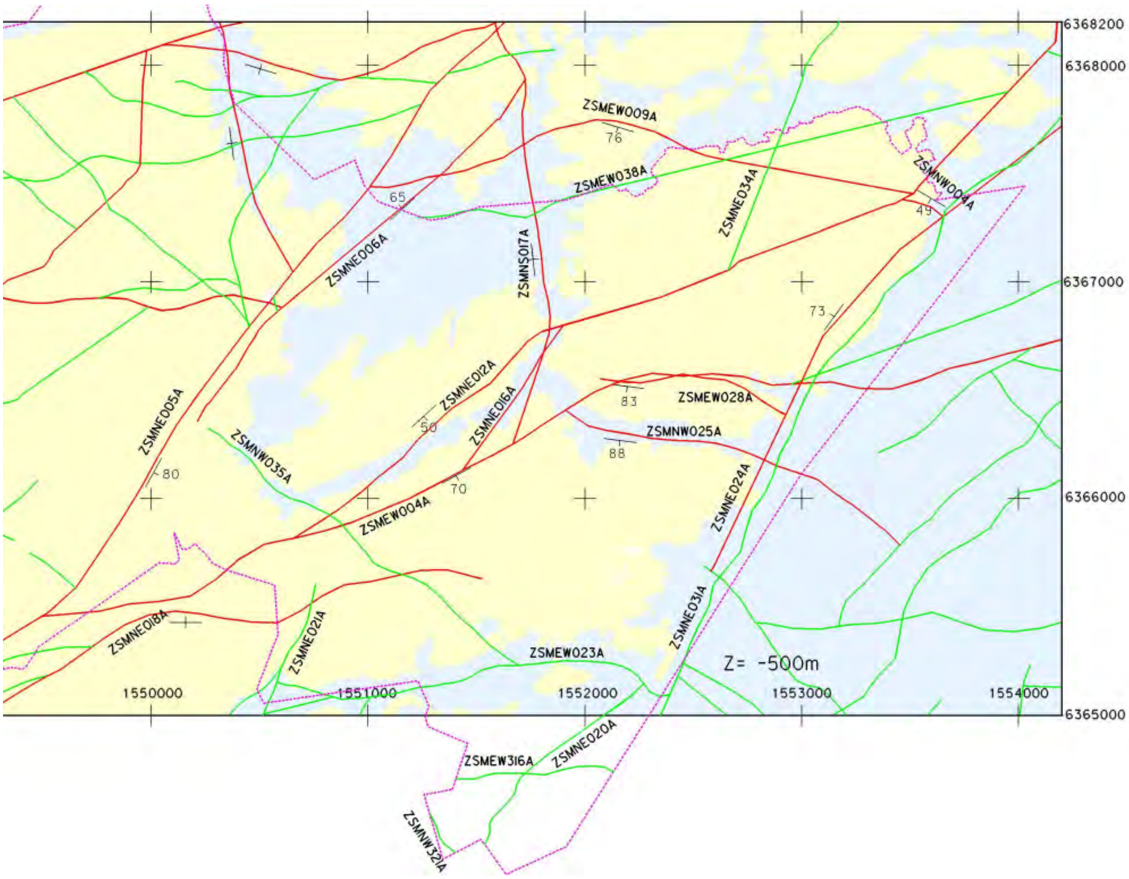

Figure D2-1. Deterministically interpreted deformation zones according to SDM Simpevarp 1.2. Zones with a high level of confidence are marked with red lines and "possible" zones (confidence level) with green lines. Level –500 m /SKB 2005a/.

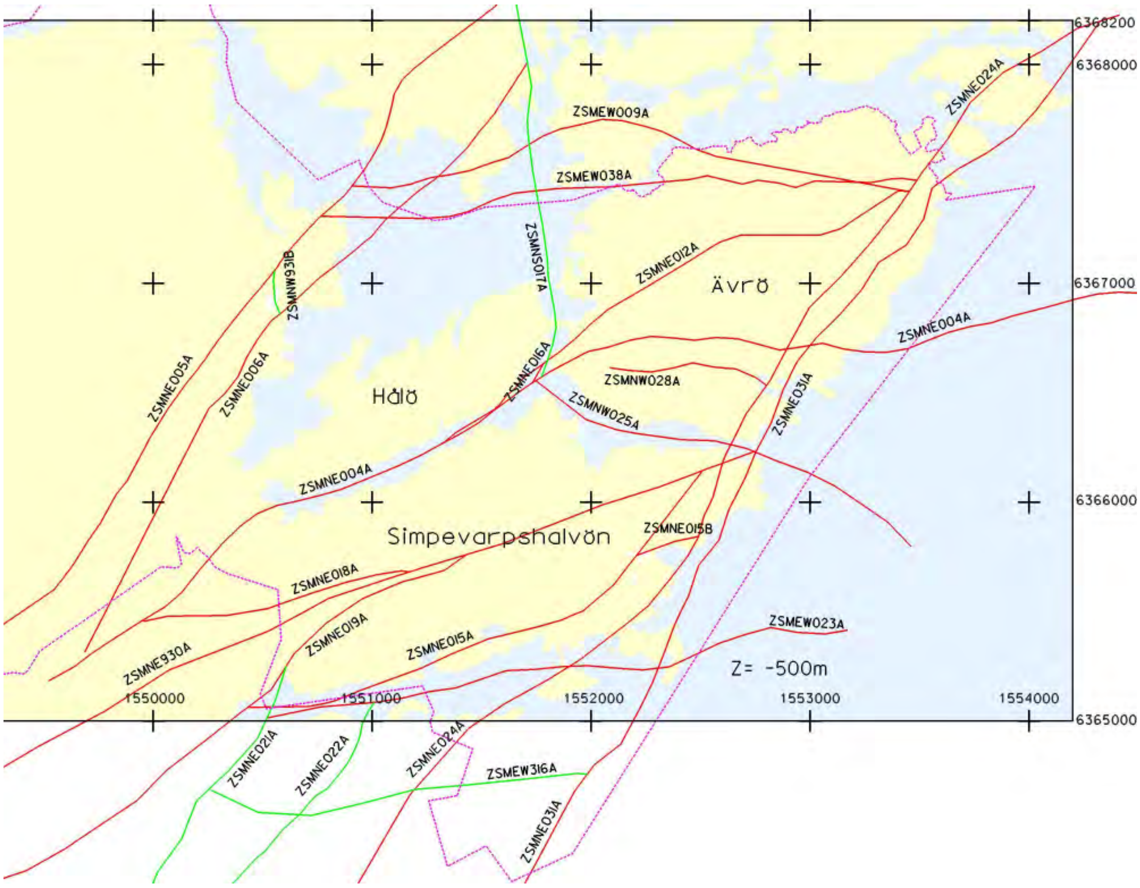

***Figure D2-2.** Deterministically interpreted deformation zones according to SDM Laxemar 1.2. Zones with a high level of confidence are marked with red lines and zones with a medium level of confidence with green lines. Level –500 m /SKB 2006a/.*

The figures show that deformation zones have been both added and deleted. Furthermore, some deformation zones have received revised denominations and upgraded levels of confidence compared to the former model. According to SDM Laxemar 1.2 there are a total of 22 deformation zones in Simpevarp to be considered when accommodating the final repository, 5 zones with medium and 17 zones with a high level of confidence. This should be compared with the former number of deformation zones which according to SDM Simpevarp 1.2 was 21 in total, 9 zones with possible and 12 zones with a high level of confidence.

Respect Distance (RD) and Margin for Excavation (MFE) for deformation zones within the Simpevarp interest area, based on SDM Simpevarp 1.2 resp. SDM Laxemar 1.2, are shown in Table D2-1. As can be seen in the table the estimated Margin for Excavation is normally considerably lower than the deformation zone's Respect Distance. However, the methodology applied gives the result that MFE exceeds the RD for deformation zone ZSMNE005A. The reason is that the interpreted characteristic width of the deformation zone amounts to as much as 250 m. The calculated MFE for the deformation zone may be questioned, but due to the fact that the zone on the whole runs parallel to deformation zone ZSMNE006A the effect on the available area for the repository is very marginal. It has thus not been considered necessary to make adjustment for this marginal change.

Table D2-1. Respect Distance (RD) and Margin for Excavation (MFE) for zones within the Simpevarp interest based on SDM Simpevarp 1.2 resp. SDM Laxemar 1.2.

Zone (ID) [a]	SDM Simpevarp 1.2 Level of Confidence	RD (m)	MFE (m)	SDM Laxemar 1.2 Level of Confidence	RD (m)	MFE (m)
ZSMEW009A	High	–	29	High	–	30
ZSMEW023A	Possible	–	16	High	100	35
ZSMEW028A	High	–	13	–	–	–
ZSMEW038A	Possible	–	9	High	100	12
ZSMEW316A	Possible	–	17	Medium	–	17
ZSMNE004A[b]	High	100	45	High	100	80
ZSMNE005A	High	100	105	High	125	155
ZSMNE006A	High	–	41	High	–	85
ZSMNE012A	High	100	51	High	100	80
ZSMNE015A	–	–	–	High	–	28
ZSMNE015B	–	–	–	High	–	12
ZSMNE016A	High	–	29	High	–	32
ZSMNE018A	High	100	28	High	–	42
ZSMNE019A	–	–	–	High	100	10
ZSMNE020A	Possible	100	34	–	–	–
ZSMNE021A	Possible	100	31	Medium	100	28
ZSMNE022A	–	–	–	Medium	–	19
ZSMNE024A	High	100	65	High	100	70
ZSMNE031A	Possible	100	35	High	100	20
ZSMNE034A	Possible	–	20	–	–	–
ZSMNE930A	–	–	–	High	100	25
ZSMNS017A	High	–	35	Medium	–	15
ZSMNW004A	High	–	40	–	–	–
ZSMNW025A	High	–	13	High	–	12
ZSMNW028A	–	–	–	High	–	12
ZSMNW035A	Possible	–	14	–	–	–
ZSMNW321A	Possible	–	11	–	–	–
ZSMNW931B	–	–	–	Medium	–	7

a) References /SKB 2005a/, /SKB 2006a/.

b) The denomination of the deformation zone has been changed and corresponds to zone ZSMNE004A in SDM Laxemar 1.2.

c) Only zones with degree of confidence ”high” and ”medium” are represented.

Deterministically interpreted deformation zones at level –500 m with RD and MFE according to Table D2-1 are shown on a map based on SDM Simpevarp 1.2 in Figure D2-3 and based on SDM Laxemar 1.2 in Figure D2-4. The corresponding deposition blocks remaining after limitation by RD and MFE based on SDM Simpevarp 1.2 are shown in Figure D2-5 and for SDM Laxemar 1.2 in Figure D2-6.

The figures show that the actual site description results in a deposition area which is divided into a greater amount of deposition blocks with lesser areas. Furthermore, a few deposition blocks have been added east of deformation zone ZSMNE024A due to the fact that this zone has been moved to the west in SDM Laxemar 1.2.

Based on SDM Laxemar 1.2 the total area available for repository is 3.5 km². This is to be compared with the former estimate of 4.1 km² based on SDM Simpevarp 1.2. Thus, the area available for repository has been reduced by 15%.

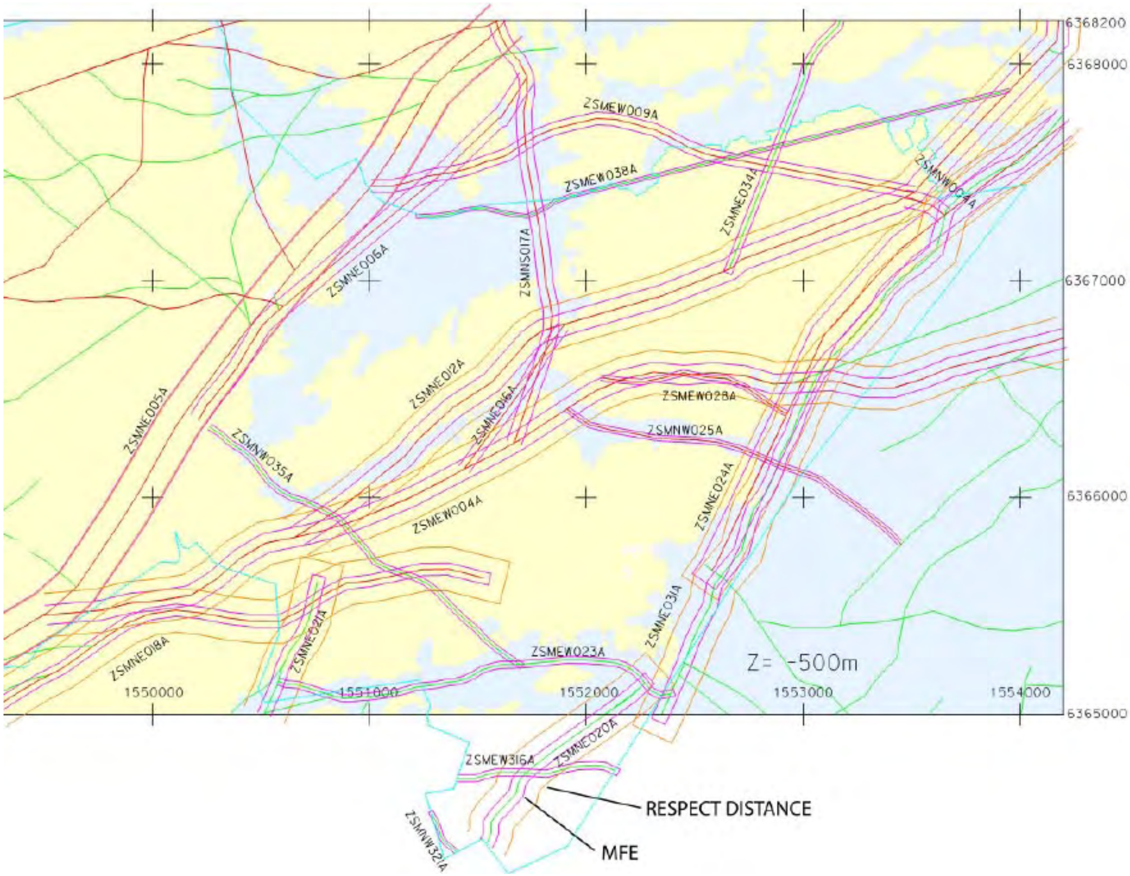

Figure D2-3. *Representation of RD and MFE for deterministically interpreted zones based on SDM Simpevarp 1.2. Zones with high level of confidence are marked with red lines and possible zones with green lines. Level –500 m /SKB 2005a/.*

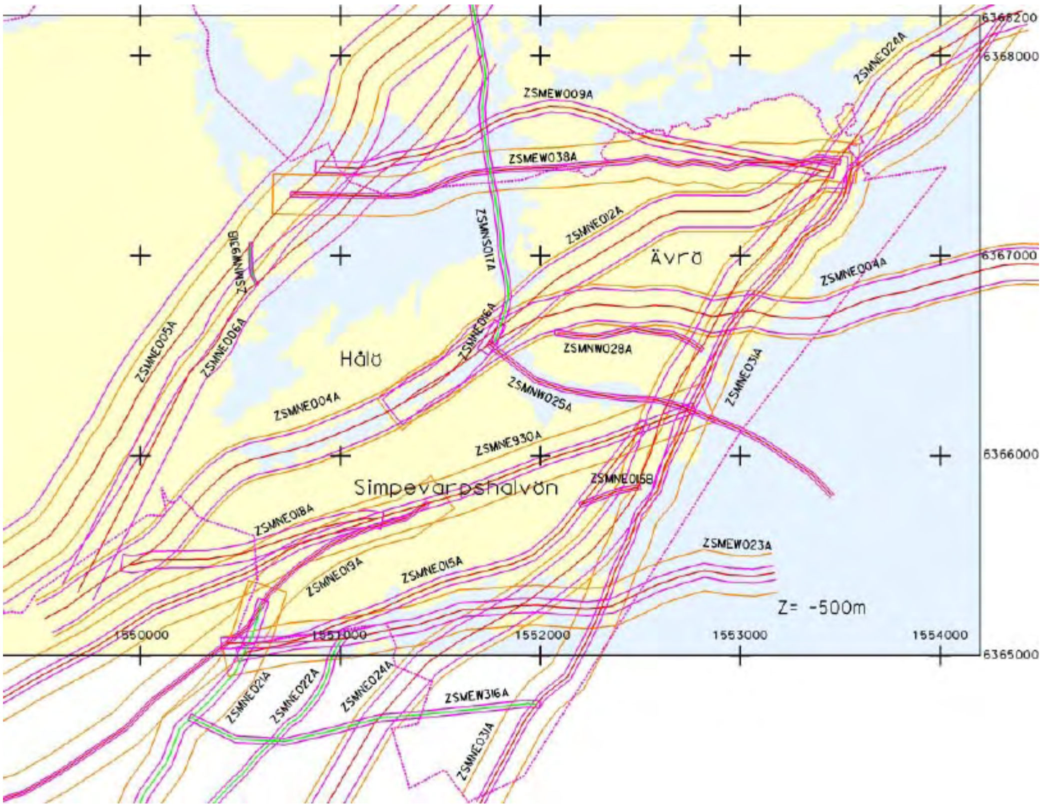

***Figure D2-4.** Representation of RD and MFE for deterministically interpreted zones based on SDM Laxemar 1.2. Zones with high level of confidence are marked with red lines and zones with medium level of confidence with green lines. Level –500 m /SKB 2006a/.*

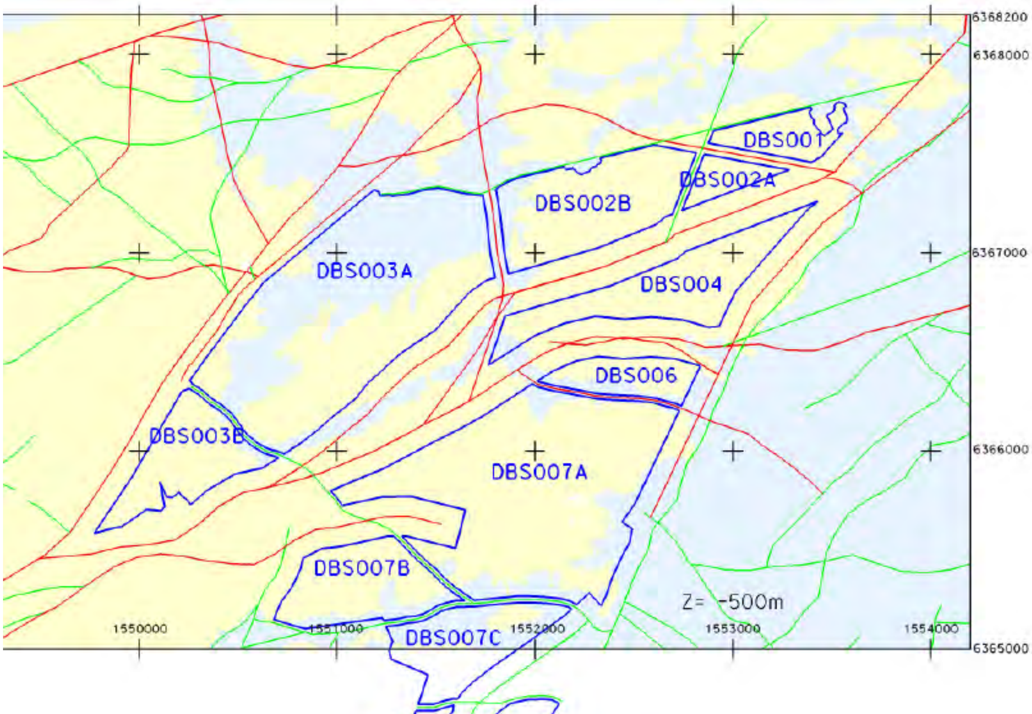

***Figure D2-5.** Areas and shapes of deposition blocks in Simpevarp based on SDM Simpevarp 1.2. Zones with a high level of confidence are marked with red lines and possible zones with green lines. Level –500 m /SKB 2005a/.*

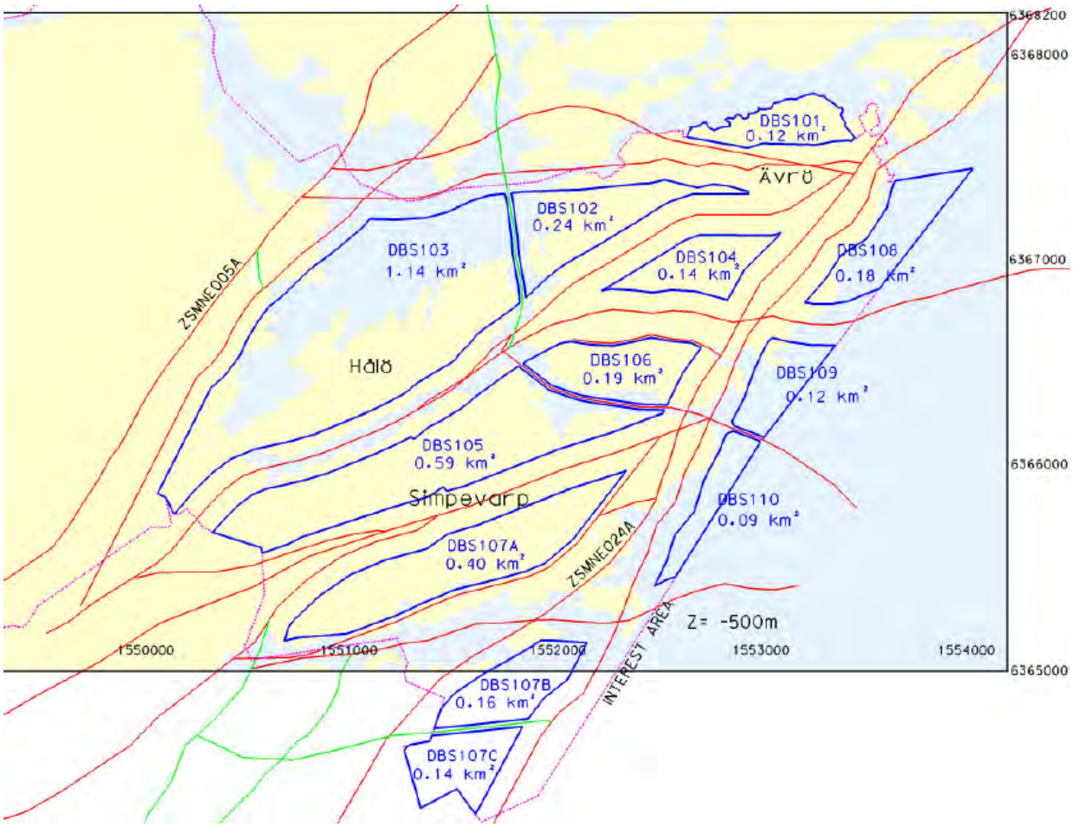

Figure D2-6. *Areas and shapes of deposition blocks in Simpevarp based on SDM Laxemar 1.2. Zones with a high level of confidence are marked with red lines and zones with a medium level of confidence with green lines. Level –500 m /SKB 2006a/.*

3 Loss of deposition holes

3.1 Stochastically determined fractures/fracture zones with great lengths that cross deposition holes

Loss of deposition holes due to stochastically determined fractures/fracture zones with great lengths has formerly been analysed by means of a DFN model (Discrete Fracture Network-model), but has again been analysed by SKB by means of a analytical method /Hedin 2005/. The input data to the analysis is based on a revised Geo-DFN of Simpevarp included in SDM Laxemar 1.2 /SKB 2006a/. The loss criteria for the analysis has been that deposition holes that are crossed by fractures/fracture zones with radius in the interval 50 m < R < 600 m are lost /SKB 2006b/.

The analytical method is based on the assumption that if the distribution of the fractures length and orientation in the rock is known and if a deposition hole is placed randomly, then it is possible to calculate the probability for a deposition hole to be crossed by a fracture that exceeds a certain length. The input data to the analysis consist of statistically described fracture lengths and fracture orientations from the site description. The analysis includes the complete fracture population, i.e. both open and closed fractures. The fractures are presumed to consist of infinitely thin circular slices.

Recorded results show an approx 16% loss of deposition positions due to fractures/fracture zones with great lengths that cross deposition holes. The result is independent of the orientation of the tunnel.

3.2 Amount of seepage water in deposition holes

Loss of deposition holes due to seepage water in deposition holes has been calculated by DFN analysis with the software NAPSAC. Input parameters to the DFN analysis have been collected from the Hydro-DFN, /SKB 2006a/. The model is supposed to be valid for rock domain B and C for levels below 300 m.

The analysis includes deposition holes at repository level 500 m placed along a deposition tunnel with orientation N42° and N132°. The loss criterion for the analysis has been q > 10 l/min per deposition hole. In order to illustrate the sensitivity of the loss criteria used, the loss has also been calculated for the criterion q > 1 l/min per deposition hole.

Reported results from the DFN analysis indicate that the amount of seepage water will be below the criterion q > 10 l/min in all deposition holes. Loss of deposition holes will happen only if the loss criterion is increased to q > 1 l/min per deposition hole. When applying the stricter criterion, the calculations show a maximum loss of 3% of deposition holes. An orientation dependent seepage for the orientations selected (N42° resp. N132°) is not statistically founded.

3.3 Summary

The performed calculations of losses due to stochastically determined fractures/fracture zones with great lengths that cross deposition holes and the amount of seepage water in deposition holes, implies that the total loss may be expected to reach a level of 15% of the theoretically possible number of canister positions available for deposition.

The probability of losses of deposition holes due to wedge breakouts and spalling is judged to be insignificant at a repository level of 500 m.

4 Layout

The original layout proposal for level –500 m based on deformation zones according to SDM Simpevarp 1.2 is shown in Figure D4-1. The layout can accommodate a total of 6,605 deposition holes distributed in seven deposition blocks.

Figure D4-2 shows the proposed layout again but includes areas that are eliminated due to the changed interpretation of deformation zones according to SDM Laxemar 1.2.

The changed interpretation of deformation zones implies that 2,430 deposition holes (37%) are eliminated from the formerly proposed layout. Thus only 4,175 holes remain available for deposition. The layout must thus be adapted in a way that further deposition blocks within Simpevarp are used.

In order to assess the actual potential in Simpevarp to accommodate a final repository within the deposition blocks that are still available, a “max alternative” has been drafted where all available deposition blocks are utilized, i.e. possible areas for repository, see Table D4-1. In the analysis the orientation of the deposition tunnels has been optimized based on the shape of the deposition blocks in order to accommodate as many tunnels as possible.

As can be seen from Table D4-1, Simpevarp has an estimated potential, based on SDM Laxemar 1.2, for a total of 7,835 deposition holes with the interest area border as the east boundary line, see Figure D2-6. In Design D1 SKB defined the deformation zone ZSMNE024A as the eastern boundary line because it largely coincided with the coastal line. Should ZSMNE024A even now form the eastern boundary line, then 6,330 canisters would be accommodated.

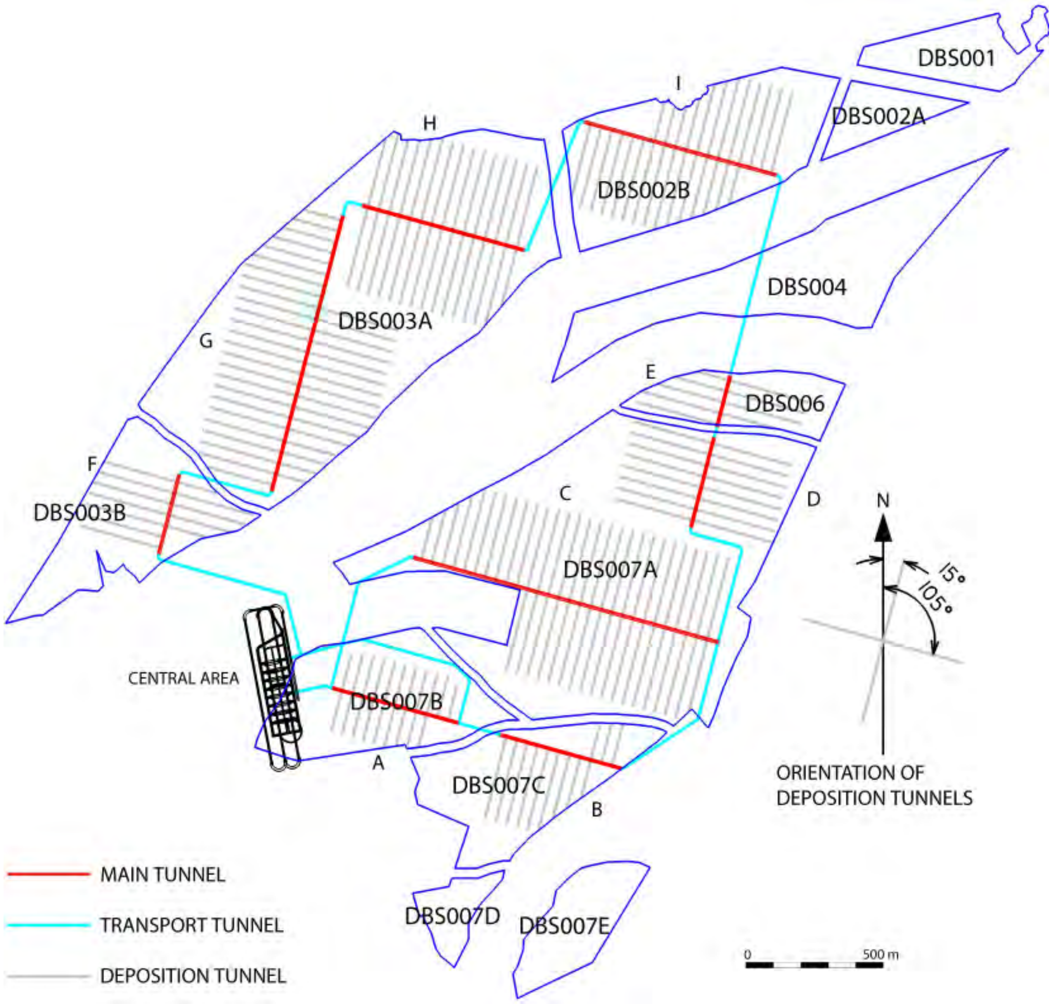

***Figure D4-1.** Originally proposed layout based on deformation zones according to SDM Simpevarp 1.2.*

Table D4-1. Potential for total number of canister positions in Simpevarp based on to SDM Laxemar 1.2.

Deposition block	Number of deposition holes
DBS101	279
DBS102	523
DBS103	2861
DBS104	258
DBS105	1363
DBS106	444
DBS107A	604
DBS107B	347*
DBS107C	347*
DBS108	378*
DBS109	300*
DBS110	131*
TOTAL	7,835

*) Canister positions east of ZSMNE024A and west of interest area, see Figure D2-6.

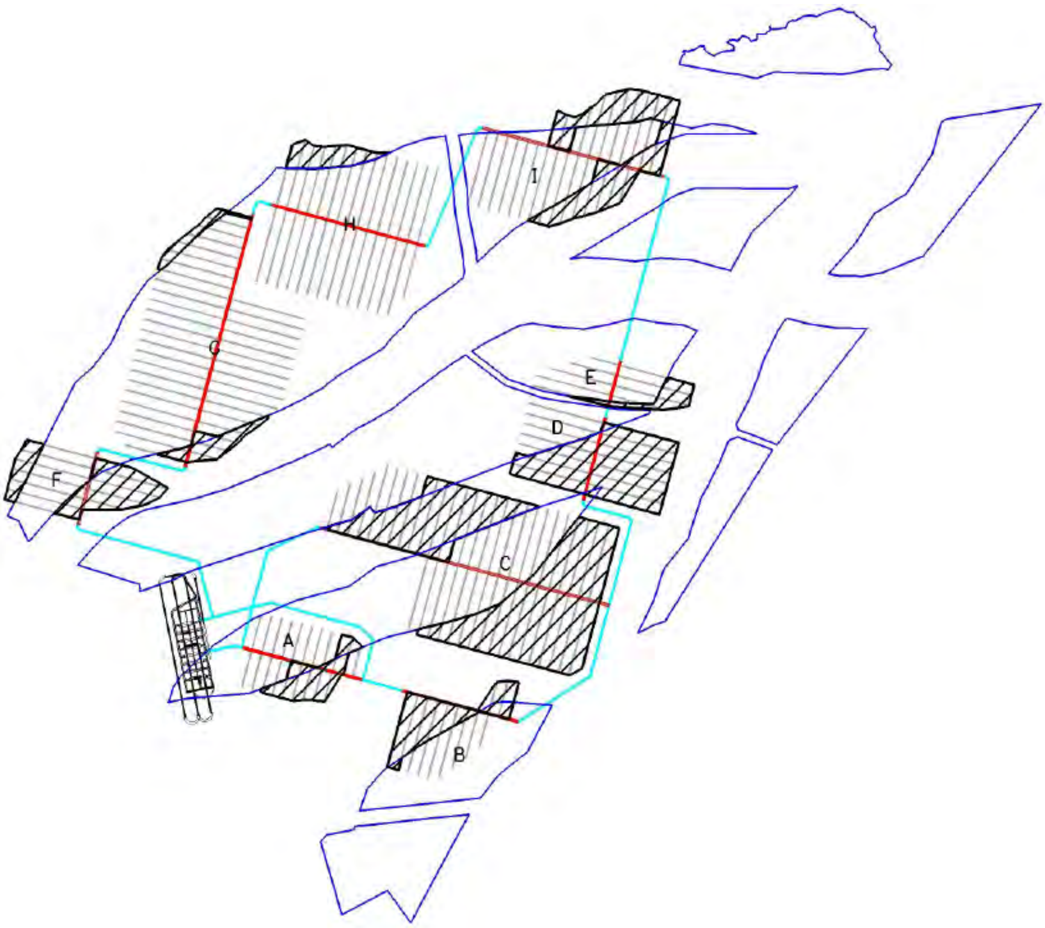

Figure D4-2. *Proposed layout where areas that are eliminated due to changed interpretation of deformation zones according to SDM Laxemar 1.2 have been highlighted.*

Based on a basic layout with 6,900 deposition holes, including a loss of 15% according to the presentation in earlier sections, Simpevarp is still estimated to have a potential to accommodate a final repository. However, the new conditions based on SDM Laxemar 1.2 require that the deposition blocks east of ZSMNE024A are partly utilized.

5 Technical risk evaluation

5.1 Assignment and conditions/pre-requisites

The changed conditions at Simpevarp, according to the site description SDM Laxemar 1.2, imply that the technical risk evaluation for Simpevarp should be updated and new MonteCarlo simulations be performed with the new conditions in order to evaluate whether the repository can accommodate 6,000 canisters. Methodology and performance would follow that presented in the main report.

The new conditions for the site concerning deterministically determined zones are in accordance with Chapter 2 and loss of deposition holes in accordance with Chapter 3 of this report.

5.2 Result

The model that was built for the original assignment is used with new input data for losses and deformation zones.

The result which shall answer the question "can the repository be accommodated within the assigned site" is simulated for two alternatives; 1) for an available area bounded by ZSMNE024A in the east and 2) for a larger available area following the interest area boundary.

With 50,000 simulations an average of 4,040 canisters (approved canister positions) can be accommodated for the alternative with ZSMNE024A as the limitation to the east, see Figure 5D-1 and Table D5-1.

Factors having the greatest influence on the result in the reported simulation are shown in Figure D5-2, ranked according to their relative contribution to the uncertainty (variance). The factor with greatest influence in this alternative is the dip at the eastern border line, i.e. ZSMNE024A. The Margin for Excavation (MFE) also has a relatively large influence on the result.

The alternative with an available area following the interest area border in the east, after 50,000 simulations, gives a mean of 6,070 approved canister positions, see Figure D5-3 and Table D5-1.

For the alternative with an available area following the interest area border in the east, MFE is the factor with greatest influence on the result. see Figure D5-4. The reason that the eastern border line in this alternative does not influence the result is that it is arbitrarily defined as a vertical structure.

The increase in the number of canister positions if the eastern boundary is changed from ZSMNE024A to the interest area is on average approx 2,000 approved canister positions, as can be seen in Figure D5-5.

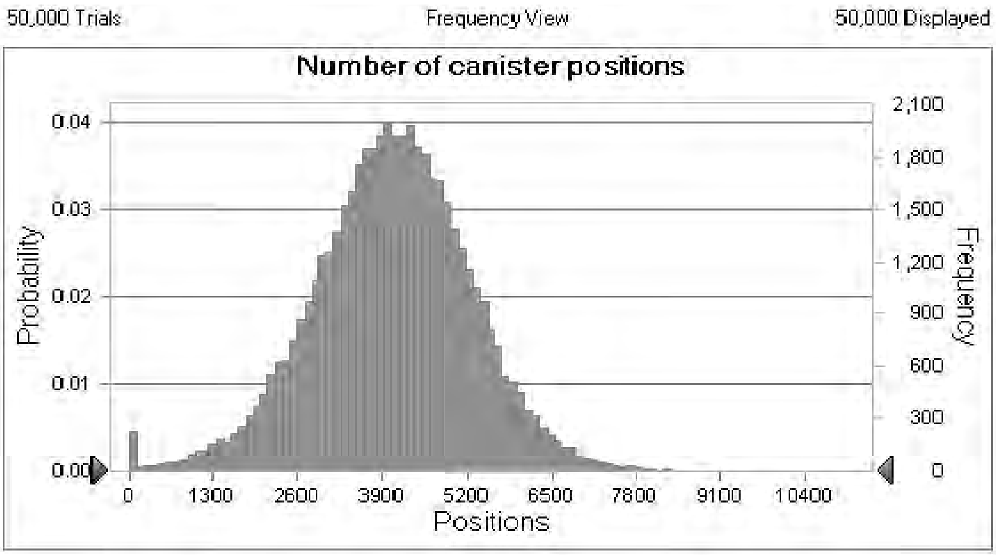

Figure D5-1. *Number of approved canister positions with eastern border line of ZSMNE024A.*

Table D5-1. Result of MonteCarlo simulations for number of approved canister positions.

	Border E: ZSMNE024A	Border E: Interest area
Number of simulations	50,000	50,000
Approved canister positions, mean	4,039	6,069
Approved canister positions, median	4,072	6,086
Standard deviation	1,207	1,132
Variance	1,455,791	1,280,403

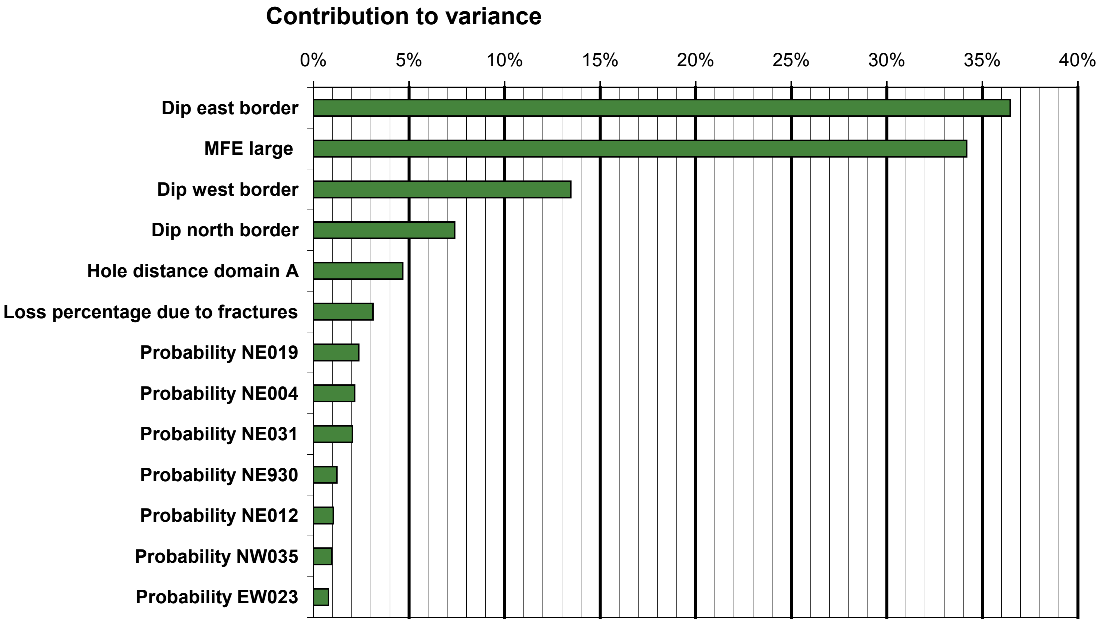

Figure D5-2. Factors ranked according to their relative contribution to the uncertainty in the analysis. Refers to the analysis of the number of approved canister positions with eastern border line of ZSMNE024A.

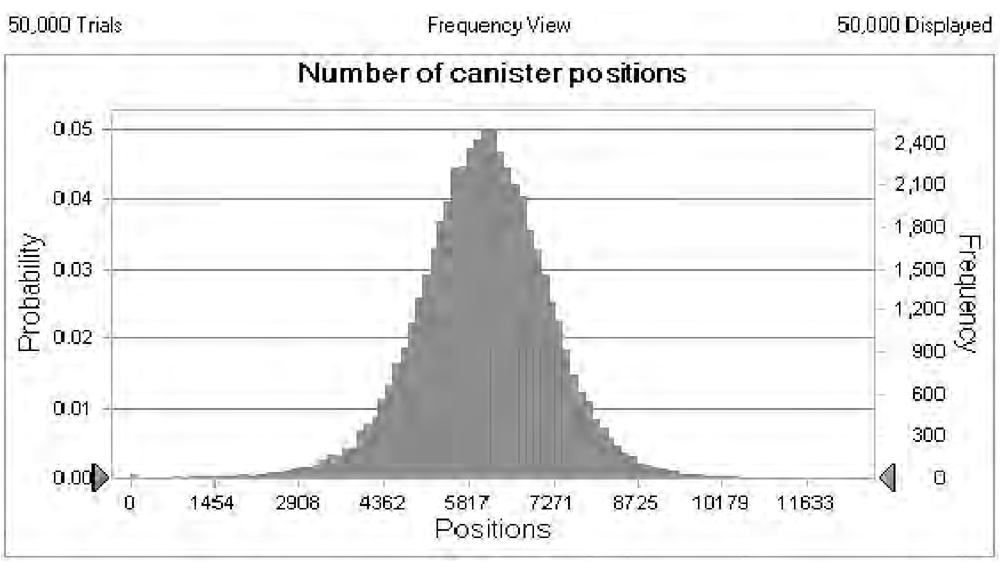

Figure D5-3. Number of approved canister positions with the interest area as the eastern boundary line.

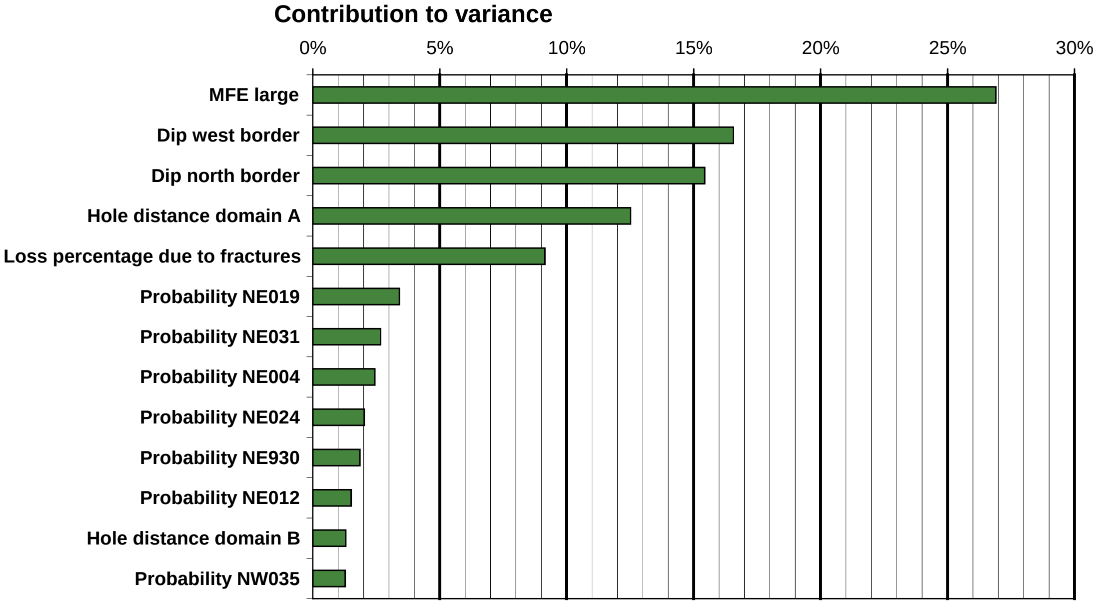

Figure D5-4. *Factors ranked according to their relative contribution to the uncertainty in the analysis. Concerns the analysis of the number of approved canister positions with the interest area as the eastern boundary line.*

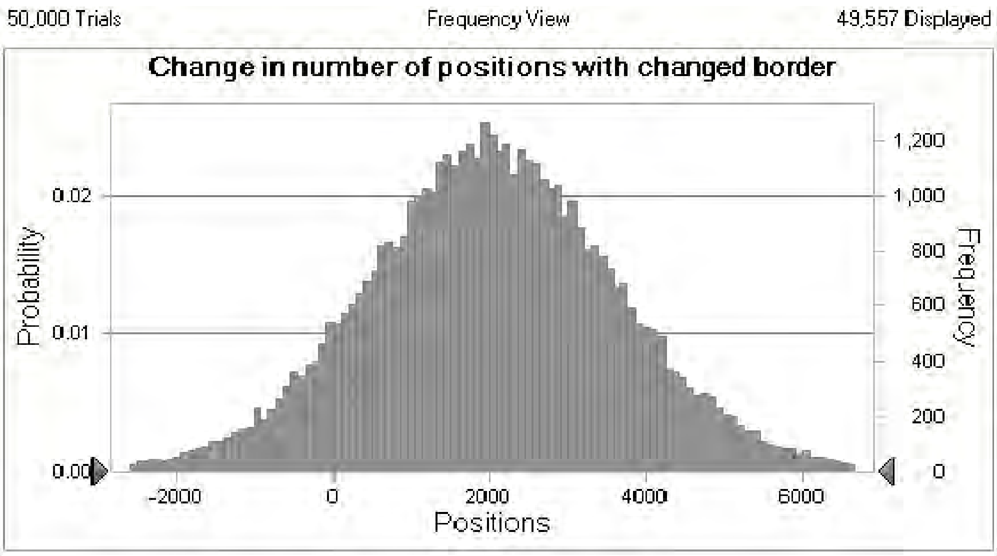

Figure D5-5. *Expected additional number of canister positions due to modification of the eastern boundary from ZSMNE024A to the interest area border line.*

6 Conclusions

The changed conditions at Simpevarp that are presented in the site description SDM Laxemar 1.2 imply that the situation for accommodating the repository in the area are less favourable. It is mainly the zones Respect Distances and MFE that have increased substantially and thereby reduced the available area for deposition from the original 4.1 km² to 3.5 km². Furthermore, one of the main coherent deposition blocks is now subdivided in an unfavourable way.

The suggested layout based on SDM Simpevarp 1.2 is considerably affected by the zones Respect Distances and MFE that have changed in connection with SDM Laxemar 1.2. Of the 6,605 canister positions placed in the layout, 2,430 will fall within corridors of Respect Distance and MFE, i.e. 37%. Thus 4,275 remain as deposition holes. Positions that have been lost can be placed in other deposition blocks.

The analysis of what can be accommodated within the Simpevarp area after revision based on SDM Laxemar 1.2 is performed partly by modifying deposition tunnel layouts and partly by a stochastic approach in an additional technical risk evaluation.

The result from the deterministic exercise indicates a total potential of 6,935 approved canister positions that includes a reduction by 900 representing a loss of 15% for the case where the whole area is utilized. If ZSMNE024A is used as the eastern boundary instead of the interest area border then the number of approved canister positions is reduced to 5,430 after reduction for losses. This means that a repository with 6,000 canisters can be accommodated within the assigned area with the interest area border forming the eastern boundary but that the area is not sufficient when using ZSMNE024A as eastern boundary.

The technical risk evaluation gives a similar result where the larger interest area gives a mean value of 6,070 approved canister positions and the smaller area, with ZSMNE024A as eastern border line, has 4,040 approved canister positions.

To summarize it can be said that both analyses show that the repository can be accommodated within the larger area. However, there are no margins in addition to those included in the analysis.