Controlling of Disturbances due to Groundwater Inflow into ONKALO and the deep Repository

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TYÖRAPORTTI: CONTROLLING OF DISTURBANCES DUE TO GROUNDWATER INFLOW INTO ONKALO AND THE DEEP REPOSITORY

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CONTROLLING OF DISTURBANCES DUE TO GROUNDWATER INFLOW INTO ONKALO AND THE DEEP REPOSITORY

ABSTRACT

The main goals of the report are identifying possible disturbances due to ingress of groundwater into ONKALO and the repository and identifying principles of the technical solutions for controlling groundwater leakages.

Inflow of groundwater into underground facilities and the subsequent pumping of the water to the ground surface during the construction and operation phase bring along flow of groundwater in the surrounding rock, movement and mixing of different water types, and drawdown of groundwater table. Flow of groundwater into the excavations causes migration of superficial water into the bedrock. From the point of view of the postclosure performance of the repository, the most significant impacts are related to the inflows into the deep parts of the underground facilities (salinity changes), the interactions between different materials and to the consumption of the buffering capacity of fracture fillings and rock in the vicinity of the disposal tunnels. The upconing of deep, saline water can be also related to the drawdown of groundwater table. Measured in metres, the upconing may be several times higher than the drawdown of groundwater table. During construction, rock engineering works and structural construction as well as installation of the systems works, will meet difficulties if high inflow rates are tolerated. During disposal operations, leakages may cause additional problems during installation of buffer and backfilling works.

Inflow of groundwater can be limited by a careful selection of the locations of the underground facilities and their surface connections and by forming watertight zone in the rock around tunnels and shafts. This is normally made by grouting the rock before or after excavation. More exotic techniques like freezing the groundwater in the fractures have also been applied for special purposes. Leakages can be limited also by constructing lining structures inside the tunnel at the intersections with major transmissive zones. Best available technology for reducing the groundwater leakages by grouting would include hydrogeological characterisation of the rock, design of the grouting techniques based on the characterisation results, grouting work according to specifications and verification of the result. Sealing (mainly grouting) may, on the other hand, also cause some potentially harmful disturbances. Chemical disturbances are associated to the use of cement (especially if low-alkaline cement can not be used) or organic grouting materials.

Inflowing amount of groundwater into ONKALO and the repository was estimated with analytical methods with and without sealing. Estimated total inflow into ONKALO without sealing was about 3000 l/min and after sealing 400...500 l/min. Construction of the repository showed only minor increase in the scale of 100 l/min to the total inflow rate. A balance needs to be found between avoiding disturbances due to leakages and, on the other hand, due to grouting and other preventing methods.

Keywords: Spent fuel, disposal, deep repository, ONKALO, sealing, groundwater inflow, grouting
VESIVUOTOJEN AIHEUTTAMAT HÄIRIÖT JA NIIDEN HALLINTA ONKALOSSA JA LOPPUSIJOITUSTILOISSA

TIIVISTELMÄ

Raportin päätaavoitteena on tunnistaa ONKALO:n ja loppusijoitustilojen vesivuotojen aiheuttamat häiriöt ja kuvata pohjaveden hallinnan vaatimat periaateratkaisut.


Vesivuotojen määrää ONKALO:n ja loppusijoitustiloihin on arvioitu analyyytisilla menetelmillä ennen ja jälkeen oletettua tiivistämistä. Arvioitu vuotovesivirtaama ONKALO:n ilman tiivistämistä on noin 3000 l/min ja tiivistäminen jälkeen 400...500 l/min. Loppusijoitustilojen rakentaminen lisää vuotomäärää varsin vähän, noin 100 l/min. Vesivuodoista johtuvien häiriöiden välttämisen ja tiivistystoimenpiteiden välillä tulee löytää tasapaino.

Avainsanat: Käytetty polttöaine, loppusijoitus, loppusijoitustilat, ONKALO, tiivistäminen, vesivuodot, injektointi
PREFACE

The present study is a part of the Groundwater Control project which supports the design of the underground rock characterisation facility ONKALO and the spent fuel repository at Olkiluoto. The work has been ordered from and supervised by Posiva Oy. The authors wish to thank Posiva’s contact persons Tapani Lyytinen, Aimo Hautojärvi, Sami Niiranen, Johanna Hansen and Jukka-Pekka Salo for their contributions.

The contributions of and fruitful conversations with many experts have also been of essential help. A special gratitude is addressed to Matti Kokko and Margit Snellman at Saanio & Riekkola Oy.
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1 INTRODUCTION

The Finnish Parliament ratified in 2001 the Government's favourable Decision in Principle (DiP) on Posiva's application to locate the repository for the spent fuel from the Finnish nuclear power plants at Olkiluoto. In May 2002, the DiP was extended to cover the spent fuel from the planned new nuclear power reactor. Around 2004 Posiva intends to start the construction of the ONKALO underground rock characterization facility as a part of the Site Confirmation Stage that will run up to the year 2010.

The main goals of this report are
• identifying possible disturbances due to ingress of groundwater into ONKALO and the repository
• identifying principles of the technical solutions for controlling groundwater leakages into ONKALO and the repository.

Inflow of groundwater can be limited
• by a careful selection of the locations of the underground facilities and their surface connections
• by forming watertight zone in the rock around tunnels and shafts. This is normally made by grouting the rock before or after excavation. More exotic techniques like freezing the groundwater in the fractures have also been applied for special purposes
• by means of lining structures inside the tunnel.

Sealing (mainly grouting) may, on the other hand, also cause some potentially harmful disturbances. Chemical disturbances are associated to the use of cement (especially if low-alkaline cement can not be used) or organic grouting materials.

Identifying of disturbances and preventing methods are dealt with in this report in the following way:
• Regulatory requirements that have influence on groundwater control are collected (Chapter 2)
• Disturbances associated to groundwater leakages are identified and evaluated from the point of view of the pre- and postclosure performance of the repository (Chapter 3)
• Techniques are described how the groundwater table and water inflow can be controlled (Chapter 4)
• Disturbances associated to grouting and other preventing methods are evaluated from the point of view of the postclosure performance of the repository (Chapter 5)
• Experiences from grouting case studies are summarized (Chapter 6)
• Water ingress into ONKALO and the repository before and after sealing is estimated (Chapter 7)
• Design principles for the sealing of ONKALO are suggested (Chapter 8).

Detailed design of sealing techniques is beyond the target of the present study. That design phase will take place when the contractual documents are prepared for ONKALO and the repository. ONKALO will be used for rock characterization as well
as for testing and demonstration of construction and disposal techniques. Development of groundwater control techniques will take place also during construction and operation of the ONKALO facility and therefore may not be considered fully matured when construction starts.

The analyses and conclusions in this report are based on the present status of site investigations and understanding of the rock properties. Consequently, estimates of inflow rates and efficiency of technical solutions can only be considered as rather rough ones.
2 REQUIREMENTS FOR CONTROLLING GROUNDWATER LEAKAGES

Regulatory requirements include some general guidance for controlling groundwater leakages. From long-term safety point of view, requirements deal with preserving natural features of the host rock and transport of substances into the repository. Leakages into ONKALO and the repository need to be controlled in a way that the host rock preserves its natural barrier characteristics. The transport into the repository of substances, which are adverse to long-term safety, such as organic or oxidizing substances, shall be limited to the minimum. (STUK 2001, YVL 8.4, Chapter 3.3)

Decisions of the Council of State (1999; VNP 478/99) includes general directions for the methods adopted for the design, construction, operation and closure stating that the methods should be well proven or otherwise carefully investigated and with high quality. Same principle is included in YVL 8.4, Chapter 3.1, where statement of introduction of the best available technique or a technique that is becoming available is made. In addition, requirement for avoidance of organic and other substances, which are adverse to long-term safety, limit choice of grouting materials.

The most important actions from the point of view of the groundwater control that follow from the regulatory requirements are:

- Disturbances caused by leakages to the geological environment as well as for implementation and operation of ONKALO and the repository shall be assessed.
- Preventive measures against these disturbances shall be identified and planned.

The following chapters from 3 to 5 concentrate on describing the disturbances and preventive measures.
3 DISTURBANCES

Construction of a large underground facility, such as ONKALO and the repository is expected to cause disturbance for the environment as well as for construction and operation of the facility. The most important disturbances are due to water ingress into the facility and concern local hydrogeological and geochemical conditions. In the case of the repository the disturbances can be divided into preclosure and postclosure disturbances. Figure 3-1 illustrates the main site scale disturbances foreseen due to the construction of ONKALO and final disposal facility. These disturbances are described in more detail in the Chapters 3.1, 3.2 and 5.

3.1 Impact of leakages for construction and pre-closure operation

During construction, rock engineering works, such as boring, charging and supporting will meet difficulties if high inflow rates are tolerated. High rate of seepage makes the properly running pumping system completely vital during construction and later during operation to avoid any flooding of the repository. In addition, structural works and installation of technical systems, such as electric and ventilation, will meet difficulties with high inflow rates in tunnels and shafts. Seepage, especially of saline groundwater creates highly corrosive environment and design of installations should take that into account for instance by using high quality stainless steel or by means of frequent maintenance.

Figure 3-1. Foreseen site scale effects due to groundwater ingress into the repository.
During disposal operations, leakages may cause additional problems by making installation of buffer and backfilling works difficult or impossible. McKinley (1997) reports that air moisture was enough to initiate swelling and break the bentonite blocks apart. In the actual case also minute leakage from a borehole (less than a few litres/24 h) caused problems. Seepage of 5 l/min over 10 m caused problems to backfill an experimental deposition tunnel at Äspö Hard Rock Laboratory and it was needed to divert the water to backfill the tunnel. It might be that all point leakages > 0.5 l/min need sealing to backfill the tunnels properly (Bäckblom 2002).

### 3.2 Postclosure effects of leakages

Inflow of groundwater into underground facilities and the subsequent pumping of the water to the ground surface during the construction and operation phase bring along flow of groundwater in the surrounding rock, movement and mixing of different water types, and drawdown of groundwater table.

Flow of groundwater into the excavations causes migration of superficial water, containing oxygen, carbon dioxide and organic substances, into the bedrock. The soil and bedrock can buffer most of the associated geochemical disturbances, but then the corresponding buffering capacity is consumed. From the point of view of the postclosure performance of the repository, the most significant impacts are related to the inflows into the deep parts of the underground facilities and to consumption of the buffering capacity of fracture fillings and rock in the vicinity of the disposal tunnels.

The drawdown of groundwater table has been estimated by Ahokas & Sallasmaa (1998). Locally, in the vicinity of the shafts and in fracture zones leaking into the excavations as well as in the bedrock above those fracture zones, the drawdown can be several tens of metres. Elsewhere, minor effects may be seen up to a distance of about one kilometre. It may be noted, that e.g. an inflow rate of 100 l/min (53 000 m³/yr) would correspond to an average infiltration rate of 17 mm/yr over an area of 3 km².

An open underground space draws groundwater from all directions. Due to the higher hydraulic conductivities more water can be expected to come from the upper parts of the bedrock than from the deep bedrock. As a consequence of this mixing, the average properties of the leakage water may not differ drastically from those prevailed at the same level before excavation. For example, according to the simulations carried out by Svensson (1997), the average salinity of groundwater leaking into the tunnel system at Äspö would be about 6.9 g/l of TDS (Total Dissolved Solids), with a maximum of 30 g/l in a major fracture zone. In the natural state the salinity at the depth of 450 metres was estimated to be about 8 g/l. The measured salinity of the inflow into the deep parts of the tunnel system (in April to November of 1995) ranged from 6 to 15 g/l (Rhén et al. 1997).

Although the average contents of the groundwater leaking into the facility may not differ so much from the natural, pre-excavation groundwater at the same level, there can be significant migration of deep, saline groundwaters towards the excavation. Groundwaters presently laying 100 to 200 metres below the disposal level may migrate in the
vicinity of the repository (Svensson 1997). This needs to be taken into account in the design of the engineered barriers (e.g. tunnel backfill) (Vieno 2000).

The upconing of deep, saline water can be also related to the drawdown of groundwater table. Measured in metres, the upconing may be several times higher than the drawdown of groundwater table. In an ideal steady state case for a coastal area, where a lens of fresh water is floating over saline water, and both zones are assumed to be homogenous and a sharp interface is assumed between the two zones, the depth of the fresh-saline water interface \((z)\) can be derived from the Ghyben-Herzberg principle (Löfman 2000, see Equation 3-1):

\[
z = \frac{\rho_o}{\rho - \rho_o} h
\]

where \(h\) is the elevation of groundwater table above sea level (m), and \(\rho_o\) and \(\rho\) are the densities (kg/m\(^3\)) of fresh and saline waters, respectively. Löfman (2000) has related the density and TDS value of groundwater by \(\rho = \rho_o + 0.71 \times \text{TDS}\). At Olkiluoto the maximum elevation of groundwater table is 10 metres above sea level. For the average salinity (42 g/l) of the groundwater between ground surface and the depth of 1500 metres of the initial salinity model used by Löfman (2000), the Ghyben-Herzberg principle would locate the fresh-saline water interface at the depth of 335 metres. According to this (ideal, steady state) approximation, the interface would thus move 170 metres upwards if the drawdown is 5 metres. (A drawdown of 10 metres would imply a more or less flat groundwater table at sea level and no bubble of fresh groundwater.) One should, however, note that in reality there is a transition zone between fresh and saline water due to diffusion and mixing caused, for example, by changes in recharge and land uplift. Also the boundary conditions are different than assumed in the simplified model described. Experiences from, for example, Aspo HRL and SFR repository show that much water flows laterally and that upconing is less than the Ghyben-Herzberg model predicts (Bäckblom 2002).

Sea water may intrude into the bedrock of the island as the facility draws groundwater from the surrounding rock and if groundwater table is drawn below sea level. As sea water is denser that the fresh groundwater in the upper parts of the bedrock, it may sink deeper into the bedrock. Sea water contains substances, e.g. sulphate and ammonia, which may be harmful for the copper canister. However, it should be borne in mind that Olkiluoto has been covered by sea for several thousands of years after the most recent glaciation. Therefore, it may be estimated that a further intrusion of sea water during the construction and operation periods of ONKALO and the repository can only make a minor impact on the groundwater mix.

The drawdown of groundwater table and the associated effects are related to the total inflow rate of groundwater into the excavations. They can thus be mitigated by limiting inflow rates in all those parts of the excavations where significant leakages would otherwise take place.

After the closing of the repository, the hydrogeological disturbances disappear soon. The repository is resaturated and groundwater table rises close to the ground surface.
Also geochemical conditions, for example the layered sequence of different types of groundwaters, return gradually towards the natural conditions. Heat generation may cause some further mixing and upward movement of groundwater from the repository. The substances (e.g. organic compounds, sulphate, ammonia, chlorine and other dissolved solids) migrated from the upper and deeper parts of the bedrock to the repository level may affect the composition of groundwater at the repository level for several hundreds or thousands of years. They shall be taken into account when the ambient postclosure conditions for the engineered barriers are estimated. In the longer term, the impacts from the inflows during the construction and operation phase are mainly limited to the potential consumption of the buffering capacity of fracture fillings and rock in the vicinity of the repository.
4 METHODS FOR CONTROLLING GROUNDWATER

4.1 Alternatives

In Chapter 3, several possible types of disturbances are described that follow from groundwater ingress into ONKALO and the repository. Preventing these disturbances necessitates that practical methods for controlling groundwater table and groundwater ingress into tunnels and shafts are identified.

In principle, there are two ways to control groundwater table:
• artificial infiltration of water into the rock and
• restricting the amount of inflowing water into the underground facility.

The artificial infiltration of groundwater does not reduce water ingress, but instead, it may limit upconing of deep saline waters to the disposal level. Artificial infiltration of water into the rock is a technique that is used in underground rock storage projects where the level of groundwater pressure around the storage needs to be maintained in a fixed level to provide sufficient pressure conditions and to prevent leakages from the storage to the environment. For infiltration a water curtain is formed where water is filtrated into the rock through holes that are bored from tunnels constructed above the storage facility. Major drawbacks of this infiltration technique are uncertainties of sufficient infiltration capacity, chemical composition of the infiltrated water, extra excavations above the repository as well as extra circulation of groundwater.

Restricting the amount of inflowing water into an underground facility can be achieved by different methods which give somewhat different results and side-effects.
• By a careful selection of the locations of the underground facilities and their surface connections,
• by constructing lining structures of concrete or steel inside the tunnels or shafts and
• by forming watertight zone in the rock around tunnels and shafts. This is most commonly made by grouting the rock before or after excavation. More exotic techniques like freezing the groundwater in the fractures have also been applied for special purposes.

Water leakages can be minimized by locating underground tunnels and shafts in as tight rock as possible. However, bedrock in the planned repository area contains several natural water conductive fractured zones and individual fractures. In practice it is impossible to avoid all intersecting transmissive zones and fractures and as a result, there will always be groundwater leakages into underground facilities. Present understanding is that in addition to favouring tight rock in locating ONKALO and the repository, the inflow rates need to be technically reduced into a very low level.
Leakage rate through a high transmissivity structure can effectively be reduced by constructing a hydrostatic lining of concrete or steel. Precondition for constructing such a structure is pregrouting of the potential leaking area. Main disadvantages of hydrostatic lining are high costs and long time schedule of construction when other work activities are reduced in the tunnel face. Deep in the bedrock the thickness of the hydrostatic lining becomes considerable (up to 2 m) and needs also enlargement of the tunnel or shaft cross sections.

The practical way to limit water ingress into underground facilities is considered to be pregrouting of the rock, which in crystalline rock types is normally used technique. In that technique, a tight zone is made in the rock around tunnels and shafts by pumping grouting material with overpressure into the fractures. Figure 4-1 illustrates the idea of pregrouting of a major fracture zone. With strict requirements for allowable water inflows, also single fractures may need to be sealed by grouting.

Another technique how a tight zone can be made in the rock surrounding tunnels and shafts is freezing of the rock. With the technique, groundwater in the fractures is freezed and thus inflow into the tunnels and shafts is prevented in these fractures. In Finland freezing method has been applied in constructing Helsinki metro through Kluuvi cleft. Freezing technique may have potential for applications in the repository construction. So far, the technique is not commonly used in rock engineering and needs further development.

It is evident that all leakages cannot in practice be technically sealed. In the underground facilities minor water drops can be redirected, if necessary, from the roof and walls by drainaged shotcrete structures or plastic membranes or other type of structural elements. This type of redirecting the leakages does not reduce their amount but can provide more favourable conditions in the tunnels or shafts for instance for electric systems.

4.2 Principal technical solutions for reducing the leakages

Dominant factor for controlling the inflow is the hydraulic conductivity of the grouted zone around the tunnel. Therefore it is important to adapt the grouting material and grouting technology to the prevailing conditions and also to ensure that the subsequent excavation does not damage the grouted zone.

Regulatory requirements as well as the present conception of the low tolerated inflow rates call for use of the best available technology in repository design and construction, especially in the sealing works. Most important sealing technique is considered to be grouting carried out as pregrouting. Hydrostatic structures can be used for sealing of highly water conductive fractured zones. Best available technology for reducing the groundwater leakages by grouting would include hydrogeological characterisation of the rock, design of the grouting techniques based on the characterisation results, grouting work according to specifications and verification of the result in the following way:
• Tunnel sections that will be pregrouted are characterised beforehand in situ by pilot holes and/or probe holes (see Figure 4-1)
• Design of the grouting fan and grouting materials is based on in situ characterisation
• Grouting materials are based on cementitious products and additives that are necessary for achieving good penetrability and sufficient open and setting times
• Grouting of very fine fractures may call for use of materials with better penetrability than cement based materials possess
• Grouting work is carried out according to specifications
• Quality of the grouting work is controlled and the work is well documented
• Results are verified.

Design and implementation of grouting works should be included in the “Observational Method” process (Bäckblom & Öhberg 2002), which shall be developed and demonstrated in ONKALO as part of “Coordination of Engineering, Investigations and Construction” (CEIC) process. In Table 4-1 are collected the principal technical solutions of sealing works at ONKALO. The preconditions leading to successful result in grouting is presented in Figure 4-2. The logic how to determine the sealing principle is simplified in Figure 4-3.

In addition to the previous description of principal technical solutions, a lot of technical issues need to be determined in the detailed design phase before construction. A special document describing the detailed sealing plan of ONKALO may be needed. Such a document would include plan for observations and technical description of the sealing works, e.g.:
• Specifications of places of and measurements at probe holes
• Drawings of grouting fans at different rock qualities
• Specifications when grouting is started and finished, when specific grouts are used and grouting pressures
• Drawings showing anticipated places of, pregrouting and postgrouting holes, hydrostatic structures and measuring weirs
• Required properties of the grouts and examples of acceptable recipes
• Specifications of possible other sealing techniques
• Specifications of quality control measurements and other actions.
Phase 1: The fracture characteristics of the bedrock are unknown. The probe holes are drilled to characterize the bedrock.

Phase 2: The fracture zone aimed to be sealed is identified and characterized, and the grouting holes are drilled.

Phase 3: The grout is injected into fractures through the grouting holes.

Phase 4: The excavation work continues after grouting materials can sustain groundwater overpressure and blasting vibrations.

*Figure 4-1. The principle of pregrouting of fractured zones.*
Table 4-1. Principal technical solutions of sealing works at ONKALO.

<table>
<thead>
<tr>
<th>TECHNIQUES OF GROUNDWATER CONTROL</th>
<th>LOCATION OF SEALING ACTIONS</th>
<th>MATERIALS</th>
<th>INVESTIGATIONS AT THE TUNNEL FACE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pregrouting of rock</td>
<td>Extensive systematic pregrouting is needed in the upper part of the access routes</td>
<td>Grouting cements and microcements with additives based on OPC products or low alkaline cements above -300 level. Use of OPC products necessitates analyses of the range and effect of the pH-plume on the rock</td>
<td>Probe holes are needed for hydrogeological characterisation</td>
</tr>
<tr>
<td>Design based on hydrogeological characterisation</td>
<td>In the middle and lower part of the access tunnel decision on pregrouting is based on measurements in the pilot or probe holes</td>
<td>Low alkaline cements at the repository level Use of silica based materials or other materials with good penetrability need to be studied further</td>
<td>Main fracture properties, such as location, transmissivity and aperture need to be determined</td>
</tr>
<tr>
<td>Result controlled by measuring the leakage rates</td>
<td>Pregrouting when required at the repository level. Decision is based on measurements in the pilot or probe holes</td>
<td>Amount of binding materials and additives need to be estimated and analysed from the point of view of long term safety</td>
<td></td>
</tr>
<tr>
<td>Hydrostatic lining at water conductive rock structures when necessary</td>
<td>All water conductive structures are pregrouted</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Disturbances continuously monitored</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 4-2. Preconditions for successful sealing by grouting.

Figure 4-3. How to end up to sealing principles.
5 POSTCLOSURE EFFECTS OF GROUTING AND OTHER PREVENTING METHODS

There are several alternative and parallel methods to limit leakages into the underground facilities. The foremost method is careful selection of the locations of the underground facilities and their surface connections.

Tight lining structures withstanding the hydrostatic pressure can be constructed in the shafts and at fracture zones in the upper parts of the bedrock. In the deep bedrock very massive or too expensive liners would be needed to prevent inflow of groundwater. Concrete can be removed from the liners before the final sealing of the repository, if that is considered necessary. It is recommended that this option is taken into account in the design of liners and other concrete structures, especially, if the structures cannot be constructed using low-alkaline concrete.

Grouting is the most common method to seal leakages into underground facilities (Sievänen 2001). Grouting is made via a large number of injection holes bored around the excavation or into an intersecting fracture zone before excavation (see Chapter 4.1). Postgrouting after excavation is clearly less efficient than pregrouting and tends mainly to redirect leakages (see Chapter 6.1.1). Grouting material (e.g. cement) is then injected via these boreholes into the rock to seal water-conducting fractures. From the point of view of the postclosure performance of the repository there are some potentially harmful effects related to grouting. Use of grouting materials containing a large amount of organic material would necessitate extensive studies to evaluate their effects. Therefore, it is recommended that such grouting materials are not used in ONKALO or in the repository.

There are also concerns related to the use of cement as grouting material, especially if low-alkaline cement can not be used. Hyperalkaline leachate from cement may significantly impair the properties of bentonite (Oscarson et al. 1997, Savage et al. 2001, Gaucher et al. 2002). Secondary minerals are formed in the interactions between the alkaline plume and fracture fillings. These minerals seal fractures and some of them have a large sorption capacity for radionuclides. On the other hand, the precipitate may also seal pores and microfractures in the surrounding rock matrix and thus reduce the effects of matrix diffusion as a retarding mechanism. Sharp gradients in pH at the edge of the plume may give rise to formation of colloids. Furthermore, the databases on the behaviour of the spent fuel, bentonite, and radionuclides in high pH is scarce. Effects of superplasticisers and other organic additives of cement are not well known.

As concerns the integrity of copper canisters, there appears to be little negative impacts from the use in the repository of cementitious materials with porefluids in the pH range 12–13 (King 2002). If the porewater pH increases prior to the establishment of anoxic conditions, the canister surface will passivate when the porewater pH value of about 9 is exceeded. The higher the porewater pH, the more strongly the copper is passivated and the less likely the canister surface is to undergo localised pitting corrosion.

Many of the problems associated with the use of cement as grouting material could be avoided, if low-alkaline (< 11 pH) cement could be used. Posiva develops low-alkaline
cement in cooperation with SKB and the Japanese NUMO. Also several other waste management organisations are developing and testing different types of low-alkaline cement. Potential non-cementitious grouts include, for example, silica sol (colloidal silica) and MgO. Effects of all materials to be introduced into the host rock shall, of course, be first studied.

Another potential problem related to the use of cement and other materials which will be degraded in the long term is that injection boreholes may then become transport pathways. This should be taken into account especially when grouting is planned to be made at locations of sealing structures.

Drawdown of groundwater table might be compensated by artificial infiltration of fresh water into the bedrock (see Chapter 4.1). Use of this method instead of grouting and liners is, however, not recommendable as it would generated extra water circulation from the ground surface to (the upper parts of) the underground facilities, and thus cause changes in groundwater chemistry.
LESSONS LEARNED FROM CASE STUDIES

6.1 Nordic experiences

This chapter summarises experiences from grouting operations in tunnels over a range of geological conditions and a range of sealing requirements. Nordic experiences are of the main interest since the bedrock is very similar than that of ours. Case studies were conducted to compile experience on:

- The reduction in tunnel water inflow/hydraulic conductivity of bedrock by use of grouting technology
- What minimum leakage that can be/has been achieved by grouting?
- What grouting materials have been used?

Short facts from several cases are collected in Tables 6-1 and 6-2.

6.1.1 Experiences of achieved water inflow

According to Pettersson & Molin (1999) in deep tunnels – or shafts – where hydrostatic heads may be 10 MPa or more, water inflows can easily be as high as several thousand liters/min.

According to Stille (2001) nowadays the acceptable water ingress of water to a tunnel in urban areas is around 1-4 l/min and 100 m of tunnel. Based on Dalmalm (2001) the maximum allowed amount of water ingress to Swedish tunnels typically vary between 0.5 and 10 l/min/100 m. In Norway, the allowable amount of water inflow to a sub-sea road tunnels is 30 l/min/100 m as a maximum, and in urban areas 2-10 l/min/100 m (Grøv 2002). In Finland the limits are similar, naturally depending on the purpose of an underground facility.

In Hästholmen VU repository (about 110 000 m³) for low and intermediate level nuclear waste located in Loviisa in South-Eastern Finland, the water inflow has diminished after construction from 300 l/min to 150 l/min in five years (Sievänen & Hagros 2002). The grouted volumes were very high, mainly done using pre-grouting. During excavation, target value for water inflow was reached in the disposal rooms but not in the access tunnel. Leakages in the access tunnel remained high after pregrouting. It is unclear how much the leakages could have been reduced if more grouting (time and money) had been made. For instance if the distance between the grouting holes had been smaller, grouting materials with better penetrability had been used or if the number of the grouting fans had been greater.

In Olkiluoto VLJ repository (about 90 000 m³), nearby the planned ONKALO and the final repository, in Western Finland the total inflow is measured to be about 45 l/min (Sievänen & Hagros 2002). Two thirds of this water comes from one fracture zone, and thus the leakage into the rest of the tunnels is about 1 l/min/100 m of tunnels. Only minor postgrouting work was carried out and the main conclusion of the work is the well known tendency that postgrouting mainly redirects the leakages. The experience also highlights the need for investigating beforehand the properties of hydraulically
important zones in order to give sufficient information for the decision of possible pregrouting.

At Leppävaara underground car park excavated in migmatitic mica gneiss and quartz-feldspar gneiss leakages were reduced by pre- and postgrouting with cement to 60 -100 l/min which corresponds 3 – 5 l/min/100 m tunnel). Grouting works could be designed well beforehand because rock mass was successfully characterised by geological pre-investigations. Probe drillings were used successfully. After the grouting works had been completed separate excavation was carried out on the ground surface for foundations of apartment buildings. This resulted in increasing leakage rates and 200 plates were installed for drop leakages (Kalliomäki 2002).

Total allowed inflow into Merihaka rock shelter (70 000 m³) situated in migmatitic micagneiss with pegmatitic veins was set to 39 l/min and this was clearly met using extensive pregrouting with cement. Total leakages at the end of year 2000 were in the order of some litres per minute. Sealing works succeeded well inspite of highly water conductive horizontal fractures and fracture fillings. Probe drillings were used successfully. (Kalliomäki 2002.)

Excavated volume of Helsinki university library is 80 000 m³. Hydraulic conductivity of the rock was estimated based on hydraulic testing to be 3·10⁻⁸ m/s. Total water inflow was estimated to be in the order of 10 l/min. The target value was set to be 7 l/min (2.7 l/min/100 m), which was met with inflow after construction being 6.7 l/min. Probe drillings were used successfully. The excavation did not have significant influence on groundwater table. Some infiltration was used (Sievänen & Hagros 2002).

Turku-Naantali district heating tunnel the required inflows were usually reached (Gardemeister & Koskiahde 1984) with relatively minor groutings; about 10% of the bedrock was pregrouted and most of those areas needed postgrouting (too short hardening time, too few probe drillings and water loss measurements). A typical problem in postgroutings was that the leakages moved to another place. The used grouts were both cementitious and chemical. In the area with limitations of less than 3 l/min/100 tunnel-m the measured inflows varied between 0.5 and 4 l/min/100 tunnel-m. There was one area with exceptionally high inflow, about 10 l/min/100 m. The average was 2.5 l/min/100 m. In the area of < 5 l/min/100 m the measured water inflow varied between 1 – 5 l/min/100 m. In the area of < 10 l/min/100 m, the measured inflows varied typically between 1 – 8 l/min/100 m, the average being about 3.5 l/min/100 m.

The demand for allowed water inflow in the Bolmen water supply tunnel in Sweden was set to be 9 l/min/100 tunnel-m in the beginning of the excavation (Bäckblom 2002). Later the limit was set to be 42 l/min/100 tunnel-m. 25% of the tunnel was grouted and the total average inflow to the tunnel was 43 l/min/100 tunnel-m, which corresponds to hydraulic conductivity of about 0.8 ·10⁻⁷ m/s. Like typically in water supply tunnels, the requirement were not as tight as in tunnels located in urban areas. On the contrary, Lundby road tunnel, the requirements were very tight, varying between 0.5 - 2.5 l/min/100 tunnel-m. The total average was 0.9 l/min/100 tunnel-m, and the target was reached nearly everywhere.
In the Äspö Hard Rock Laboratory the total inflow after (restricted) grouting is 2100 l/min, which makes up about 60 l/min per 100 m (Hedman 1999). The passage of one water bearing fracture zone NE-1 was studied. This zone was located at a depth of about 190 m below sea level. Before entering the zone the measurements indicated transmissivities even of about $1 \times 10^{-4}$ m/s in the nearby rock. It was estimated that, without sealing, the inflow to the tunnel would have been up to 6000 l/min from this zone (SKB 1996).

In Baneheia road tunnel in Norway the water inflow was successfully diminished from the estimated 150 l/min/100 tunnel-m to 3 l/min/100 tunnel-m and in Storhaug road tunnel from 70 l/min/100 tunnel-m to 1 l/min/100 tunnel-m (Statens Vegvesen 2001 referred by Riekkola et al. 2002). In these two cases the sealing was done as continuous pregrouting. In Akrafjorden subsea road tunnel the estimated water inflow without grouting was as high as 3000 – 5000 l/min/100 tunnel-m and after grouting 25 l/min/100 tunnel-m. Short facts are collected to Table 6-1. More information of those Norwegian cases can be found for example in (Davik & Andersson 2002).

In Romerikspoorten railway tunnel the requirements for water inflow were similar than usually in Norway (Beitnes 2002). In that project it was noticed that the biggest problems arose with the floor section. The major part of excessive leakage came through the floor section, much due to a thin or insufficient pregrouting fan and later blasting for the drain ditch.

6.1.2 Experiences of achievable sealing efficiency, achieved hydraulic conductivity and sealed fracture apertures

For the normal variation of the rock mass permeability in Swedish rock, which is greatly similar to Finnish bedrock, the sealing effect is around 90 to 99% for the tunnels located down to a depth around 150 m (Stille 2001). According to Stille (2001), the achievable permeability of the grouted zone is around 1 Lugeon or 0.3 to $3 \times 10^{-7}$ m/s for ordinary cement grout of today. To achieve the lowest value special cement types have to be used. This is acceptable for very shallow tunnels with a low pressure head and when allowable ingress of water is higher than around 2 l/min/100 tunnel-m. For all other occasions like deeper tunnels or at very low demands for shallow tunnels, much higher demands must be put on the grouted zone. At least ten times lower value on the permeability of the grouted zone is required corresponding 0.5 $\times 10^{-8}$ m/s. It is therefore obvious that special attention has to be paid to the penetrability of the cement suspension into very fine joint apertures.

According to studies by Dalmalm (2001) to reduce flow with 90% in a rock mass, joints down to an aperture between 110 – 440 μm need to be partially sealed. To reduce flow 99% in a rock mass, joints down to an aperture between 50 – 200 μm need to be sealed. He also states that a sealing up to 90% is normally enough to seal all shallow located tunnels and up to 50 m located with an initial conductivity of $1.7 \times 10^{-6}$ m/s. For more permeable rock masses and/or for deeper location, hence higher water pressure, the sealing efficiency will need to be higher.
In Södra Länken road tunnel’s grouting trials, the fractures of a hydraulic aperture lower than 100 μm were neither succeeded to be sealed with Injektering 30 cement nor microcement (9.5 μm) (Dalmalm et al. 2000).

Stille (2001) also summarises the experiences from Swedish tunnels: the first pregrouting operation may reduce the permeability to around $1 \times 10^{-7}$ m/s. The first regrouting can further reduce the permeability to around $0.5 \times 10^{-7}$ m/s, and further regrouting operations will only marginally reduce the permeability further down to around $0.3 \times 10^{-7}$ m/s. According to Stille (2001), generally experiences of postgrouting are poor.

Dalmalm (2001) refers to the several experiences gathered by Vägverket (1993) that with normal cement grouting and without additives it is possible to reach a sealing of a grouted zone down to about $0.5 - 1.5 \times 10^{-7}$ m/s. To further seal the rock mass, other grouting techniques or chemical grouting techniques have to be used. In general when grouting with a cement, a reduction of the initial conductivity with approximate a factor 10 could be expected and the lowest possible expected conductivity could reach $5 \times 10^{-8}$ - $5 \times 10^{-7}$ m/s (Kutzner 1996, referred by Dalmalm 2001).

In Romeriksporten railway tunnel in Norway it has been found that by use of stable microcements a maximum effort could bring the average permeability in fracture zones down towards some $5 \times 10^{-8}$ m/s corresponding to a hydraulic diameter of the remaining conductive fractures about 100 μm in moderately fractured rock (Beitnes 2002).

Experiences from grouting tests in Stripa mine (Pusch 1992, Pöllä et al. 1994) gave much encouraging sealing results: even rock mass of a hydraulic conductivity between $1 \times 10^{-10} - 1 \times 10^{-8}$ m/s were possible to be sealed depending greatly on the fracture characteristics (fracture density and infillings). In the grouting of a natural fracture zone they succeeded to reduce the hydraulic conductivity from $1 \times 10^{-8}$ to $1 \times 10^{-9}$ m/s. At the Stripa project microcement with a superplasticizer was used as well as static and dynamic grouting pressures.

The inflow of groundwater can be as low as around 1 l/min per 100 tunnel-m for a large cement-grouted tunnel (section area 100 m$^2$) excavated some 10-50 metres below the water table. This would indicate a grouted zone in the order $5 \times 10^{-9}$ m/s, but this is no verified value (Bäckblom 2002).

6.1.3 Experiences of drawdown of groundwater table and groundwater movements

Groundwater around the VLJ-repository at Hästholmen in Loviisa in Finland has become more saline and during construction the interface of the fresh and saline groundwater has risen about 10 – 50 m. However, due to diminishing amount of leakages after construction, the interface of fresh and saline groundwater is slightly lowering.

Experience from the Äspö HRL also shows that oxygenated groundwater should not reach deeper than ~ 100 m as long as the amount of organic matter in the groundwater is

Swedish experience from Äspö Hard Rock Laboratory, in Southern Sweden, and in the SFR is that much water flows laterally and that up-coning is less than the Ghyben – Herzberg model calculates. It is imperative to calibrate and run regional and semi-regional models as one basis for deciding grouting requirements (Bäckblom 2002).

6.1.4 Experiences of long term behaviour

It has been observed that seepage decreases with time for quite a few underground facilities (Bäckblom 2002), c.f. Table 6-1. The table summarizes data from a few underground facilities in Canada, Finland and Sweden. Decrease in seepage is in the order of 0.3 – 1.1 % per month. Several explanations are offered for the decrease, like precipitation of calcite and/or bacteria, degassing and rock creep etc., but no generally accepted explanation is presented.

It is possible that the seepage at SFR, Äspö and Loviisa VLJ repository will level out in the future, like at the URL. The Table 6-1 shows e.g. the seepage to the URL shaft during the 10 year period 1984 – 1994. There is first a transient during 1986 when seepage is down to 20 m³/day from 30 m³/day, and then there is a slow decrease until it stabilizes around 10 m³/day some 5 years after the excavation was finished.

There are also underground facilities, where there is no or very limited decrease in seepage with time (Lundby Tunnel, Olkiluoto VLJ-repository).

Table 6-1. Decrease in seepage for selected underground facilities (Bäckblom 2002).

<table>
<thead>
<tr>
<th>Facility</th>
<th>Seepage in l/min at a certain date</th>
<th>Seepage in l/min at a certain date</th>
<th>Decrease l/min, month % per month</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CANADA</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>URL</td>
<td>14 l/min; Jan 1987</td>
<td>7 l/min; Jan 1991</td>
<td>0.2 l/min, month 1.0%/month</td>
</tr>
<tr>
<td><strong>FINLAND</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loviisa VLJ</td>
<td>300 l/min; Jan 1996</td>
<td>140 l/min; Jan 2000</td>
<td>3.3 l/min, month 1.1 %/month</td>
</tr>
<tr>
<td><strong>SWEDEN</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAB</td>
<td>56 l/min; Jan 1987</td>
<td>32 l/min; Jan 1999</td>
<td>0.2 l/min, month 0.32 %</td>
</tr>
<tr>
<td>SFR</td>
<td>720 l/min; Jan 1988</td>
<td>400 l/min; Jan 2000</td>
<td>2.2 l/month; 0.31 %</td>
</tr>
<tr>
<td>ÅSPÖ HRL</td>
<td>2100 l/min; Jan 1995</td>
<td>1490 l/min; Jan 2000</td>
<td>10.1 l/month; 0.48 %</td>
</tr>
</tbody>
</table>
6.1.5 Miscellaneous experiences

In Norway systematic probing/grouting schedule is normally included in the tunnelling procedure in such a way as to minimise the delay and maintain the high tunnelling advance rate (Grøv 2002).

According to Karlsrud (2002) pre-grouting carried out with maximum efforts can in many cases give an acceptable leakage level with reasonable costs; experiences from Oslo show that the cost of systematic pre-grouting adds 50 – 70% to the excavation costs (without any grouting). This cost is still far lower than establishing a permanent watertight lining, which may add another 100 – 150% to the excavation cost (Karlsrud 2002). Also Roald et al. (2002) emphasise the cost and schedule impact of pregrouting. The cost of excavation is highly dependent on rock quality; poor rock not only increases the cycle time but also reduces the length of each round. Since grouting improves rock quality, the money spent on grouting is paid back by cheaper excavation and rock support. This was experienced for example in Banheia tunnel and Storhaug road tunnel, both located in Norway.

Davik & Andersson (2002) state based on their case studies that even for tunnels with moderate water inflow requirements, sporadic grouting efforts based on leakage from probe holes were insufficient. The use of standardised systematic grouting schedule throughout the tunnel, was pointed as most advantageous for the excavation cycle and the inflow of water.

The cost of pumping water from the repository is in fact quite high as the water need to be lifted around 500 m for a period up to hundred years. In addition to energy cost there are substantial costs to maintain the main pumps, auxiliary pumps, the pipes and valves. This cost can be optimised against the cost of more sealing work (Bäckblom 2002).

In Nordic countries grouting fans consist typically some tens of grouting holes designed so that the thickness of the grouted zone would be at least 5 m. Challenging conditions may require as much as 100 holes per fan (Beitnes 2002). Experts start more and more to recommend split-spacing of boreholes, i.e. that grouting is made in two rounds; the first is for sealing the major seepage (Bäckblom 2002). The second round holes are drilled in between the previous holes as control holes and these holes are grouted depending on the recorded water-loss measurements. Careful blasting is essential, mostly in the tunnel floor and the procedures at Lundby tunnel and Södra Länken should be honoured (Bäckblom 2002).

In poor rock it is difficult to drill longer holes than 20 m and shorter fans are recommended. Grouting should start with holes having the largest seepage/water-loss. In the complicated rock, holes should be grouted one by one in spite of the short-term impact on advance rate (Bäckblom 2002).

Experience at e.g. Hallandsås shows no disadvantage with using as high pressures as 5 MPa over the ambient groundwater pressure (Bäckblom 2002). Instead, in Södra Länken road tunnel higher grouting pressure (4.5 MPa) did not result to better sealing
than that in the bid of the contract offer, which was 2.5 MPa (Dalalm et al. 2000). High grouting pressures are very customized in Norway. It is also a general observation that the use of high grouting pressures (at least 3 – 4 MPa) enhances the grout take and the achieved reduction in permeability (Karlsrud 2002). On the other hand, even as high as 9 - 10 MPa pressures are used (Davik & Andersson 2002, Riekkola et al. 2002). The reason for better penetrability is that the use of high pressures cause hydraulic fracturing and thus, give easier contact with the most water-bearing channels even if a grout hole is not in direct contact with such channels (Karlsrud 2002). The use of high pressures may therefore to some extent allow fewer grouting holes to achieve the same result than when more moderate grouting pressures (1 – 3 MPa) are used.

Bäckblom (2002) comments that besides additional requirements on the grouting material, there are problems setting packers at high pressures. In poor rock it could be needed to install steel casings to mount the packers, c.f. the design at Äspö HRL. Also the drilling rig needs high-pressure pumps for flushing; if not water flows through the drill rod into the machine instead of the other way around.

According to Bäckblom (2002) several opinions exist. If pressure increases, the same w/c is used to maximum pressure. If no pressure increase develops, the w/c is lowered by and by depending on grout take. An internationally acclaimed method is to use the Grouting Intensity Number, (Lombardi & Deere 1993), and (Brantberger et al. 2001). The GIN-value is \( p \cdot V \) where \( p \) = grouting pressure at zero grout take and \( V \) = volume of grout at zero grout flow. Grouting continues if \( p < p_{\text{max}} \); \( V < V_{\text{max}} \) and for a constant product of \( p \cdot V \). Example: At Hallandsås the \( p_{\text{max}} \) is suggested to 5.5 MPa and \( V_{\text{max}} \) to 25 kg/m borehole and the suggested GIN-value is \(~275 \sim 340\).

Also Bäckblom (2002) states that the experts do not agree on the importance of cautious blasting not to destroy the grouted zone. Bäckblom (2002) concludes that careful blasting is essential, mostly in the tunnel floor and the procedures at Lundby tunnel and Södra Länken should be taken into account. However, it is more difficult to achieve smooth blasting in a declining ramp as more energy is needed to lift the rock debris out from the tunnel face.

When only minor seepage is permissible it is difficult to show compliance during the period of construction. Problems have been encountered with installations of measurement weirs as they are not constructed in due time. It is also very difficult to measure small seepage and deduct the production water, leaking temporary pipes and valves.

According to Bäckblom (2002) when ambient water pressures are high > 100 m, the early strength development is important so that packers can be removed without the grout extruding the grout holes. The combinations of low early yield strength for penetration and high strength for removal of packers calls for use of accelerators. CaCl\(_2\) has been used as the accelerator at the Äspö HRL.

According to Johansen (2002) water leakage rates down to 4 l/min/100 tunnel-m is a very low figure indeed and requires grouting efforts using ultrafine microcements, modern grouting equipments and high grouting pressures. Several experts presently
favour the Cementa Injekteringscement 30 µm (Bäckblom 2002). The more fine cements create practical problems like creation of aggregates. Alternative grout materials have been tested systematically. The uses of colloidal silica or cement foam are interesting developments in progress. The use of extremely fine-grained cements (12-16 µm) is complicated. It should be kept in mind that the cost of microcements may typically be 3 – 4 times higher than Standard Portland cements (Karlsrud 2002). According to Davik & Andersson (2002) enhanced use of superplasticizers and silica additives have increased penetrability and pumpability for both grouting and microcements.

It has been difficult to reproduce laboratory data in field mixers for very fine cements. The common equipment for mixing is the colloid mixer. Good dispersion is achieved, but due to the high-energy input, temperature is increased that may start creation of unwanted agglomerates (Bäckblom 2002). During the trials at Södra Länken road tunnel it was noted that the mixing of the cement suspension was much more important than earlier understood, especially for the microcement types (Dalmalm 2001).

Experts have strong opinions on whether grouting should start with stable grouts or not (Bäckblom 2002). According to Davik & Andersson (2002) better penetrability has made it possible to use low w/c ratios, which in turn has improved the quality of the grout, and increased pumping capacity in dry cement. Many practicing engineers favour starting with high water-cement ratio and then decrease the w/c with time. However, many think that such a practice should not be applied.

Roald et al. (2002) have presented that grouting improves rock quality; in poor rock the effect may be even 2 – 3 quality classes (Q-classification). Also experience from Storhaug tunnel proves that the grouting efforts have a major effect on the rock mass stability, reducing the need for rock support (Davik & Andersson 2002).

Watertight concrete linings may be required as a complementary sealing measure besides grouting. A concrete lining itself is not watertight even if it is equipped with waterstops in the joints due to cracks (shrinkage or temperature changes) (Karlsrud 2002). Observed leakage through concrete linings without any special measures other than waterstops in the joints has ranged between 10 and 40 l/min/100 tunnel-m. That means that this type of lining has practically no effect at all on the leakage level.

At Oslo railway and subway tunnels reinforced concrete linings were used (Karlsrud 2002). Systematic contact grouting was introduced followed by relatively high pressure grouting in the interface between the lining and the rock to reduce the leakage through the lining. This turned to be quite successful and leakage in these tunnels have been measured to less than 1 l/min/100 tunnel-m. That is estimated to be significantly less than what has so far been achieved in only pregrouted tunnels.

In Festningen road tunnel an un-reinforced concrete lining was chosen with local grouting/sealing of cracks/leaks where and when they would appear (Karlsrud 2002). It turned out to be extremely difficult to stop the leakage in this case, primarily because of the continuous cycles of temperature changes causing grouted cracks/joints to reopen or
new ones to appear over time. In Festningen tunnel it has therefore been necessary to regroup old and new cracks many times over the years the tunnels has been in operation. Furthermore, the leakage still corresponds to about 6 l/min/100 tunnel-m.

According to Karlsrud (2002), the construction of conventional concrete lining takes a lot of time and interferes considerably with progress of blasting and excavation works if it is carried out close to the tunnel face. In practice, the lining will often trail 100 m or more behind the tunnel face. Also a lining will always have to be combined with pregrouting and possibly also with groundwater recharge wells until the lining is in place and has been made sufficiently watertight.

Especially what concerns nuclear waste disposal, experience from the FEBEX-experiments and from Åspö HRL shows that only a small fraction of seepage will make it difficult to emplace buffer and backfill (Bäckblom 2002).

Even when using environmental-friendly products it is a good idea to find a good discharge recipient. At Åspö HRL the diverted water is mixed in the outlet water from the nuclear reactor O II (Bäckblom 2002).

6.2 Conclusions of the case studies

The opinions of difficulties in rock grouting are generally shared but the methods to control the groundwater during tunnelling is under wide discussion and research. The importance of water tightness in tunnels has risen and thus development in R&D have been achieved – at least the basic understanding has increased a lot during the latest decade.

Grouting is a complicated engineering activity that should optimise grouting material and grouting technology to the heterogeneous geological conditions for a range of seepage requirements. However the current status of R&D is not sufficient to fully understand the properties of the grouting material that control its penetrability.

The grouted zone will be in the range of $10^{-7}$ to $10^{-8}$ m/s using a cementitious grout material, the results being dependent on geological conditions, choice of cement and methodology used. This corresponds to some - several litres of water per minute per 100 m of tunnel. Much effort has to be put to achieve better results. That probably requires comprehensive preinvestigations, and the use of advanced grouts and grouting techniques. The most optimistic estimations of achievable hydraulic conductivity is between $1\cdot10^{-9}$ - $1\cdot10^{-8}$ m/s, so the inflow of groundwater can be as low as around a few l/min per 100 tunnel-m for a cement-grouted tunnel excavated some 10-50 metres below the water table.

Based on several experiences the water inflow usually decreases with time, the decrease being around 0.3 – 1% per month. Groundwater table is to be lowered, even remarkably. This may lead to upconging of saline waters, as was observed in Hästholmen VLJ repository and Åspö HRL.
<table>
<thead>
<tr>
<th>Purpose</th>
<th>Depth (m)</th>
<th>Volume (m³)</th>
<th>Length (m)</th>
<th>Required inflow (l/min)</th>
<th>Achieved inflow (l/min)</th>
<th>Pregrouting</th>
<th>Pressure (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Olkiluoto, Waste disposal</td>
<td>95</td>
<td>90.000</td>
<td>1000</td>
<td>-</td>
<td>45 l/min</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>Hästholmen, Waste disposal</td>
<td>119</td>
<td>110.000</td>
<td>-</td>
<td>195 l/min</td>
<td>150-300 l/min (1996)</td>
<td>Portland ja Rapid, w/c=2/1-3/1</td>
<td>0.4-5</td>
</tr>
<tr>
<td>Tärku – Naantali, heating</td>
<td>-90 - +26</td>
<td>?</td>
<td>14000</td>
<td>3-10 l/min/100 m</td>
<td>350-490 l/min</td>
<td>Rapid, w/c=2/1-4/1</td>
<td>1-3</td>
</tr>
<tr>
<td>Leppävaara, Parking and civil shelter</td>
<td>-1 - 3.5</td>
<td>106.000</td>
<td>-</td>
<td>-</td>
<td>60-100 l/min</td>
<td>Rapid/Rheocem 650</td>
<td>0.5-1</td>
</tr>
<tr>
<td>Merihaka, Sport and civil shelter</td>
<td>-20</td>
<td>72.000</td>
<td>-</td>
<td>1-20 l/min, whole facility</td>
<td>0.2 l/min/100 m</td>
<td>Microcem. Rheocem 650</td>
<td>0.2 - 1</td>
</tr>
<tr>
<td>Kluuvi Bookstore</td>
<td>-24 - 7</td>
<td>80.000</td>
<td>-</td>
<td>7 l/min</td>
<td>6.7 l/min</td>
<td>Rapid, w/c=1/1-3/1</td>
<td>0.5-1.5</td>
</tr>
<tr>
<td>Bolmen, Water supply</td>
<td>?</td>
<td>?</td>
<td>80000</td>
<td>9-42 l/min/100 m</td>
<td>43 l/min/100 m</td>
<td>SH-sem., 128 µm+bent., w/c=3-0.5125 kg/tunneli-m.</td>
<td>2.5</td>
</tr>
<tr>
<td>Lundby Road</td>
<td>5 - 40 m</td>
<td>405.000</td>
<td>4400</td>
<td>0.5-2.5 l/min/100 m</td>
<td>0.5-1.5 l/min/100 m</td>
<td>Injektering 30 µm</td>
<td>Stopping pressure 2, 0.5 MPa shallow</td>
</tr>
<tr>
<td>Göteborg, Arlanda Railway</td>
<td>?</td>
<td>800.000</td>
<td>10 km</td>
<td>5 l/min/100 m</td>
<td>?</td>
<td>Rheocem 800. 2.020.000 kg</td>
<td>0.15 – 4</td>
</tr>
<tr>
<td>Stockholm, Arlanda Railway</td>
<td>?</td>
<td>800.000</td>
<td>3 stations</td>
<td></td>
<td></td>
<td>Rheocem 800. 2.020.000 kg</td>
<td></td>
</tr>
<tr>
<td>Road</td>
<td>Distance (m)</td>
<td>Volume (m³)</td>
<td>Flow (l/min/100 m)</td>
<td>Injecting Rate (l/min/100 m)</td>
<td>Type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>--------------</td>
<td>-------------</td>
<td>--------------------</td>
<td>-------------------------------</td>
<td>--------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Södra Länken</td>
<td>4500</td>
<td>1-4</td>
<td>1-1.5</td>
<td>Injektering 30 μm</td>
<td>Microcem., w/c&lt;2-1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stockholm, Sweden</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storhaug</td>
<td>110.000</td>
<td>3/10</td>
<td>1.6</td>
<td>Ultrafin 12, w/c=1.1-0.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stavanger, Norway</td>
<td></td>
<td></td>
<td></td>
<td>+GroutAid, SP40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Baneheia, Kristiansand, Norway</td>
<td>44-87</td>
<td>3000</td>
<td>1.8</td>
<td>Ultrafin 12, w/c=0.9-0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Norway</td>
<td></td>
<td></td>
<td></td>
<td>+GroutAid, SP40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bragernes, Norway</td>
<td>72-83 m²</td>
<td>2310</td>
<td>8-25</td>
<td>Rapid+Rescon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Norway</td>
<td></td>
<td></td>
<td></td>
<td>w/c=1-0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Täsen, Norway</td>
<td>65-80 m²</td>
<td>1860</td>
<td>25.7</td>
<td>Rapid+Rescon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>w/c=2-1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
7 ESTIMATION OF LEAKAGE RATES

7.1 Location of ONKALO and the repository

7.1.1 Layout

The 3D-figure of the design of ONKALO at main drawing stage is presented in Figure 7-1. An alternative location of ONKALO and the repository is presented in Figure 7-2. The estimations for the amount of water inflow into ONKALO and the repository are presented in Chapter 7.2. The calculations are performed for so called near-field alternative location of ONKALO access tunnel, which is foreseen to be also the access tunnel to the repository.

Figure 7-1. 3D-illustration of ONKALO.
7.1.2 Location in the bedrock

The ONKALO access tunnel will intersect geological structures or hydraulic conductors R24, KR4_IH, R19A, R19B, R46, R20A, R20B, R17B and R56. The measured transmissivities of these structures are collected in the Table 7-1 and the intersections are presented in the Table 7-2. Besides these intersections the access tunnel is in the vicinity (< 20 m) of the structures R19B at the depth interval -30...-50 m and R20B at the interval -380...-410 m. More detailed description of the structures is presented in Appendix 1. The horizontal and vertical sections are presented in Appendix 2.

The calculations concerning the inflows from intact rock mass are based on the hydrogeological description of the bedrock according to the bedrock model 2001/2 (Saksa et al. 2002 based on Vaittinen et al. 2001). The analysis includes 12 investigation holes. Some changes are presented in the recent bedrock model version in the nearfield of investigation hole KR8, but in this study those changes are regarded insignificant due to the large amount of data of intact rock mass. Instead, as new data as possible is used when calculating inflows from R-structures.
Table 7-1. Transmissivities of R-structures to be intersected by ONKALO access tunnel, research tunnels and shaft. If several measurements, the T-value used in the calculations is also presented. Estimated logT after grouting is also presented.

<table>
<thead>
<tr>
<th>R-structure</th>
<th>Measured Log T</th>
<th>Log T used in calculations</th>
<th>Estimated logT after grouting</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>R24</td>
<td>-5.5</td>
<td></td>
<td>-7</td>
<td>Difficult fracturing</td>
</tr>
<tr>
<td>KR4_1H</td>
<td>-5.2</td>
<td></td>
<td>-7</td>
<td>Intersection at a difficult angle</td>
</tr>
<tr>
<td>R19A</td>
<td>-5.0</td>
<td></td>
<td>-8</td>
<td>1 fracture</td>
</tr>
<tr>
<td>R19B</td>
<td>-5.8...-5.0</td>
<td>-5.4</td>
<td>-6 (1. Intersection)</td>
<td>Gently dipping, difficult fracturing, intersection at a difficult angle</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-7 (2. Intersection)</td>
<td></td>
</tr>
<tr>
<td>R46</td>
<td>-5.6</td>
<td></td>
<td>-6</td>
<td>Gently dipping, difficult fracturing, intersection at a difficult angle</td>
</tr>
<tr>
<td>R20A</td>
<td>-4.6...-4.9</td>
<td>-4.7</td>
<td>-6</td>
<td>Gently dipping, difficult fracturing, intersection at a difficult angle</td>
</tr>
<tr>
<td>R20B</td>
<td>-5.5...-5.2</td>
<td>-5.3</td>
<td>-6</td>
<td>Gently dipping, difficult fracturing, intersection at a difficult angle</td>
</tr>
<tr>
<td>R56</td>
<td>-8.2</td>
<td></td>
<td></td>
<td>Grouting not possible</td>
</tr>
<tr>
<td>R17B</td>
<td>-8.1</td>
<td></td>
<td></td>
<td>Grouting not possible</td>
</tr>
</tbody>
</table>

*hydraulically conductive fracture, not included in the bedrock model 2001/2

7.2 Water ingress

7.2.1 Inflow into the ONKALO access tunnel, main characterisation level and lower characterisation level

The analytical calculations are above all rough, meant for design purposes and uncertainties are high as always when estimating inflowing waters to tunnels.

The water inflow from the intact rock mass

The data for calculations of water inflow from intact rock mass are based on the distribution of hydraulic conductivities at 100 m depth intervals (Sievanen 2003). The estimations are made assuming the situation without grouting and after certain grouting result. Calculations are based on the work by Sievanen (2003). A short description of calculations is presented here. Uncertainties in the calculations (methods, assumptions and basic data) are dealt with in the studies by Sievanen (2001, 2002 and 2003).

The water inflow from intact rock mass is a sum of leakage rates per 100 metre of tunnel at different depth intervals. When studying the effect of grouting, it was assumed that rock masses exceeding the hydraulic conductivity $K_{2m} = 1 \cdot 10^{-8}$ m/s ($T = 2 \cdot 10^{-8}$ m/s²), are grouted down to that limit. This limit has to be understood as an averagely aimed sealing result. The properties of fracturing, which affect a lot groutability, were not able to be taken into account. This limit can generally be regarded relatively strict grouting result with cementitious grouts.
The inflow into ONKALO (the access tunnel and the interconnecting tunnel (4450+1050 m, main characterisation level 1200 m, demonstration level 350 m (at a depth of -420 m) and lower characterisation level about 300 m) from intact rock mass make up about 475 l/min (about 6 l/min/100 m) without grouting. Of this, the inflow into the access tunnel is about 430 l/min, to the interconnecting tunnel about 7 l/min, to the characterisation tunnel 8 l/min, to the demonstration level 3 l/min and to the lower characterisation level 20 l/min. After grouting the total inflow from intact rock mass was estimated to be 165 l/min (about 2 l/min/100 m). The tunnels between the depth interval -400...-500 m are in these calculations estimated not to be groutable due to very low initial hydraulic conductivity. If the grouting requirement for the intact rock mass would be about 50 l/min, the hydraulic conductivity after grouting should be about $5 \times 10^{-9} \text{ m/s}$, which is generally regarded as very strict grouting aim for cementitious grouts.

**Water inflow from fracture zones**

The inflow from the geological structures to be intersected by the tunnels were calculated separately in this work, but with similar methods as in the work by Sievänen (2003). The calculations were made assuming the cases without grouting and assuming certain sealing effect, and also assuming that the structure is intersected perpendicularly. It should be noted that if the structure is intersected nearly parallel— as is the case with some structures—the inflows will be somewhat higher and grouting much more difficult. Based on general experiences the sealing effect in the fracture zones is about 90-99%. The amount of water inflows from all structure intersections are presented in Table 7-2. The influence radius of the flow was estimated to be 100 m and the radius of the tunnels 3 m.

**Table 7-2. The water inflow from geological structures without and after grouting, and the limits for generally achieved grouting results (90-99%) in fracture zones.**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Intersection depth (m)</th>
<th>Water inflow without grouting (l/min)</th>
<th>Water inflow after grouting (l/min)</th>
<th>90-99% sealing effect (l/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R24</td>
<td>-5</td>
<td>1.7</td>
<td>0.1</td>
<td>0.2...0</td>
</tr>
<tr>
<td>R19B</td>
<td>-40</td>
<td>17.1</td>
<td>0.4</td>
<td>1.7...0.2</td>
</tr>
<tr>
<td>KR4_1H</td>
<td>-60</td>
<td>40.7</td>
<td>0.1</td>
<td>4.1...0.4</td>
</tr>
<tr>
<td>R19A</td>
<td>-80</td>
<td>86.0</td>
<td>8.6</td>
<td>8.6...0.9</td>
</tr>
<tr>
<td>R19B</td>
<td>-100</td>
<td>42.8</td>
<td>10.7</td>
<td>4.3...0.4</td>
</tr>
<tr>
<td>R19B</td>
<td>-160</td>
<td>68.5</td>
<td>1.7</td>
<td>6.8...0.7</td>
</tr>
<tr>
<td>R46</td>
<td>-280</td>
<td>75.6</td>
<td>30.1</td>
<td>7.6...0.8</td>
</tr>
<tr>
<td>R20A</td>
<td>-290</td>
<td>622</td>
<td>31.2</td>
<td>62.2...6.2</td>
</tr>
<tr>
<td>R20B</td>
<td>-300</td>
<td>162</td>
<td>32.3</td>
<td>16.2...1.6</td>
</tr>
<tr>
<td>R56</td>
<td>-310</td>
<td>0.2</td>
<td>0.2</td>
<td>Not groutable</td>
</tr>
<tr>
<td>R20B</td>
<td>-400</td>
<td>216</td>
<td>43.0</td>
<td>21.6...2.2</td>
</tr>
<tr>
<td>R17B</td>
<td>-420</td>
<td>0.4</td>
<td>0.4</td>
<td>Not groutable</td>
</tr>
<tr>
<td>R17B</td>
<td>-520</td>
<td>0.4</td>
<td>0.4</td>
<td>Not groutable</td>
</tr>
<tr>
<td>R56</td>
<td>-520</td>
<td>0.4</td>
<td>0.4</td>
<td>Not groutable</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1330</td>
<td>160</td>
<td></td>
</tr>
</tbody>
</table>
Total inflow into the tunnels

The cumulative water inflow (intact rock and structures) into the access tunnel as a function of the tunnel length (pale number) is presented in Figure 7-3. Together, with structures, the total inflow into the access tunnel was estimated to be about 1750 l/min without grouting. The major inflows come from structures R20A and R20B. The other parts of ONKALO will not intersect major fracture zones (only R17B and R56) and inflows from these zones are negligible.

Assuming that the intact rock mass exceeding the hydraulic conductivity of $K_{2m} = 1 \cdot 10^{-8}$ m/s is grouted to that limit and the transmissivity of the most conductive structures are reduced by as described above, the total inflow into the access tunnel after grouting is 290 l/min. The research tunnel, the main characterisation level, the demonstration level and the interconnecting tunnel are assumed to be located in such a tight rock that no sealing effect is assumed in this estimate. The inflow into the lower characterisation level was estimated to be 5 l/min after grouting. Thus, the total inflow into ONKALO after grouting was estimated to be 325 l/min.

7.2.2 Inflow into the shaft

The shaft of ONKALO intersects the structures R19A (-70 m), R19B (-110 m), R46 (-310 m), R20A (-320 m) and R20B (-360 m).

![Cumulative water inflow into the access tunnel of ONKALO](image)

**Figure 7-3.** Cumulative water ingress into ONKALO access tunnel with proceeding excavation.
The water inflow into the ONKALO shaft was calculated by summarising the inflow from intact rock and fracture zones without grouting and after grouting. The assumptions made in the calculations (transmissivities and grouting results) are the same as earlier.

Without grouting the inflow into the shaft from the intact rock mass is about 45 l/min and after grouting about 15 l/min. About half of the waters from the intact rock comes from the uppermost 200 m and one third from the depth interval -200...-300 m. The water inflows from the structures are presented in Table 7-3. The greatest inflows are expected to come from the structures R20A and R20B.

**7.2.3 Total water inflow into ONKALO**

Table 7-4. summarises how the water inflow into ONKALO is distributed between different parts and between intact rock and structures.

**Table 7-3. The water inflows from the fracture zones intersected by ONKALO shaft.**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Intersection depth (m)</th>
<th>Water inflow without grouting (l/min)</th>
<th>Water inflow after grouting (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R19A</td>
<td>-70</td>
<td>75.3</td>
<td>7.5</td>
</tr>
<tr>
<td>R19B</td>
<td>-110</td>
<td>47.1</td>
<td>1.2</td>
</tr>
<tr>
<td>R46</td>
<td>-310</td>
<td>83.7</td>
<td>33.3</td>
</tr>
<tr>
<td>R20A</td>
<td>-320</td>
<td>686</td>
<td>34.4</td>
</tr>
<tr>
<td>R20B</td>
<td>-360</td>
<td>194</td>
<td>38.7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>1090</td>
<td>115</td>
</tr>
</tbody>
</table>

**Table 7-4. Estimated groundwater ingress into ONKALO.**

<table>
<thead>
<tr>
<th>Water inflow (l/min)</th>
<th>Without grouting</th>
<th>After grouting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Access tunnel and interconnecting tunnel</td>
<td>Intact rock mass 437</td>
<td>147</td>
</tr>
<tr>
<td>Research tunnel</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Main characterisation level</td>
<td>44</td>
<td>4</td>
</tr>
<tr>
<td>Demonstration level</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Lower characterisation level</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Shaft</td>
<td>Intact rock mass 45</td>
<td>15</td>
</tr>
<tr>
<td>Structures</td>
<td>1090</td>
<td>115</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>2977</td>
<td>457</td>
</tr>
</tbody>
</table>
7.2.4 Inflow into the repository

The final repository for the spent nuclear fuel is planned to be constructed in several stages. Here the calculations are performed to the 1-level alternative VE400 KBS-3 for 6500 tons of uranium. The layout adaptation is presented in Figure 7-2.

The inflow due to the construction of the repository was calculated similarly than the inflow to ONKALO, i.e. summarising the calculated inflows per 100 tunnel-m. Sievänen (2003) estimated that at depth interval -400...-500 m, the inflow is about 0.7 l/min/100 m (remarkable fracture zones are assumed to be avoided). In the calculations it is assumed during one stage that part of the tunnels are open and the earlier parts are backfilled. The remaining parts are to be excavated later. The excavation of the central tunnel is planned to proceed in five stages.

In the calculations it is assumed that the inflow from the backfilled disposal tunnels into the central tunnel is 10% of the original inflow. Based on the above described assumptions, the cumulative inflow into the 1-level alternative VE400 KBS-3 for 6500 tons of uranium is presented in Figure 7-4. According to the figure the inflow from intact rock mass increase from about 35 l/min to about 110 l/min as the excavation and backfilling proceeds. Note that the decrease in inflow after the stage 8 is because the excavated tunnel length in stage 7 is longer than in stage 8, and thus after closing stage 7, the inflow slightly decreases.

![Cumulative water inflow into final repository, layout adaptation VE 400 KBS-3 6500 tU rev 3](image)

*Figure 7-4. Cumulative inflow into a 1-level repository, alternative VE400 KBS-3 for 6500 tons of uranium.*
Besides the inflow from the intact rock mass, the leakages are expected from some geological structures intersected by the tunnels. Major water bearing zones are to be avoided, but some minor zones have to be intersected. Sievänen (2003) estimated that the inflows from B- and C-class zones may be some litres per minute without grouting and some decilitres after grouting.
8 DESIGN PRINCIPLES FOR SEALING OF ONKALO AND THE REPOSITORY

8.1 General

The need for grouting is dependent on the requirements for water inflow and the geological environment and the depth. Sievänen (2002) has evaluated the general grouting conditions at the Olkiluoto site. In this context the preliminary design principles are presented for ONKALO, not for the repository. These design principles are based both on studies by Sievänen (2003), principal technical solutions presented in Chapter 4 and detailed examination made in this work for current layout of ONKALO (Chapter 7).

Based on estimations by Sievänen (2003) it seems evident that both "intact rock" and the structures need to be sealed effectively. Best value for sealing can be achieved by pregrouting the individual structures. Sealing of the intact fractured rock needs a lot of work because every 100 m of tunnel is estimated to include from few to several such fractures that need to be grouted.

8.2 The need for grouting in ONKALO access tunnel and characterisation tunnels

8.2.1 Structures

According to the current plans (near-field alternative) and the latest picture of bedrock the ONKALO access tunnel and characterisation tunnels intersect the structures/hydraulic conductors R24, KR4_1H, R19A, R19B, R46, R20A, R20B, R17B and R56. Some of the structures will not be intersected perpendicularly. Instead, the structure is to be intersected nearly parallel and this is taken into account when studying the length of the tunnel to be grouted. Besides this, the tunnel goes near (< 20 m) the structures R19B at the depth of -30...-50 m and R20B at the depth of -380...-410 m. These structures have to be taken into account when designing grouting, because both are major water bearing zones and may conduct large amount of water into the tunnel via small, single fractures.

The tunnel sections where the grouting of structures is supposed to take place are collected in the summarising Table 8-1.

8.2.2 Intact rock mass

When estimating the need for grouting of the intact rock mass, the average distances of the water conductive \((K_{2m} \geq 1 \cdot 10^{-8} \text{ m/s})\) fractures were studied. The used data consist of the hydraulic measurements made in 13 investigation holes. The analyses are presented by Sievänen (2003, Figure 2-11), where the study was made for every 100 m depth interval. To understand the figure it have to be notified that 1) both the conductive
Table 8-1. Summary table of sealing principles in ONKALO access tunnel, interconnecting tunnel and characterisation tunnels.

<table>
<thead>
<tr>
<th>Depth interval</th>
<th>Structure to be intersected</th>
<th>Tunnel length to be grouted (m)</th>
<th>Sum (m)</th>
<th>Proportion of intact rock (m)</th>
<th>Pregrouting of intact rock (%)</th>
<th>Pregrouting length (m)</th>
<th>Sum of pregrouting (m)</th>
<th>Total pregrouting (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Access tunnel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-50</td>
<td>R24</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>KR4_1H</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50-100</td>
<td>R19A+R19B</td>
<td>450</td>
<td>745</td>
<td>255</td>
<td>100</td>
<td>255</td>
<td>1000</td>
<td>100</td>
</tr>
<tr>
<td>100-200</td>
<td>R19B</td>
<td>170</td>
<td>170</td>
<td>830</td>
<td>60</td>
<td>500</td>
<td>670</td>
<td>67</td>
</tr>
<tr>
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<td>R20A+R46</td>
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<td>715</td>
<td>485</td>
<td>20</td>
<td>95</td>
<td>810</td>
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<td>1960</td>
<td></td>
<td></td>
<td></td>
<td>3555</td>
<td>46</td>
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</table>
sections in the intact rock and structures are included (in later study tried to eliminate), 2) if there are two or more water conductive sections consecutively they are calculated to be one water conductive section, 3) there are artificial breaks due to the data processing at every 100 m intervals, which are seen as water conductive sections in the figure and 4) it is not possible to see how the water conductive sections are distributed in the bedrock. It is thus estimated that the figure presented by Sievänen (2003) gives slightly conservative idea of the need for grouting.

The basis for the calculations are presented in Table 8-2. It was estimated by Vaittinen et al. (2001) that a structure occurs in investigation holes about every 100 m intervals. This is taken into account in these calculations. The effective length of grouting fan was estimated to be 15 m. The need for pregrouting in the intact rock mass is presented as a percentage of tunnel length.

The need for postgrouting was estimated to be about 25% of pregrouted tunnel. This means the grouting of sporadic minor leakages (drops) and leaking rock bolt holes.

Table 8-2. The basis for calculating the need for pregrouting of intact rock mass in ONKALO.

<table>
<thead>
<tr>
<th>Depth interval (m)</th>
<th>Average distance between water conductive sections (m)</th>
<th>Number of water conductive sections (pes)</th>
<th>Number of water conductive sections without structures (pes)</th>
<th>The proportion of tunnel to be pregrouted (%)</th>
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</thead>
<tbody>
<tr>
<td>0...-100</td>
<td>~ 10</td>
<td>10</td>
<td>9</td>
<td>100</td>
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<tr>
<td>-100...-200</td>
<td>~ 20</td>
<td>5</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>-200...-300</td>
<td>~ 30</td>
<td>3</td>
<td>2</td>
<td>30</td>
</tr>
<tr>
<td>-300...-400</td>
<td>~ 45</td>
<td>&gt; 2</td>
<td>&gt; 1</td>
<td>20</td>
</tr>
<tr>
<td>-400...-500</td>
<td>~ 60</td>
<td>&lt; 2</td>
<td>&lt; 1</td>
<td>10</td>
</tr>
<tr>
<td>-500...-600</td>
<td>~ 60</td>
<td>&lt; 2</td>
<td>&lt; 1</td>
<td>10</td>
</tr>
</tbody>
</table>
8.3 The need for grouting in the ONKALO shaft

The shaft of the ONKALO intersects the following structures:

- R19A at depth of -70 m
- R19B at depth of -110 m
- R46 at depth of -310 m
- R20A at depth of -320 m
- R20B at depth of -360 m.

The shaft of ONKALO could be grouted for the whole length. The water inflow from the uppermost 150 m of the shaft has been estimated to be about 20 l/min after grouting and from the section (-150...-300 m) less than 5 l/min. The lower part of the shaft (< 300 m) is estimated to conduct 100 l/min.

In the lower part of the shaft the grouting should be completed with a hydrostatic structure in the depth interval -300...-380 m to seal the geological structures. With the hydrostatic lining in the lower part of the shaft, the inflow into the shaft is estimated to be about 25 l/min. By constructing a hydrostatic structure in the upper part of the shaft it seems that under these circumstances the obtained reduction in the inflow could be scarce. The decision for constructing this kind of massive and expensive lining structures in the shaft should finally be made based on measurements and observations during and after raiseboring of the centre shaft and before starting the enlargement excavation.
9 DISCUSSION

9.1 Postclosure performance

A balance needs to be found between avoiding disturbances due to leakages and, on the other hand, due to grouting and other preventing methods. The following points might be helpful in this task, which is not all straightforward.

- The foremost method to limit leakages is to carefully select the locations of the underground facilities and their surface connections, i.e. avoiding intersections with major fracture zones.
- The option to remove concrete structures before the final sealing of the repository should be taken into consideration.
- In grouting and other places, where removing of cement before the closure of the repository is not possible.
  - Low-alkaline cement should be used if practicable.
  - Non-beneficial and unnecessary use of ordinary and also low-alkaline cement should be avoided. Grouting should be made in places where it most effectively reduces inflow of groundwater.
- The subhorizontal fracture zones R19 and R20 divide the flow pattern into layered zones (Vieno et al. 2003). Therefore, cement below the fracture zones is of a greater concern than cement remaining above or in the fracture zones themselves. In the early phase of the construction in the upper part of the bedrock, above the subhorizontal fracture zones, focus may be put on limiting the inflows as low as possible and learning effective sealing methods of rock in the actual in situ conditions. Based on the experiences and on the ongoing development of alternative grouts, the practices shall be reviewed.
- Degradation of cement-based injection materials in the long term should be taken into account in the design of the array of injection boreholes in the vicinity of sealing structures.
- Effects of superplasticizers and other organic additives of grouts need to be studied. Effects of potential non-cementitious grouts shall be studied as well, if they are to be used.
- The use of colloidal silica (silica sol) as grouting material would necessitate evaluation of its potential harmful effects as concerns mineralogical alteration in the bentonite buffer and as a source of colloids in the geosphere.
- Mitigating of the drawdown of groundwater table by artificial infiltration of fresh water into the bedrock is not recommended.
- Experiences obtained in the upper parts of the excavations shall be evaluated and utilised as the construction proceeds closer to the planned disposal level.

The recommendations to restrict the use of ordinary and even low-alkaline cement, and organic and colloidal grouting materials are based on the present, limited knowledge of their potential harmful effects. It may be possible to reduce the uncertainties and to cease to the associated recommendations for restrictions, but only by means of extensive further studies.
9.2 Design and construction

The present conception is that only small amount of groundwater leakages can be tolerated at the repository because of their harmful effects on the geohydrology and geochemistry. In addition, it is evident that high groundwater leakages would create problems for repository construction, operation and closure. However, it is not clearly understood what would be the acceptable level of the leakages.

In designing and construction of ONKALO and the repository best available technology with proven and robust solutions should be utilised. Principles of the best available technology in controlling of groundwater leakages are described in Chapter 4. Changes caused in rock and groundwater state should be measured during construction and sealing techniques should be sized in order to prevent harmful disturbances. Construction phase of ONKALO should be used for developing grouting and other sealing techniques as well as methods for estimating leakage rates and possible disturbances.
REFERENCES


APPENDIX 1

Description of the R-structures intersected by ONKALO access tunnel and characterization levels (according to bedrock model 2001/2)


Structure R17

Structure R17 (160°/20°) consists of two parts (A ja B) and it occurs at the depths -320…-640 m. The thickness of R17A is 4 m and of R17B is 20 m. It is directly observed in boreholes KR10 and KR4.

Structure R17A includes RiIII-IV sections and the fracture density is 5...>12 fractures/m. Microfractures are observed. 70% of the fractures are open and/or filled and the infillings are calcite, chlorite, kaolinite and illite. Thickness of the infillings varies between 0.1...1.8 mm. Fracture surfaces are typically rough or semi-rough.

Structure R17B includes also RiIII-IV sections and the overall fracture density is 0...>10 fractures/m. Open and filled fractures (85%) are observed (chlorite, illite, calcite ja kaolinite, thickness 0,1...1 mm). Fracture surfaces are typically semi-rough. The measured logT of the structure is −7.8.

Structure R19

Structure R19 consists of two parts: A (360°/3°) and B (180°/24°) (depth range -50...-280 m). The observed thickness is 2 m. It is directly observed in boreholes (KR4 ja KR8).

R19A is classified as a RiIII zone and the fracture density is between 2...> 13 fractures/m. The fracture density in R19B is 3...6 fractures/m. Fractures are open and/or filled. (clays, chlorite, calcite). The measured logT values of R19A vary between −4.7...−4.5 (2 measurements) and of R19B between −5.8...−4.7 the median being about −5.3 (4 measurements).

Structure R20

Structure R20 is composed of three parts (A (138°/17°; depth range -150...-500 m; thickness 2...32 m), B (138°/17°; -170...-500 m; 2...11 m), and C (no intersection). R20A is intersected by four boreholes, like R20B, too.

In R20A there are RiIII, RiIII-IV and RiIV zones. Fracture density varies between 0...≥ 30 fractures/m. Fractures are open and/or filled (calcite, clays, chlorite; thickness 0.1...0.9 mm). Fracture surfaces are typically semi-rough or rough. The measured logT values are −4.1 and −4.9.
Structure R20B includes RIII and RIII-IV fracture density being 1...≥ 22 fractures/m. Fractures are similarly open/filled as in R20A. Fracture surfaces are usually semi-rough. LogT varies between −5.0 and −4.7 (2 measurements).

Structure R24

Structure R24 (A and B) (144°/47°) will be intersected near the surface, nearby the start of the access tunnel. The thickness is observed to vary between 2...5 m. R24 is classified as RIII-IV zone fracture density being 1...18 fractures/m. Fractures are filled (clays, chlorite and calcite). LogT value is −4.2.

Structure R46

In structure R46 (090°/25°) the fracture density is 1...8 fractures/m. Fracture infillings are clays, chlorite and calcite. The logT value is measured to be −5.6.

Structure R56

Structure R56 (190°/60°) is a RIII zone, in which the fracture density varies between 0...> 20 fractures/m. Fracture infillings are clays, chlorite and calcite, which are in places over 0.5 mm thick. The measured logT is −8.2.